Abstract

The thesis examines two projects, a grain silo at Corby, Northamptonshire, and a store with a basement at Basildon, Essex. The two sites are examined from their conception, through the site investigation, design, construction and finally reports on the completed structures.

In the case of the grain silo at Corby the contract was termed "fast track" which meant that the designers and contractors were on a very tight schedule to complete the structure. The thesis discusses the implications that this had on the decisions reached regarding the foundation and pile design as the site investigation specification had not included obtaining parameters for the founding limestone stratum and the underlying Lias Clay.

The first pile test at the Corby site resulted in a failure and it showed that heave was a problem for the closely spaced piles. The piling contractor stated that pre-boring to any depth was not necessary. The results from a re-tapping exercise show that pile heaves were the norm and that even partial pre-boring made little difference to the magnitude of the heave.

The complex was monitored by precise levelling. The indications from settlement v log(time) plots are that only about 40% consolidation had occurred at the last date of measurement. Even so the indications are that the design method overestimated the magnitude of the initial settlement by at least a factor of 4. A simple back analysis would suggest that this is primarily a result of a low value of Young's Modulus calculated from the laboratory work. It is also suggested that a better understanding of the stiffness of the overall structure would lead to a better prediction of across slab differential settlement.
The Basildon case record looks at the design principles behind allowing substantial load sharing between the piles and the raft. The thesis highlights the fact that there was not the correct guidance for the site investigation to produce those parameters necessary for a particular design. A stiffness value for the London Clay was not determined and the ground water conditions in the area of a deep basement were not monitored prior to construction.

A long term design condition of maximum water pressure was present during construction when there were minimal dead loads to resist the upward force. It is felt that the construction method of taking all the dead load through the falsework scaffold system helped to resist the slab doming the water pressure induced. As a result the piles were taken into tension at a stage in the construction process when they were designed to be in compression. The thesis proposes that the water pressure present has domed the slab such that the raft cells have broken contact with the London Clay and as a result are now weighing themselves in the form of "negative" effective stress readings.

The method of design has predicted safe loads in the piles with the exception of the corner piles. The design load for the corner piles appear to have been exceeded. The compressive steel in the pile was able to resist a tensile load of this magnitude and no damage resulted.
Acknowledgements

The contract was carried out under Professor NE Simons and Dr CRI Clayton and my thanks extend to them for employing me on the SERC contract.

Prof. NE Simons and Dr CRI Clayton, acted as my supervisors; their discussions and guidance are much appreciated. My thanks must also extend to Mr MCG Matthews and Mr MA Huxley, Lecturers in Geotechnical Engineering, whose discussions were of great benefit.

Marion Bryant (now Wicks), Assistant Experimental Officer, must be thanked for the times she undertook the long day out to Corby with me only to find that once again it was cold with high winds and rain.

The owners of the Flour Mill at Corby and the Consulting Engineers have fully cooperated in supplying the facility and the design detail, Mr L Carvalho in particular must be thanked. Permission, by all concerned, to use the data is appreciated.

The Basildon Development Corporation and the Consulting Engineers have fully cooperated in supplying the design detail and the monitoring detail they had, Mr B Mainie is thanked in particular for his time and enthusiasm. Permission, by all concerned, to use the data is appreciated.

The levelling at Basildon and the interpretation of the Pile and Slab Load Cells was undertaken by Mr GW Price of the Building Research Station.
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1.0 Introduction

It has been generally recognised, over the past two decades, that there is a shortfall in the knowledge in field behaviour of structures on piled raft foundations. The two structures, a large mill at Corby and a Store at Basildon, have afforded the opportunity to publish data comparing design predictions with field measurements.

The Flour Mill is a very heavily loaded, large structure. It is situated in the Corby area and is founded on the site of a backfilled open cast iron ore mine. The foundation slabs are sited over some 9m of uncompacted, largely cohesive, fill material with the piles founded on 2 to 3m of Limestone. The Limestone overlies Lias Clay to considerable depth.

The design predictions for Corby are compared with field measurements of settlements. The live loads, which are substantial, are monitored by central computer for the stock control of each of the silo bins in the complex. From these two sets of data a unique picture of load and settlement against time can be built up for a number of points across the structure.

The thesis includes discussion on the design process for the foundations with particular reference to the heavily loaded Wheat Silo Foundation Slab. It highlights the problems in assessing the appropriate parameters for the soil and structure for use in modelling the soil-structure interaction. Perhaps more importantly, the thesis highlights difficulties that may be encountered when the design and construction process is undertaken at very great speed, so that in the future such problems may be avoided. The decisions made during the design process, which overlapped the commencement of construction, are discussed in the light of the knowledge as it became available. Recommendations are made for parameters which are suggested by the actual performance of the structure. Finally the
observed performance of the structure has been back-analyzed in order to examine the magnitude of the parameters necessary to give a good prediction for the structure's behaviour.

The Alders Building at Basildon is part of a large shopping complex. The complex was developed by the Basildon Development Corporation as an entity and as such entailed a high degree of inter-connectivity. There are expensive finishes throughout which would all be sensitive to differential settlement. The complex at Basildon is founded on London Clay to considerable depth. To reduce differential settlement between all of the buildings all of the foundations were piled. This was particularly important for those structures adjacent to the two deep basement foundations.

The Alders department store has a superstructure comprising a reinforced concrete frame with brick cladding. Columns are arranged on a 10.8m by 9.6m grid. The foundation comprises a piled-raft formed in an excavation 50m by 70m in plan and 5m deep.

Field measurements for settlements, pile load, raft effective stress and water pressure are compared with the design expectations.

The thesis includes discussion of the design process with particular reference to the Alders basement. It highlights the design process involved in producing a safe yet innovative and more importantly economic design for the raft slab taking into account the load sharing potential for a piled raft solution. The readings for the raft cells are discussed as some of the cells are reading "negative" effective stress. It is suggested that this might be due to the cells hanging from the underside of the floor slab as it domes due to the water pressure. This will leave the floor cells weighing themselves. The field data are investigated in conjunction with the site diary to investigate the actual construction loading. This loading regime is then compared with that assumed in the design. Finally the report has
highlighted the occurrence of an atypical situation in that full water pressure is not normally expected in the London Clay for some time after an excavation is closed. In this case it would appear that substantial water pressure was present during the construction of the basement prior to it being backfilled.
2.0 Corby Case Record

2.1. Site Details

Corby is situated in the north of Northamptonshire, its position in relation to Northampton, Peterborough, Leicester and Nottingham is shown in Fig.2.1/1. The area around Corby has, until recent times, been quarried for iron ore using open cast mining techniques. The quarry was worked in 20m strips with a large walking dragline which stood below ground surface level on a layer of limestone to remove the overburden above the iron bearing strata. The over-burden was tipped into the adjacent "worked" strip on top of the "unworkable" fraction of the iron bearing strata forming the characteristic "hill and dale" formation, Plate 2.1/1. At some time later the site was levelled leaving some 10m of uncompacted fill overlying the unworkable fraction of the iron bearing strata. The quarried area was then returned to agricultural uses. With the expansion of Corby pressure grew to develop these areas for housing and industrial usage.

2.1.1. The Geology of the Locality

The original lithology in the vicinity of Earlstrees Industrial Estate, Corby (Approximate National Grid Reference SP 893 908) is thought to be as follows. The interpretation is made from Sheet 171 "Kettering and Corby" Geological Survey Map in conjunction with Taylor (1963).

Thickness  Stratum and Description

Up to 15m:- Boulder Clay

Chalky-Jurassic boulder clay and sparse representatives of an older largely chalk-free till
minimal  
Upper Lincolnshire Limestone
Coarse skeletal oolite, locally current bedded

up to 12m: - Lower Lincolnshire Limestone
Oolitic & pellet limestones. Fissile sandy limestone locally at base

4.5 to 7.5m: - Lower Estuarine
Pale grey or yellow brown sand argillaceous beds.
Pale grey sands, silts or silty clays.

Northampton Sand Ironstone

2.5-4.5m
Upper Chamosite-Kaolinite Group
Upper Siderite-Mudstone-Limestone Group
Lower Chamosite-Kaolinite Group Main Oolitic Ironstone Group

2.5-3m
Lower Siderite Mudstone Group

49-58m: - Upper Lias
Grey Clays occasional cementstone nodules - two thin limestone bands separated by paper shales at base

The thickness of the original strata above the Lower Siderite Mudstone Group of the Northampton Sand Ironstone is not known and hence the ranges according to Taylor(1963) for sheet 171 have been indicated. Geological detail for the above strata is available in the memoir to sheet 171 (Taylor 1963).

2.1.2. The Effect of Mining.

The Northampton Sand is mainly composed of rocks of the two lowest subdivisions, the Main Oolitic Ironstone Group and the Lower Siderite Mudstone-Limestone Group. The so called "bastard zone" (normally 2 to
3.5m), left in the floor of the ironstone quarries corresponds in
general to the Lower Siderite Mudstone-Limestone Group. The "workable
stone" belongs for the most part to the Main Oolitic Ironstone Group
(usually 3 to 4m). Locally it includes (at the top) 300mm or so of
sideritic mudstone, sometimes sparsely oolitic, belonging to the Upper
Siderite Mudstone-Limestone Group. The Lower and Upper
Chamosite-Kaolinite Groups are poorly developed within the area.
Within the general area the maximum thickness of the Northampton Sand
recorded is 7.5m. Commonly the formation is 4.5 to 6m thick. As the
lower part of the Northampton Sand is almost always unworkable as
ironstone, the custom has in general been to stop the mining at a
depth of 4.5 to 6m dependent upon the analysis of the workings (Taylor
1963).

The practice was to drill and blast to the depth of the workable ore,
excavating the shattered material. This had the effect of fracturing
the remaining material left unexcavated, leaving a stratum of altered
characteristics, affecting the stiffness of the stratum and possibly
affecting the bearing capacity. The additional fractures induced by
the blasting are likely to be clean unlike those naturally occurring.
Thus whilst it was exposed the rate of weathering might be increased
due to the presence of the blast induced fractures. This would reduce
the stiffness of the stratum further.

The made ground, composed of the uncompacted fill material is a
mixture of the strata above the unworked Northampton Sand Ironstone
and is a result of the method employed to win the workable ore. The
quarry was worked in 20m strips with a large walking dragline standing
on the Lincolnshire Limestone (see Fig.2.1/2). The overburden was
tipped into the adjacent strip forming a characteristic "hill and
dale" formation, Plate 2.1/1. The procedure was carried out in two
phases:
1) With the dragline positioned on the Lincolnshire Limestone the overlying Boulder Clay was stripped (Fig.2.1/2).

2) The Lincolnshire Limestone then required blasting and was stripped along with the Lower Estuarine Series to expose the Northampton Sand Ironstone formation.

The two stages were organised such that the fragments of the limestone strata were tipped into the adjacent worked strip directly onto the unworkable Ironstone strata. The Boulder Clay and Lower Estuarine were then tipped indiscriminately onto the blasted fragments of the Lincolnshire Limestone. The workable element of the Northampton Sand Ironstone strata was shattered to the required depth and removed by face shovel. Thus the generalised succession of the made ground can be summarised as follows:

1) Fragments of Lincolnshire Limestone in direct contact with the lower unworkable element of the Northampton Sand Ironstone

2) A mixture of Boulder Clay and Lower Estuarine strata in uncompacted form. The mixture is generally cohesive in nature.

The fill was left in its "hill and dale" formation and surveyed by aerial photography. At some time later the site was levelled by scraper and had in the past been restored to agricultural use. It was general practice to install drainage to dewater the site whilst the open cast excavation was in operation. Usually this was left in place during the backfilling which has the effect of keeping the water table just below rockhead.
2.1.3 Implications of the Geology and the Literature on the Site

Investigations

An examination of the above geology and the published literature suggests that foundations at this location should be piled. Problems with differential settlement are reported on monitored housing projects on these backfilled open-cast mines (Penman and Godwin 1974, Charles, Naismith and Burford 1977 and Charles, Earle and Burford 1978) where the differential loading across the bearing slabs is small. The structures at the Flour Mill consisted of very heavily loaded foundations interconnected with slabs, which in comparison, are lightly loaded. With this large differential loading, the above literature indicates that the load should be taken down to the rock stratum to avoid differential settlement. The aim of the site investigation must include the following two objectives for the design of the foundation. These are

1) to establish the parameters for the layer of "unmined" Northampton sand ironstone suggested in the literature. Of particular importance are the strength of the material onto which piles may be founded and the thickness of the layer. A knowledge of the likely toe stress, the strength of the limestone and the thickness of the limestone will then allow an engineering decision to be made.

2) to establish the parameters for the Lias Clay so that,
   a) the stratum can be checked for bearing capacity failure against the stress transmitted through the Limestone as a result of the load from the piles,
   b) the stratum can be evaluated for the piles being founded on or in the Lias Clay should the Limestone not be present.
Added to this, account should be made of the negative skin friction that will increase the design pile loads (Burland, 1973 and Charles and Burland, 1982).

2.1.4. Site Investigation

As is usual in such cases, time and the financial constraint dictated the sequence of events. It would appear from the correspondence, site construction, design and site investigation activity that time was indeed short.

An initial site investigation, was carried out in April 1981 with the report submitted in July 1981. The report cast doubts on the validity of the design of the structure as question marks were raised about the founding stratum for the proposed end-bearing piles.

A geotechnical consultant was engaged to advise on the likely settlements. A second site investigation was recommended to investigate the problems raised by the initial site investigation. The investigation and the report were completed in August 1981, but it had been necessary to start the piling contract in early August 1981. The purpose of the second investigation was now to confirm the parameters, assumed for the pile design, as being realistic. Should the parameters prove inappropriate then procedures to confirm the pile design by pile testing were to be proposed.

The Initial Ground Investigation.

The initial ground investigation was carried out to investigate the ground conditions and provide disturbed and undisturbed samples for subsequent laboratory testing. The investigation comprised of 7 No. Shell and Auger light percussion and 2 No. rotary drilled boreholes. The location of these borehole with respect to the proposed structure are shown in Fig.2.1/3.
Laboratory Testing.

The following types of laboratory tests were carried out on the samples:

1) Unconsolidated Undrained Triaxial Tests
   15 No. on the fill material
   2 No. on the Lias Clay

2) One-Dimensional Consolidation Tests
   6 No. on the fill material

3) Atterberg Limits
   4 No. on the fill

Figs.2.1/4 to 7 shows a summary of the borehole logs and Tables 2.1/1 to 7 summarise the results of the laboratory testing.

Findings and Recommendations of the Initial Ground Investigation.

The report on the ground conditions as reported in the site investigation report is duplicated below.

CLAYEY FILL varying in depth from 8.4 to 9.7m below existing ground level. The fill appeared to be Boulder Clay in origin with a generally soft to stiff consistency. Fig.2.1/8 shows that there appeared to be no apparent gain in strength with depth. Fig.2.1/9 shows that there appeared to be no decrease in coefficient of volume compressibility with depth.

LIMESTONE, varying in thickness between 2.5 and 3.9m. Extrapolated SPT results from one borehole indicated a
weak rock. Inspection of core and one unconfined compressive test result indicate moderately weak to moderately strong rock.

UPPER LIAS CLAY, proved to a maximum depth of 19m below ground level in one borehole. Two unconsolidated undrained triaxial test results indicated an undrained shear strength lower than was expected by the site investigation firm as they stated their experience would have assigned an undrained shear strength of 400 to 600kN/m². Hand penetrometer results and moisture content determinations indicated seams of much softer material.

From the findings of the laboratory tests the following points were made:

1) The ground conditions suggested that the foundations should be piled.

2) With the presence of the limestone stratum the piles should be designed as end bearing piles in the limestone ignoring skin friction derived from the clay fill. From initial calculation, a 750mm diameter shafted bored pile drilled 1m into the rock would give a safe load of 1325kN.

This figure was obtained by extrapolating the SPT blow counts through the Inferior Oolite to give values for full penetration. This gave an $N = 120$ which was related to an undrained shear strength of $1000kN/m^2$. Hence the ultimate bearing capacity would be equal to $9 \times 1000 = 9000kN/m^2$. With a factor of safety of 3 this gives an allowable bearing capacity of $3000kN/m^2$ considered by the site investigation company to be applicable to piles drilled about 1m into the rock. They pointed out that for surface loading of the rock from bored piles the figure should be reduced to about $1500kN/m^2$. Hence for a 750mm
diameter straight shafted bored pile, with an adequate socket into the Inferior Oolite, the safe load is 1325kN.

3) As this would leave some 1.5 to 2.0m of limestone below the pile tip the Lias Clay would be loaded. The design should check the bearing capacity and settlement characteristics of the Lias Clay.

4) An allowable bearing capacity for the Lias Clay would be in the order of 580kN/m² if the undrained cohesion were taken as 260kN/m² for a factor of safety of 3 for a deep foundation.

5) The weathered seams indicated by the moisture content and hand penetrometer readings whilst not reducing the bearing capacity of the foundations would probably increase the total settlements.

6) Consideration of 5) should be given to pile spacing in that the bearing pressures on the clay beneath the limestone should not exceed 400kN/m².

7) A controlled system of redriving would be necessary due to heave that would be expected as a result of close pile spacing.

8) Negative skin friction on the pile shaft would be expected as a result of consolidation of the pile as the pore pressures, due to piling, dissipated. An allowance should be made or partial elimination should be provided in the form of a slip membrane.

It can readily be seen that the end conclusion of the site investigation was the same as that outlined in the section
"Implications of the Geology and the Literature on the Site Investigation". However the rock had not been fully proved and the Lias Clay had not been investigated and as such the piles could not be designed. The investigation was not sufficiently comprehensive to provide adequate parameters for the design. It has already been indicated that planning at the desk study stage would have highlighted those parameters necessary to found 100tonne piles. It could also be said that the Site Investigation firm should have realised during the site work that the method of acquiring the rock core was not producing samples of sufficient quality and that the sampling method required changing. The outcome was that the investigation did not provide the required parameters. Time was lost. The need for the second investigation and the loss in time could both have been avoided if:

a) there had been adequate planning,

b) the site investigation company had demonstrated that the drilling method employed to obtain the rock core was not suitable.

It is easy to criticize the site investigation contractor on the results he presents. However, it must always be remembered that, for the most part, he is working to instruction. It would appear obvious that a structure founded on closely spaced 100tonne piles would load the Lias Clay. A stratum significantly loaded should be sampled in a number of locations and to the depth of the stress bowl. The site investigation reported only two laboratory results from the Lias Clay. Indeed it only reported one laboratory result on the limestone, the other stratum likely to be heavily loaded by the proposed piles. The responsibility for these inadequacies must be borne by those who issue the instructions as well as those who carry out the work. Pointing out the shortfalls before the sitework is complete, and the situation cannot be rectified without remobilization of the site investigations plant, appears obvious but how often it is done and taken note of must
be questioned. The responsibility of the presentation of the report and the data contained lies with the site investigations firm. The fill material was sampled, as per specification, to determine the settlements of services to the structures and to give an indication of the likely skin friction attracted to the piles due to consolidation of the fill. No results were reported for these items. The unconsolidated undrained triaxial tests on fill material and on the Lias Clay mention no specimen size. It must be assumed that the specimen size was 38mm diameter as the sampling was carried out in the fill material using light percussion, the core size was U100, and there were 3No. samples reported for each depth. The rotary coring sample size is not quoted but again as 3No. specimens were tested at each of the two depths 38mm specimens are indicated. The specimen size for the rock core is not so easy to determine. The International Standard for Rock Mechanics suggests a minimum of 54mm, but smaller core diameters might be acceptable dependent upon the grain size.

Apparent Angles of friction were quoted for the unconsolidated undrained triaxial tests on the fill (Tables 2.1/1 & 2.1/2) and on the Lias Clay (Table 2.1/5). The comment in the Report is that these angles indicate that not all the specimens were saturated but they may have further implications. They may result from fissuring, poor sampling, handling of the samples or poor production of the specimens. Each of the above can result in the opening of fissures to a different degree on the three specimens. On overconsolidated specimens, leaving a specimen overnight in the cell will result in an uptake of water resulting in a drop in the load at shearing. The unconfined compressive strength test on rock core can hardly be commented upon with only one result. Suffice is to say that the Deviator Stress was misquoted in kN/m² and not MN/m² or MPa. The One Dimensional Oedometer Test on the fill material merely indicates the tests were undertaken at a stress increment of 100kN/m². With a material whose bulk density is reported to range from 15.5 to 20.7kN/m³ it can only be helpful to give the test vertical stress values such that the appropriate value
of $m_v$ can be used in any calculations. The likely vertical stresses have been added to Table 2.1/4 assuming a bulk density of 20kN/m$^3$.

Overall the investigation does not give confidence on two major counts.

1) The quoted objective of the investigation was to investigate the ground conditions and to discuss the findings with a view to founding 100 tonne piles and the likely settlement of the recently placed fill. The investigation was concerned with the fill material and indicates no planning concerning the founding of the 100 tonne piles. The responsibility for this must remain with the consultant. It has been indicated earlier that the desk study would have indicated the correct areas to sample. The work undertaken at the desk study stage must be economic in comparison to the cost of a second investigation and more important the loss of time.

2) The sampling, sample handling and specimen preparation all give rise to doubt. The points in the above section concerning the laboratory results are all mentioned in the discussion and recommendations, however, they could as already stated be due to the sampling method or the subsequent handling.

The Need for Further Site Investigation

Time was now of the essence and the initial site investigation raised a number of question marks over the pile design. The consulting engineer engaged a geotechnical consultant on the following brief (communication between the consultant and the geotechnical consultant):
1) To advise on the likely settlements such that provision could be made for the bending moments induced as a result of differential settlement.

2) To make any further observations that may be relevant in respect of the soil-structure interaction.

3) If further soil tests are required to make prognoses to advise of the type of test.

An urgent response was requested as the consulting engineer was on a tight programme to produce working drawings. The response from the geotechnical consultant was as follows (communication between the geotechnical consultant and the consultant).

1) The site investigation document seemed mainly concerned with the clay fill and there was very little information relating to suitable founding strata for such a heavy and sensitive structure.

2) The thickness of the limestone strata was known at only three positions giving thicknesses of 3.9, 3.6, and 2.5m. It was felt that it was unlikely that this has shown the minimum thickness and thus the degree of variation could not be assessed.

3) The records of the boreholes indicate that the limestone was suitable as a founding strata if present in sufficient thickness. Descriptions indicated a lower quality as silty clays were present at some locations.

4) There was no adequate information on either the strength or compressibility of the Lias Clay, thus no calculation of the total differential settlement was possible.
5) Borehole depths were inadequate. If the structures were to be founded on the inferior oolite limestone the subsoil would be significantly stressed to 30 to 40m below ground level.

6) Proposed pile toe stresses were quoted at about 4500kN/m². Thus it was essential that the quality and the thickness of the limestone should be established to avoid punching failure.

7) From the only information available for the Lias Clay, descriptions and pocket penetrometer tests indicated a variable and sometimes compressible material. If these records were representative of the soil insitu, differential settlements far in excess of the 4 to 5mm tolerable might be expected.

8) Further site investigation was considered essential. At least 6 further holes should be rotary drilled to obtain good quality core of the limestone and to determine its thickness. At least 2 holes should be drilled into the Lias to a depth of 30 to 40m to obtain good quality samples for visual inspection and laboratory testing. It was further recommended that it would be possible to extend the cover of the investigation at relatively little additional cost using geophysical methods.

9) Further site investigation should be carried out under a strict specification with good site supervision.

A meeting between the consulting engineers and the geotechnical consultant at this time further added to the above (communication between the geotechnical consultant and the consultant confirming the discussion).
a) The quality of the Lias Clay insitu was likely to be better than as seen in the first site investigation. The material could well have been softened or remoulded during drilling.

b) The further site investigation would be carried out by a second site investigation contractor. That the rotary drilling should be carried out using bentonite flush, "P" and "S" size barrels, horizontal extrusion of core using a core plug and good sealing against moisture loss.

It should again be noted that the second site investigation work was carried out at the same time as the piling contract commenced installing 533mm diameter prebored driven and insitu shell piles at 1.5m centres working on a 4.5MN/m² toe stress.

The Second Site Investigation

The site investigation was carried out in August 1981 under the following guidance from the geotechnical consultant (communication between the geotechnical consultant and the site investigation firm).

Objectives of the site investigation: Since the site was to be used for a mill complex and the foundations would be piled the following were required.

i) Knowledge of the precise thickness of the rock.

ii) The depth from ground surface to the rockhead.

iii) Top quality core of the rock for laboratory testing.
iv) Top quality core of the Lias Clay, underneath the rock, for laboratory testing.

v) Some properties of the clay fill from the ground surface to rockhead.

The typical soil profile for the site and the drilling instructions were issued as follows (communication between the geotechnical consultant and the site investigation firm):

G.L. to 8m
Shell and Auger Boring. No samples were to be taken except in BH2 where 6No. U100 samples were required between ground level and 8m. Should rock have been encountered above 8m then rotary drilling was to start at that level. Chiselling was not permitted except at shallow depth. The limestone was thought to be typically only 2.3 to 2.9m thick and of barely sufficient thickness to support the piles proposed for the structure.

8 to 15m
Rotary Core, PF size with bentonite mud flush. The mud flush was to be kept thick when drilling the Lias Clay. Minimum downthrust was requested when drilling the Lias Clay, as excessive downthrust tends to break the core into "discs". This was essential if the Lias Clay were to be proved as being of very good quality. The core was to be carefully sealed in its Mylar sheet using plastic tape and waxed end plugs of muslin to prevent loss of moisture during storage. Care during boxing was specifically requested to avoid breaking the core. Drilling records were to be sent to the geotechnical consultant daily, stressing the urgency required as time was short.
All specimens were to be returned to the geotechnical consultant for laboratory testing and retention.

**Laboratory Testing**

It would have been desirable to undertake testing in addition to those reported below. Unfortunately this was not possible in the two weeks available to undertake the laboratory testing and settlement analysis. With this in mind the majority of the testing was completed on the borehole core first received.

Uniaxial Unconfined Compressive Strength Tests on Rock Core 23 No. (Table 2.1/8 and Fig.2.1/10)

Undrained Triaxial Compression Tests on Lias Clay 13 No. (Table 2.1/9 and Fig.2.1/11)

Drained Reload Triaxial Compression Tests on Lias Clay 2 No. (Table 2.1/11)

One Dimensional (Oedometer) Tests on Lias Clay 12 No. (Table 2.1/12 and Fig.2.1/12)

The drained reload triaxial tests on the Lias Clay each lasted 12 days. The object of the test was to provide a Young's Modulus value unaffected by any loosening of fissures or bedding between the sample and the platens. The oedometer tests were carried out as the rate of test for the drained reload test precluded sufficient testing to assess variability. Results were also required more rapidly for the soil-structure interaction computer analysis so as to gain a feel for the behaviour characteristics of the structure. In order to obtain the results quickly, the oedometer specimens were subjected to only two loading stages. The initial stage was to reinstate the approximate vertical effective stress at the original specimen depth. The
second stage raised the vertical stress level by the expected increase in total vertical stress due to the foundation loading.

It should be noted that experience, sample disturbance effects and considerations of stress paths and pore pressure coefficients suggest that $1/n_{fe}$ values will lead to over estimates of combined immediate and elastic settlement, and thus these values were used in the initial settlement analysis.

**Findings and Recommendations of the Second Site Investigation**

From the findings of the laboratory tests and the soil-structure interaction analysis, discussed later, the following points were made in the report to the consultant by the geotechnical consultant:

1) The thickness of the limestone was found to vary between 2.12 and 3.19m. Rock thicknesses as great as those described in the first site investigation report were not found.

2) With a minimum unconfined compressive strength of $2\text{MN/m}^2$ and an average of $8\text{MN/m}^2$ for the rock the pile design was considered inadequate. Based upon conventional wisdom, unconfined compressive strengths of the order of 20 to $25\text{MN/m}^2$ would be required to sustain the $5\text{MN/m}^2$ design pile toe stresses. Local failure could be expected, particularly if the piles did not bed into the top of the rock during driving.

3) The thickness of the rock was only just sufficient to spread the pile loads to the Lias Clay below.

4) Uniaxial unconfined compressive strengths tests indicated that the proposed pile toe stresses were excessive.
Assuming that the piles seated into the rock without significant penetration, a maximum stress of about 1MN/m² was indicated. This was in general agreement with the first site investigation. However if the piles achieved significant penetration into the rock then the Lias Clay might well be overstressed. It was thus essential that the distance of penetration of the piles into the upper part of the rockhead should be determined.

5) Since the ability of the limestone to support the end bearing piles could not be justified by laboratory experimentation it was recommended that a significant number of piles be tested with care in order to establish the available bearing capacity. At least ten, and preferably fifteen slow maintained load tests, using proving ring measurement and dial gauge displacement, were recommended to be carried out. The load was to be 1.5 and in some cases 2.5 times working load.

6) For redesign purposes a value equal to the estimate for shaft adhesion should be deducted from the pile test load. Equally for a true pile load test to 1.5 or 2.5 times the working load to be undertaken the value estimated for shaft adhesion should be added to the load. The contribution of shaft adhesion to bearing capacity during testing should be in the order of 500 to 600kN.

7) Negative skin friction should be allowed for in the calculation of applied pile loading.

8) The undrained shear strength of the Lias Clay was considered adequate to give a reasonable factor of safety against overall bearing capacity failure of the piles.
provided the Limestone was thick enough to spread the high contact stresses applied by the piles.

9) The total settlement of the Wheat Silo was unlikely to exceed 190mm due to movement in the Lias Clay provided the pile toe stresses reduced to acceptable values. Compression within the limestone on this basis was considered negligible.

The value of 190mm was based on the Young's Modulus values obtained from the one dimensional, oedometer, consolidation tests. Later correspondence (communication between the geotechnical consultant and the consultant), when the values were available from the reload testing, suggested that this value in reality would be half to a third of 190mm say 60 to 85mm as explained in the Laboratory testing section.

Discussion on the results of the second site investigation

The laboratory testing was carried out knowing that there was little time to do justice to the quality of the sampling. One objective of the investigation was achieved as a result of the drilling method. The measurement of the thickness of the limestone was shown to be more uniform when sampling with rotary coring as opposed to light percussion techniques. Laboratory testing commenced with the arrival of the first core. There were only some two weeks to undertake the laboratory work and the analysis. As a result great reliance had to be placed on the visual examination of the core. Detailed logging of the core as it arrived provided a record by which the core used in the laboratory testing could be judged as being representative of that from other boreholes. It is notable that, despite good quality coring and laboratory testing on specimens taken from one borehole, the test results display a wide range of values for Cu (Fig.2.1/11).
The testing for the unconfined compressive strength on rock (Table 2.1/8) was not to the ISRM specification (1978). The tested core diameter was 38mm and not the minimum NX core size (54mm approx). The standard does state that the diameter of the specimen should be related to the largest grain size by a ratio of at least 10:1 and that the height/diameter ratio should be 2.5 to 3. A study of the rock fracture log would suggest that this ratio could be achieved with a specimen diameter of about 38mm. The undrained triaxial test on Lias Clay (Table 2.1/9) did not have a core size specified in the Report. The core was taken from a "P" size corebarrel which is 91.95mm diameter (Clayton, Simons and Matthews 1982). As the core is unrestrained in a liner the specimens were likely to have been prepared by trimming the ends of the samples and would have been some 90mm diameter. This is further supported as there is only one specimen tested at each depth.

The above comments on the specimen size also apply to the drained reload specimens (Table 2.1/11). The objective of the drained reload test is interesting in the light of current thinking. The test was carried out to provide a modulus value not affected by bedding. To achieve this the specimen was loaded, in the triaxial cell, sufficiently to remove bedding but not enough to induce consolidation. The specimen was then subjected to a reload loop, but not completely removing the load. It was thought that removal of all the load may result in the return of some of the bedding. The slope of the reload portion of the plot was then measured to provide a more representative modulus. The conflict between this method and the current testing occurs as the measurement of the strain incorporated the bedding of the sample and the compliance of the apparatus. Current test methods remove bedding errors and the compliance of the apparatus by using strain measurements across the middle of the specimen (Jardine et al 1984 and Clayton & Khatrush 1986) instead of external measurement by LVDT or dial gauge.
The Oedometer tests (Table 2.1/12) were subjected to two loading stages. The first reinstated the specimen to the stress state in the ground. The second stage added the stress increase expected due to the structure. The constrained modulus is derived from $1/m$ values and compares well with the value of undrained modulus $E_u$ reported in Table 2.1/9. The strain values at which both moduli were measured are not representative of those in the ground due the bedding effects on the laboratory specimens. As a result of the difference in strain level for testing a direct comparison of the $E_u$ values obtained is not possible. However by using the undrained strengths reported in Table 2.1/9 a value has been estimated from the dimensionless plot of $E_u/C_u$ vs strain reported by Jardine et al (1984). The values are tabulated next to those for $E_u$ obtained from the undrained triaxial test and are shown in Table 2.1/10 along with values suggested by Butler (1974). All of the values for Modulus are also plotted in Figure 2.1/13 against depth. The curves reported by Jardine et al (1984) and the values suggested by Butler (1974) were for London Clay. This is a similar overconsolidated material but the London Clay is not as old geologically. Thus the Lias Clay would be expected to behave in a similar but stiffer manner and the values in Table 2.1/10 should only be regarded as an indication.

The values indicated in Table 2.1/10 after Butler are derived from two empirical relationships between $E_u$ and $C_u$. The first $E_u=220C_u$ is a relationship derived from experience over a number of years and is commonly used for design purposes for London Clay. The second $E_u=400C_u$ is a relationship derived by Butler which best fit the back analyzed case records he reported in the paper (1974). The empirical relationship was derived from drilling techniques not as advanced as the rotary coring used at Corby. Use of the empirical relationship with the $C_u$ values obtained from these very good quality specimens would yield a higher value of $E_u$ than would be expected from the conventional sampling techniques the relationship was derived for.
The values indicated in Table 2.1/10 after Jardine have been derived from a dimensionless plot of $E_u/C_u$ against strain for London Clay presented in their paper. The values for $E_u$ derived by this means would be expected to yield conservative values of undrained modulus for the Lias Clay on two counts.

a) The Lias clay is older than the London Clay and would thus be expected to be closer in nature to a weak rock than the London Clay.

b) The sampling of the London Clay reported by Jardine et al (1984) was by thin wall piston sampler. The same method of sampling is thought not to have been possible for the Lias Clay at this location where the samples were obtained by rotary coring techniques.

It can be seen from Table 2.1/10 that the ratio $E_u/0.01/E_{u,\text{triax}}$ is of the order of 6 and that $E_u/0.1/E_{u,\text{triax}}$ is of the order of 2. Jardine et al (1984) also provide the same dimensionless plot for Upper Chalk from a site in North Kent. Where as $E_u/0.1/C_u$ is 1100 for the London Clay $E_u/C_u$ for this medium is reported from 2500 to 4000 indicating a very much stiffer initial response. As both chalk specimens failed at 0.07% strain a comparison of $E_u/0.1/E_u/0.01$ is not possible. However an indication of how the material maintains its initial stiffness can be gained from the reported values of $E_u/0.06/E_u/0.006$ 0.723 and 0.854. These compare to the values of $E_u/0.1/E_u/0.01$ for London Clay of 0.371 and 0.387. This suggests that likely values of modulus for the Lias Clay show a much stiffer initial response to those postulated for $E_u/0.01$ in Table 2.1/10 and that the material is likely to maintain a larger proportion of its stiffness as strain increases. The lowest dimensionless ratio would suggest that $E_u/0.01/E_{u,\text{triax}}=15$ and that $E_u/0.1/E_{u,\text{triax}}$ might be of the order of 12 compared to the values of 6 and 2 respectively for the two equivalent London Clay figures. The
actual values for the Lias Clay will lie somewhere between the two sets of figures given above.

Looking at Table 2.1/10 and Figure 2.1/13 it can be generally seen that:

a) the values for \( E_u \) compare well with those for the expression \( E_u = 400C_u \).

b) the design value of \( E_u = 220C_u \) are about 1.5 times \( E_{u_{\text{triax}}} \) for the Lias Clay

The literature reports that local strain measurements reports values for modulus 2.5 to 4 times those reported by external strain measurement. This would result in a possible range of 45 to 200 MN/m² calculated from the range of 17 to 48 MN/m² reported in Tables 2.1/9, 2.1/10 and 2.1/11 which is clearly not a basis on which to base a settlement calculation.

Investigation on the Adjacent Site

A ground investigation was carried out on an adjacent site in October 1982. The site work consisted of light percussion boring and rotary drilling to produce four holes. Rotary drilling was carried out using PF size double tube swivel type core barrel, bentonite mud flush and Mylar linings. Cores were sealed with tape and wax to prevent moisture loss.

No water was encountered during drilling.
Laboratory Testing.

The following tests were carried out on the samples:

1) Undrained Triaxial Compression Tests.
   10 No. on the Made Ground
   6 No. on the Lias Clay

2) Uniaxial Unconfined Compressive Strength Tests
   18 No. on the Rock Core

Findings of the Site Investigation.

The report indicated the following lithology.

Firm SILTY or SANDY CLAY with scattered gravel and occasional boulders (made ground). The average undrained strength was 51kN/m² with a range of 36 to 90kN/m². The depth of the stratum varied from 6.0 to 7.8m

LIMESTONE varying in thickness between 2.7 and 2.9m. The rock was classified as weak to moderately strong. The Unconfined Compressive Strength varied from 2.5 to 13.4MN/m² with an average of 7.4MN/m². (The results are shown on Fig.2.1/10 with the results from the second site investigation).

UPPER LIAS CLAY very stiff to hard dark grey clay proved to 5.2m below the base of the limestone. Undrained Shear Strengths varied from 98 to 414kN/m² with an average strength of 264kN/m². (The results are shown in Fig.2.1/11 along with those from the second site investigation).
Conclusions from the Three Site Investigations.

The geotechnical consultant made the following concluding remarks on the results of the three site investigations.

When comparing the above with the findings of the initial and second investigations the following points are noted:

1) The results for the quick undrained triaxial tests on the made ground for both the initial investigation and the investigation on the adjacent site show the uncompacted fill to be highly variable (Table 2.1/13).

Two results from the initial investigation of 268 and 503kN/m² were excluded from the above.

2) Although the average strength for the initial investigation is higher, a similar range of results is evident. In both sets of test results there did not appear to be an apparent gain in strength with depth. Higher strengths were generally attributed to sampling intact lumps in the uncompacted fill. All three investigations indicated that negative skin friction was to be expected on a piled foundation due to self settlement. If driven piles were used negative skin friction would also arise due to dissipation of pore pressures induced as a result of piling.

3) The results of the Uniaxial Unconfined Compressive Strength Tests on rock core for the second investigation and the investigation on the adjacent site were in agreement (Table 2.1/14).
4) The thickness of the rock stratum from the four holes produced during the investigation on the adjacent site varied from 2.7 to 2.9m. This range lies within the range of thickness produced during the second investigation, 2.12 to 3.1m. Nowhere was the rock found to be approaching the thickness of 3.9m, reported in the initial investigation, which was determined by light percussion boring.

5) The results of the Unconsolidated Undrained Triaxial Tests on the Lias Clay for the second investigation and the investigation on the adjacent site are in good agreement (Table 2.1/15).

It was now apparent that the rock stratum could not be proved adequate for the pile design and that as the pile contract had already commenced it was now necessary to prove the pile design by pile load test.
Table 2.1/1 Unconsolidated Undrained Triaxial Test on Fill Material

specimen size 38mm diameter

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<th>m/c (%)</th>
<th>Bulk Density (kN/m³)</th>
<th>Dry Density (kN/m³)</th>
<th>Lateral Pressure (kN/m²)</th>
<th>Deviator Stress (kN/m²)</th>
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### Table 2.1/2

Unconsolidated Undrained Triaxial Test on Fill Material

Specimen size 38mm diameter

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<th>Apparent Angle of Cohesion Friction (kN/m²)</th>
<th>(degrees)</th>
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Table 2.1/3  Unconfined Compressive Test on Rock Core

Specimen size not specified

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Table 2.1/4  One Dimensional Consolidation Oedometer Test on Fill Material

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Table 2.1/5 Unconsolidated Undrained Triaxial Test on Lias Clay

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<th>Lateral Pressure (kN/m²)</th>
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Table 2.1/6 Chemical Analysis of the Fill

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### Table 2.1/7 Atterberg Limits for the Fill

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### Table 2.1/8 Unconfined Compressive Strength Test on Rock

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Table 2.1/9  Undrained Triaxial Test on Lias Clay

specimen size 94mm diameter

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<tr>
<th>B.H. Depth No.</th>
<th>m/c (%)</th>
<th>Bulk Density (kN/m²)</th>
<th>Dry Density (kN/m²)</th>
<th>Cell Pressure (kN/m²)</th>
<th>C_u (kN/m²)</th>
<th>E_u (kN/m²)</th>
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<td>25.8 Local soft spot assumed caused low failure</td>
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Table 2.1/10 Modulus Values for the Lias Clay (MN/m² or MPa)

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<th>B.H. Depth No.</th>
<th>C_u (kN/m²)</th>
<th>E_u (MN/m²)</th>
<th>Butler 220C_u</th>
<th>400C_u</th>
<th>Jardine E_u0.01</th>
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<td>34</td>
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<td>157-186</td>
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Butler (1974)

Table 2.1/11 Drained Reload Compression Test on Lias Clay

specimen size 94mm diameter

<table>
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<tr>
<th>B.H. Depth No.</th>
<th>m/c</th>
<th>Bulk Density (kN/m²)</th>
<th>Dry Density (kN/m²)</th>
<th>Cell Pressure (kN/m²)</th>
<th>Back Pressure (kN/m²)</th>
<th>Loading (MN/m²)</th>
<th>First Loading (MN/m²)</th>
<th>Second Loading (MN/m²)</th>
<th>Young's Modulus</th>
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Table 2.1/12 One Dimensional Consolidation Test on Lias Clay

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<tr>
<th>Borehole</th>
<th>Depth (m)</th>
<th>Load Range (kN/m²)</th>
<th>Coefficient of Compressibility (m²/MN)</th>
<th>Constrained Modulus (MN/m²)</th>
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### Table 2.1/13 Variability of the Made Ground

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<th>Investigation</th>
<th>Apparent Range ($\text{kN/m}^2$)</th>
<th>Cohesion Average ($\text{kN/m}^2$)</th>
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<tr>
<td>Initial</td>
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<td>0.4 to 8.4</td>
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### Table 2.1/14 Range of Unconfined Compressive Strength Test on Rock Core

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<td>Adjacent Site</td>
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### Table 2.1/15 Range of the Unconsolidated Undrained Triaxial Test on Lias Clay

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<th>Shear Strength Range ($\text{kN/m}^2$)</th>
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<td>Adjacent Site</td>
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Figure 2.1/1 Map Showing the Location of Corby
Figure 2.1/2  The Workings of the Open Cast Mine
Figure 2.1/3 Plan of the Flour Mill Site
Borehole No. 1216/1

- Firm locally stiff dark grey light grey or yellow to orange-brown gravelly silty clay, locally slightly to very sandy, gravel fraction of fine and medium grained chalk and flint, locally organic. FILL
- Boulder of dark grey limestone blocking base of borehole.
- Dark green fine grained oolitic weathered LIMESTONE, slightly to very friable.
- Grey green fine grained partly oolitic weathered LIMESTONE with calcareous silty clay.
- Grey green becoming dark grey thinly laminated micaceous clayey SILTSTONE, initially calcareous and slightly sandy, locally fossiliferous.

Water Strike at 9.8m

Borehole No. 1216/2

- Firm locally stiff dark grey light grey or yellow to orange-brown gravelly silty clay. FILL
- Boulder blocking base of borehole. No recovery.
- Firm locally soft or stiff dark grey light grey or yellow to orange-brown gravelly silty clay, with pockets of fine, medium and coarse gravelly sand and soft silt, locally grumose. Gravel fraction of fine and medium grained chalk and flint, and locally flint and sandstone. FILL
- Dark green becoming grey green floculent oolitic weathered LIMESTONE with calcareous sandy clay.
- N=70 for 225mm
- N=57 for 150mm
- N=44 for 75mm

Water Strike at 9.8m

Borehole No. 1216/3

- Firm locally stiff dark grey light grey and yellow to orange-brown gravelly silty clay. Generally calcareous and locally laminated with minor pockets of silty sand, gravel fraction of fine and medium grained chalk and flint. FILL
- Dark green fine to medium grained oolitic weathered LIMESTONE with calcareous silty clay.
- Grey green fine grained partially oolitic weathered LIMESTONE with calcareous silty clay.
- N=58 for 150mm
- N=36 for 75mm

Water Strike at 10.1m
Figure 2.1/5 Boreholes 4, 5 and 6

Borehole No. 1216/4

Firm locally soft or
stiff dark grey light
gray and yellow to
orange-brown gravelly
silty clay. Locally
organic or calcareous
and occasionally very
sandy, and very silty.
Gravel fraction of fine
and medium-grained chalk
and flint with minor
sandstone, siltstone and
coarse-grained
limestone. FILL

Dark green fine to medium
grained dolostone weathered
LIMESTONE with minor
ironstone and shell
fragments. Initially
with calcareous sandy
silty clay.

Grey green fine grain
partially dolomitic
weathered LIMESTONE with
calcareous sandy silty
clay.

Water Strike at 10.2m

---

Borehole No. 1216/5

Firm locally soft and
stiff dark grey light
gray and orange-brown
gravelly silty clay. Locally
organic or calcareous
and very sandy, very
silty clay. Gravel
fraction of fine
and medium-grained chalk
and flint with minor
sandstone, siltstone and
limestone. FILL

Dark green fine grain
dolostone weathered
LIMESTONE becoming grey
green fine grain
partially dolomitic
weathered LIMESTONE with
minor calcareous sandy
silty clay.

Water Strike at 10.2m

---

Borehole No. 1216/6

Firm locally soft to
very stiff clay. Gray
gray-brown and orange-
brown gravelly slightly
sandy silty calcareous
clay and clayey or sandy
clay. Gravel fraction
principally of fine and
medium-grained chalk.
Locally organic. FILL

Dark green fine grain
dolostone becoming partially
dolomitic weathered
LIMESTONE slightly
fractured.

Grey green fine grain
partially dolomitic
weathered LIMESTONE with
calcareous sandy silty
clay.

Water Strike at 9.9m
Figure 2.1/6  Borehole 7
Weak to moderately weak well-jointed light grey green LIMESTONE
Moderately strong grey green LIMESTONE
Weak to moderately strong blue-grey LIMESTONE
Very weak to weak clayey SILTSTONE
Stiff probably locally soft grey CLAY

Very weak to weak grey MUDSTONE. Probably contains interstratified SILTSTONE and CLAY strata.

Light grey green very thinly to thinly bedded moderately weak sparsely oolitic fine grained LIMESTONE becoming red/yellow to thinly bedded moderately strong coarse grained LIMESTONE. Ferruginous staining at top. Fine grained and very weak at base becoming soft calcareous clay.

Stiff locally soft fine or very stiff clay laminated to very thinly bedded oolitic silty or very silty CLAY. Locally hard very clayey silt. Occasional fossils, pyritisation, brecciated fragments and 0.5 to 3.0mm slit partings.

Dark grey thinly laminated very weak slightly clayey SILTSTONE.

Soft to fine grey silty CLAY. Calcareous clasts and concretions and mudstone clasts below 15.2m.

Very weak to weak dark grey thinly laminated clayey SILTSTONE with occasional partings of firm to stiff silty clay. 15.25 to 16.0m soft to firm very clayey silt with very silty clay. 16.4 to 16.6m oolitic breccia. Occasional strong light grey limestone bands (0.50 to 0.8m).

Stiff locally soft firm or very stiff dark grey silt to very silty CLAY, with occasional clayey silt horizons.

Dark to very weak thinly laminated grey silty MUDSTONE.

Figure 2.1/7 Boreholes 2(II) and 7(II)
Figure 2.1/8 $C_u$ against Depth for the Fill
Figure 2.1/9 $m_Y$ against Depth for the Fill
Figure 2.1/10 Unconfined Compressive Strength of the Limestone
Figure 2.1/11 Undrained Shear Strength of the Lias Clay
Figure 2.1/12 Compressibility of the Lias Clay Boreholes A and B
Modulus (MN/m$^2$)

- $E_U$ from the undrained triaxial test
- $220\sigma_u$ (Butler 1974)
- $400\sigma_u$ (Butler 1974)
- $E_{\text{min.}}$ lower bound (Jardine et al 1986)
- $E_{\text{max.}}$ upper bound (Jardine et al 1986)
- $1/M_v$

Figure 2.1/13  Modulus Values Against Depth for the Lias Clay
Plate 2.1/1 Aerial Photograph of a Quarry and the Hill and Dale Backfilling at Corby
Plate 2.1/2  The Flour Mill Complex, Corby
2.2. Building Details of the Mill Complex.

2.2.1. General Site Details

The Flour Mill Complex consists of the wheat and flour silos, tempering bins, warehouse, bulk tanker outload and office block (Figure 2.1/3 and Plate 2.2/1). Provision has been made, and is indicated in the figure, to extend the mill complex at some future date. Each unit of the complex is founded on an independent foundation slab. This results in each slab producing individual loading characteristics transmitted through the foundation system to the load bearing strata with resulting individual settlement characteristics. Account had to be taken of the individual slab differential settlement and inter-slab differential settlement. The former is an individual slab design criterion, the latter comes about due to interconnecting machinery from unit to unit.

The regions most critical to the design were the Wheat and Flour Silos.

The larger and more heavily loaded unit is the Wheat Silo which imposes a maximum average stress of 360kN/m² at foundation slab level, is of considerable plan area and poses the most critical foundation design problems.

The foundation slab is rectangular, 15.9m wide by 33.2m long and supports 15 No. rectangular silo bins in a plan area 15.9m wide by 26.3m long, Fig.2.2/1. At one end, occupying the remainder of the slab area, very little live load was anticipated since the elevators and cleaning machinery are housed here.

The Wheat Silo itself is 45m high with the silo bins commencing at 5.4m and terminating at 34.4m above the foundation slab, Fig.2.2/2. The 15 No. bins are square in cross-section and were constructed in
reinforced concrete using a slipforming technique. Structurally the bins are supported by columns. Externally the columns are incorporated in the reinforced concrete silo walls. Internally there are 10 No. cruciform reinforced concrete columns.

The foundation slab was piled. Its as-built form, after slight modification taking the settlement analysis and pile load tests into account, is shown in Fig.2.2/1. As can be seen there were 245 piles under the wheat silo foundation slab. These were 533mm diameter West segmental shell piles. 208 of these were to support the heavily loaded part of the structure supporting the grain bins. The remaining 37 were to support the elevator and pre-clean area.

The Flour Silo was the second area of concern imposing an average stress at foundation level of 321kN/m². The foundation slab is rectangular 14.75m wide by 21.5m long, Fig.2.2/3. The structure supports 32 bins of various sizes commencing on the third and fourth floors of a structure some 40m high, Fig.2.2/4. The building is of structural steel and reinforced concrete construction.

The foundation slab of the flour silo was also piled, Fig.2.2/3, using 150 No. 533mm diameter West segmental shell piles.

2.2.1. Foundation Construction.

Time and finance were restricted because of the commercial pressures and hence the design and construction of the piled foundations were interrelated in an unusual way. The initial site investigation had shown rock at a relatively shallow depth, and although the report highlighted a number of areas of uncertainty, the ground conditions lent themselves to a foundation design using driven piles. An initial design had been completed and construction works were under way at the time the second site investigation was commissioned. The primary purpose of the second investigation was to obtain the necessary
parameters to undertake the pile design. The initial pile design layout was modified and additional piles were added when the results of the settlement analysis and pile load tests were taken into account. The following sequence of events occurred:

On receipt of the initial site investigation it was necessary to proceed rapidly with the foundation construction. Tenders were invited from three specialist piling contractors and the piling subcontractor was awarded the contract and had started piling at the time of the second site investigation.

The accepted subcontract consisted of about 700 precast shell piles as it offered a number of advantages. These were as follows:

1) The piles have a conical toe which were expected to bed better into the rock.

2) The integrity of the piles could be guaranteed with less site control.

3) The high strength shells provide additional pile strength and resistance to sulphate attack.

4) The concrete core can be poured to the required cut-off levels rather than pile platform level.

5) Although the toe stresses were high, they were less than offered by other piling contractors.

6) The offer included pre-boring, monitoring and re-driving should pile heave become a problem due to adjacent pile installation.
Added to this the piling contractor was confident of his system, based on its performance in similar ground conditions elsewhere. Despite the evidence of the first site investigation, which indicated the very weak nature of the rock, the piling contractor judged on his experience that;

i) a 2m thickness of rock was more than adequate,

ii) the unconfined compressive strength method of predicting the safe end bearing resistance of piles is grossly conservative,

iii) clays beneath limestones are usually very stiff,

iv) negative skin friction is usually grossly overestimated by geotechnical engineers and might be 10 to 15 tonne per pile.

Up to the first pile test, the piling contractor maintained, based on his experience, that preboring to any depth was unnecessary. At this time the site supervising staff also showed considerable reluctance to insist on preboring. In order to establish that preboring was not necessary the piling contractor drove three 444mm diameter piles in a line at 4.2m centres, and by conventional levelling established that no measurable heave occurred. A considerable number of piles were then driven without preboring before the design engineer could reverse the decision. He made this decision 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 preboring did not take place.
Subsequently preboring took place but only to a depth of 6m because the piling contractor’s offer was not specific and he did not wish to carry out work for which he had not tendered.

The insistence of the design engineer to prebore appeared justified on the completion of the first pile test. Observations on selected piles during driving indicated that piles were penetrating up to 0.3m into the top of the rock before the set was reached. Typically the penetration was between 0.1 and 0.15m, demonstrating that sufficient rock remained beneath the toes of the piles to spread the load onto the Lias Clay. Since the end bearing pressures of the piles could not be justified on the basis of the laboratory test results and calculations of the second site investigation, a programme of slow maintained load testing had been recommended and was carried out. 9 No. piles were tested to 1.5 times their nominal capacity. Fig.2.2/5 shows the result of the pile load test on pile 261, the first pile to be tested. At a load of 65tonne (65x9.81kN) the pile failed due to excessive settlement. At 65tonne the settlement was under 5mm and at 100tonne the settlement was 38mm. At a settlement of 45mm bearing capacity improved. On analysis it can be seen that at 65tonne the limit of skin friction on the pile had been reached and the pile was settling without taking additional load as there is no end bearing resistance to additional load. At a settlement of 45mm the bearing capacity improved indicating that the pile had heaved away from the rock whilst the driving of neighbouring piles had taken place. If the test is re-plotted from the reload loop it can be seen that a substantially normal pile load test result is obtained.

The pile was in a group of 70 piles, with a spacing/diameter ratio in one direction of 2.72 and 3.75 in the other, which had not been prebored. The obvious explanation was that a shaft friction of 65 tonne had been mobilised, and that the pile had then reseated itself onto the rock. Some 320 piles were therefore redriven. A summary of the redriving is given in Table 2.2/2. During redriving the pile
heads were relevelled and it became clear that preboring to 6m had not eliminated pile heave. 245 piles were under the wheat silo foundation slab, 37 of which support the elevator and pre-clean area and were not prebored. The remaining 208 were under the heavily loaded part of the structure supporting the grain bins, the wheat silo itself, of which 192 were prebored to 6m. Although all 208 were to be prebored at the insistence of the design engineer it would appear that 16 were not because of site construction problems.

A comparison of the 53 piles that were not prebored and the 192 that were showed that, when the piles were redriven to a set, preboring 6m did not make any appreciable difference in the amount of redriving required (Fig.2.2/6). The reason for the designers' insistence to prebore in the wheat silo was that

1) the pile spacing/diameter ratios were considerably smaller than in the elevator and pre-clean area,

2) the piles carried higher loads than those in the elevator and pre-clean area.

It could logically be argued that had the closely spaced piles not been pre-bored they would have required more and not equivalent re-tapping. However, an examination of the line of piles closest to the elevator and pre-clean area suggests a different outcome. This line of piles were not pre-bored, had the smallest spacing/diameter ratio and yet required equivalent re-tapping to those pre-bored piles under the wheat silo and the piles under the elevator and pre-clean area. The pile spacing/diameter ratios in the wheat silo are 1.92 and 1.99 around the edge and 2.81 internally compared with 2.81 and 3.47 in the elevator and pre-clean area. Subsequent pile load tests gave satisfactory results, with maximum settlements under 165 tonne ranging from 5.5 to 8.0mm, and residual settlements after unloading between 0.4 and 1.7mm (Table 2.2/1 and Fig. 2.2/7). Uncertainties of how the
load is shed 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 led to a further series of tests. Three additional 533mm diameter piles were installed outside the silo areas and were tested to 2.5 times their nominal capacity (Figs.2.2/8 to 2.2/10). Maximum settlements under 250 tonne ranged from 8.8 to 10.8mm, with residual settlements after unloading of between 1.5 and 4.6mm (Table 2.2/1).

Once the design of the piles had been established practically by the pile load tests, as recommended by the second site investigation, the foundation slabs and superstructure construction could commence.
Table 2.2/1 Summary of the Pile Test Data.

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Depth to Toe (m)</th>
<th>Estimated Penetration into Limestone (m)</th>
<th>Test Load (kN)</th>
<th>Maximum Settlement (mm)</th>
<th>Residual Settlement (mm)</th>
<th>Penetration for last 10 Blows (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>93</td>
<td>9.2</td>
<td>0.0</td>
<td>1175</td>
<td>5.04</td>
<td>0.70</td>
<td>15</td>
</tr>
<tr>
<td>196</td>
<td>10.3</td>
<td>0.2</td>
<td>1650</td>
<td>6.18</td>
<td>1.20</td>
<td>15</td>
</tr>
<tr>
<td>261</td>
<td>10.1</td>
<td>0.3</td>
<td>1700</td>
<td>21.38</td>
<td>16.04</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1500</td>
<td></td>
<td></td>
<td>51.89</td>
</tr>
<tr>
<td>387</td>
<td>9.6</td>
<td>0.1</td>
<td>1650</td>
<td>5.82</td>
<td>0.84</td>
<td>14</td>
</tr>
<tr>
<td>391</td>
<td>9.5</td>
<td>0.1</td>
<td>1650</td>
<td>6.90</td>
<td>1.72</td>
<td>20</td>
</tr>
<tr>
<td>570</td>
<td>8.6</td>
<td>0.0</td>
<td>1650</td>
<td>7.99</td>
<td>0.17</td>
<td>16</td>
</tr>
<tr>
<td>594</td>
<td>8.7</td>
<td>0.1</td>
<td>1650</td>
<td>5.51</td>
<td>0.43</td>
<td>15</td>
</tr>
<tr>
<td>678</td>
<td>8.7</td>
<td>0.2</td>
<td>1650</td>
<td>5.94</td>
<td>0.39</td>
<td>16</td>
</tr>
<tr>
<td>705</td>
<td>8.7</td>
<td>0.0</td>
<td>1650</td>
<td>5.78</td>
<td>1.14</td>
<td>18</td>
</tr>
<tr>
<td>815</td>
<td>9.6</td>
<td>0.2</td>
<td>2500</td>
<td>10.12</td>
<td>1.68</td>
<td>10</td>
</tr>
<tr>
<td>816</td>
<td>8.7</td>
<td>0.3</td>
<td>2500</td>
<td>10.75</td>
<td>4.55</td>
<td>10</td>
</tr>
<tr>
<td>817</td>
<td>10.2</td>
<td>0.3</td>
<td>2500</td>
<td>8.79</td>
<td>1.52</td>
<td>7</td>
</tr>
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</table>
Table 2.2 Summary of the Wheat Silo Pile Re-Driving Settlements

<table>
<thead>
<tr>
<th>Settlement after Re-Tapping (mm)</th>
<th>Percentage of the Pile Population</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10 to 0</td>
<td>4.1</td>
</tr>
<tr>
<td>0 to 10</td>
<td>12.2</td>
</tr>
<tr>
<td>10 to 20</td>
<td>15.4</td>
</tr>
<tr>
<td>20 to 30</td>
<td>13.8</td>
</tr>
<tr>
<td>30 to 40</td>
<td>8.1</td>
</tr>
<tr>
<td>40 to 50</td>
<td>13.8</td>
</tr>
<tr>
<td>50 to 60</td>
<td>7.3</td>
</tr>
<tr>
<td>60 to 70</td>
<td>6.5</td>
</tr>
<tr>
<td>70 to 80</td>
<td>8.9</td>
</tr>
<tr>
<td>80 to 90</td>
<td>1.6</td>
</tr>
<tr>
<td>90 to 100</td>
<td>2.4</td>
</tr>
<tr>
<td>100 to 110</td>
<td>1.6</td>
</tr>
<tr>
<td>110 to 120</td>
<td>2.7</td>
</tr>
<tr>
<td>140 to 150</td>
<td>1.6</td>
</tr>
<tr>
<td>180 to 190</td>
<td>0.8</td>
</tr>
<tr>
<td>200 to 210</td>
<td>0.8</td>
</tr>
<tr>
<td>510 to 520</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Population of the piles Pre-bored to 3m: 123
Population of the piles not pre-bored to 3m: 53
Figure 2.2/1 Plan of the Wheat Silo
Figure 2.2/2 Elevation of the Wheat Silo
Figure 2.2/3 Plan of the Flour Mill
Figure 2.2/4  Elevation of the Flour Mill
Figure 2.2/5 Plot of the First Pile Test
Figure 2.2/6 Histogram of Pile Heave for Pre-Bored and Not Pre-Bored Piles
Figure 2.2/7 Plot of a Pile Test to 1.5 times Working Load. Pile 594
Figure 2.2/8 Plot of a Pile Test to 2.5 times Working Load. Pile 815
Figure 2.2/9 Plot of a Pile Test to 2.5 times Working Load. Pile 816
Figure 2.2/10 Plot of a Pile Test to 2.5 times Working Load. Pile 817
2.3. Corby Design Assumptions

2.3.1. General Design Considerations

The initial site investigation established the ground conditions, in general terms, with the view of founding the structures on 100 tonne piles. Consideration was given to the likely settlement of the recently placed fill material. If the strata was as expected it appeared that a piled foundation had many attractions. The finding of the initial site investigation however highlighted several problems associated with founding the structure on piles but more importantly highlighted the shortfalls in the parameters obtained for the strata to predict the resultant settlements.

The foundations were to support machinery on relatively rigid slabs which were not structurally rigidly connected. It was necessary to keep inter-slab differential settlement to within reasonable limits of tolerance due to the inter-connecting machinery in the plant.

The shortfalls in the knowledge of the soil parameters were confirmed when a specialist consultant advised the designers that a prediction of the likely settlements was not possible on the basis of the data available at the time. He recommended that a second site investigation be carried out to ascertain with a greater degree of certainty the parameters for the Limestone stratum and the underlying Lias Clay. With the soil parameters and the structural loading parameters it would then be possible to analyze the structure as a whole and make some predictions as to the likely slab differential settlement and inter-slab differential settlements.

Structural design considerations dictated that the Wheat Silo slab be kept to a maximum thickness of 1600mm and the Flour Silo slab 1500mm thickness. With these limitations the structural designer decided that across slab differential settlement be kept below 5mm to limit
the required reinforcement steel to manageable and physically sensible proportions.

At this stage the piling sub-contact had already been let and consisted of a design based on 533mm diameter prebored West shell segmented piles at 1.5m centres with 4.5MN/m² toe stress and at the time it was not at all certain that the limestone could maintain this level of stress without punching failure or indeed that the Lias Clay could maintain the induced stress levels without resulting in large deflections.

2.3.2. Assessment of the Pile Design

The nominal pile capacity quoted by the piling contractor was 110 to 120 tonne. This figure was obtained from consideration of the concrete used in the pile. Permissible concrete stresses in driven cast-insitu piles are normally restricted to 25% of the 28 day strength (CP2004 and CP114). For the standard 1:2:4 mix given in CP114:1957 for a design strength of 21N/mm² an allowable toe stress of 5.2MN/m² is obtained. On a 533mm diameter pile this toe stress gives a capacity of 118 tonne.

When safe pile toe stresses are obtained it is also required that a third to a fifth of the unconfined compressive strength of the rock is not exceeded as an allowance for fracturing the rock (Bowles 1978). Thus the least toe stress calculated from the above two conditions is generally taken as the safe toe stress.

Information relating to end bearing capacity of socket piles has been presented in the Proceedings of the International Conference on Structural Foundations on Rock (1981) and a comprehensive survey of the field tests was presented by Williams and Pells (1981). The behaviour of piles in rock is highly dependent upon the way the rock is modified by pile installation, and yet there is no information
relating to full scale field tests for driven piles. 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) and Rehnman and Broms (1970 and 1971) which indicate that the maximum bearing capacity is 5 to 15 times the unconfined compressive strength of the rock. Fractures in the rock are known to reduce the bearing capacity of the rock but the relationship between loaded area, fracture spacing and the degree of opening of the fracture remains unknown. For piles driven into rock, the situation is further complicated in that the depth of penetration and the variability of the rock quality are largely unknown. Reliable determination of the load capacity can only come from experience and thus consideration should be given to the views of piling contractors who have relevant experience in the field conditions locally. However by the time the second site investigation had been completed it was quite clear that the pile design might be inadequate. The information from the first site investigation relating to the Limestone was sketchy and by this time the sub-contract had been let. From the second site investigation the uniaxial unconfined compressive strength at the top of the rock was of the order of 1 to 5MN/m², the rock was fractured, and the proposed working toe stress was 5.2MN/m² which would have required an unconfined compressive strength of 16 to 26MN/m² in the Limestone for the toe stress to be safe (Bowles 1978).

The soil-structure interaction analysis (discussed in section 2.4) showed that the edge soil/structure contact stresses were over three times those obtained for the centre soil-structure contact stresses. This highlighted the requirement for the edge and corner piles to carry more load for a symmetrical pile arrangement (Hooper 1979, Green and Hight 1976 and Cooke et al 1981) and that the design arrangement be scrutinised carefully to ensure there was enough load bearing capacity at these locations. With the parameters used in the soil-structure analysis the geotechnical consultant found that the edge contact stresses were of the order of 650 to 680kN/m², and yet
the corner soil/structure contact stresses might be considerably higher than edge stresses. The undrained shear strength of the Lias Clay averaged about 225kN/m² below the Limestone and it was unlikely that contact stresses higher than 700kN/m² could be sustained without local yield occurring with the ultimate bearing capacity estimated at 1300kN/m². These stress levels are normally restricted to the outer 2m of the slabs so that yielding of the piles into the top of the Limestone may help reduce the bending moments theoretically applied by the founding strata to the structure. Even so the analysis again highlighted the shortfall in the pile design. The analysis for the distribution of stresses, the range of settlements and the settlement profile is sensitive to the stiffness parameters used in the calculations. However the designer found that although total settlements were reduced by assuming greater stiffness of the soil, differential settlements were not significantly reduced (Table 2.3/1). The differential settlement value of 7.7mm (Table 2.3/1) was thus still significantly greater than that specified by the designer.

The maximum pile load due to the dead load plus live load (wheat plus wind) was calculated as 105 tonne, which did not take into account the possibility of negative skin friction. This would occur due to the settlement of the uncompacted fill following pile instillation as a result of dissipation of excess pore pressures induced by piling. Based on an undrained shear strength of 50kN/m², a single 9m long pile might expect to attract a maximum down drag of about 40 to 75 tonne (depending upon the adhesion factor) but in a group centred at 1.5m in both directions the maximum weight of soil available was estimated to induce a down drag of 37 tonne on each pile. From this it was estimated that the 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. With the piling contract already let the geotechnical consultant recommended that a greater number of pile tests than is usual be carried out in order to verify the pile design.
It was pointed out at this time that because of the likelihood of negative skin friction;

the load required for a pile test requires careful consideration. In the case considered a 533mm diameter pile designed to accept a design load of 75 tonne might transfer 110 tonne 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 tonne at the top of the pile but only 45 tonne on the rock at the toe of the pile.

The other problems anticipated before the second site investigation proved to be less intractable, but again highlighted the need to specify the site investigation to produce the desired parameters in view of the overall conceptual design.

The first site investigation had significantly underestimated the undrained shear strength of the Lias Clay and had overestimated its variability. This was thought primarily to be due to sample disturbance. The limited number of boreholes penetrating the limestone had produced a wide range of thicknesses. The second site investigation, employing a tighter specification for sampling and defining the parameters required to justify the pile design showed the Lias Clay to be a more uniform material and in a larger number of penetrations showed the limestone to have a more uniform thickness.

With this knowledge the combination of rock thickness and the strength of the underlying Lias Clay was thought to be adequate, provided that the piles did not penetrate the rock by a significant amount.

2.3.3. Design Parameters

The parameters are summarised in Table 2.3/2 for both the Wheat and the Flour Silos. In the case of the Wheat Silo an allowance for the combined stiffness of the raft and the structure was made as a best
estimate of 11.5x10^6 kNm²/m width at the time of running the initial settlement programs. The designers were attempting at this time to evaluate an equivalent stiffness of the raft taking into account the superstructure. Potentially the superstructure is very stiff as the silos produce a cell type of structure. The area of uncertainty is as a result of the connection between the bins and the raft. Externally the raft is connected to the bins by the reinforced columns which are an integral part of the external reinforced concrete wall. Internally the connection is by 10 cruciform reinforced concrete columns and it is here that the difficulty arises in the assessment of how much of the inherent stiffness of the superstructure is imparted to the equivalent raft/superstructure stiffness.

In the case of the Flour Silo a best guessed estimate for the equivalent stiffness for the raft taking into account the columns and the walls was made as 8.3x10^6 kNm²/m width. Again the structural designers were attempting to evaluate an equivalent stiffness at the time of the analysis.

2.3.4. Young's Modulus Value for Concrete

It should be noted that the long term equivalent value for E may be significantly reduced due to creep and shrinkage effects (CP110:1972 and Hand book on the Unified Code for Structural Concrete 1974). In this instance knowing the age at which the structure was loaded and the duration of the loading the equivalent value of E should be taken as 13000N/mm². The implications of this in the light of the soil-structure interaction analysis predicted settlement is discussed later.

The following soil parameters are a summary of those described in section 3, which were considered during the soil-structure interaction analysis.
2.3.5. Ground Water

a) from the first site investigation

Water strikes were made just below the rock/fill interface. In one borehole the water was sealed off by driving a casing into the limestone to a depth of 11.5m below ground level. An overnight observation after the removal of the casing showed the water level to rise from 9.8m to 8.8m below ground level.
### Table 2.3/1 Predicted and Measured Maximum Settlements for the Wheat Silo

<table>
<thead>
<tr>
<th>Stiffness of the Structure $E_1 \times 10^6$ (kNm²/m width)</th>
<th>Modulus Profile of the Soil $E_2$ (MN/m²)</th>
<th>Maximum Settlement (mm)</th>
<th>Differential Settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHORT CENTRELINE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.86</td>
<td>25</td>
<td>100</td>
<td>180.1</td>
</tr>
<tr>
<td>11.5</td>
<td>25</td>
<td>100</td>
<td>179.3</td>
</tr>
<tr>
<td>20.0</td>
<td>25</td>
<td>100</td>
<td>178.1</td>
</tr>
<tr>
<td>30.0</td>
<td>25</td>
<td>100</td>
<td>177.6</td>
</tr>
<tr>
<td>60.0</td>
<td>25</td>
<td>100</td>
<td>177.0</td>
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<tr>
<td>100.0</td>
<td>25</td>
<td>100</td>
<td>176.8</td>
</tr>
<tr>
<td>8.86</td>
<td>35</td>
<td>100</td>
<td>136.9</td>
</tr>
<tr>
<td>MEASURED SETTLEMENTS ON THE SHORT CENTRELINE</td>
<td></td>
<td>21.0</td>
<td>2.0</td>
</tr>
<tr>
<td>LONG CENTRE LINE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.86</td>
<td>25</td>
<td>100</td>
<td>194.3</td>
</tr>
<tr>
<td>MEASURED SETTLEMENTS ON THE LONG CENTRELINE</td>
<td></td>
<td>21.0</td>
<td>10.0</td>
</tr>
</tbody>
</table>

### Table 2.3/2 Design Parameters for the Wheat and Flour Silos

<table>
<thead>
<tr>
<th>Plan Dimensions (mm)</th>
<th>Wheat Silo</th>
<th>Flour Silo</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raft Thickness (mm)</td>
<td>16000x26500</td>
<td>14750x21500</td>
</tr>
<tr>
<td>Total Load (kN)</td>
<td>147148</td>
<td>97039</td>
</tr>
<tr>
<td>Average Contact Stress (kN/m²)</td>
<td>347</td>
<td>306</td>
</tr>
<tr>
<td>(including weight of the raft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average Contact Stress (kN/m²)</td>
<td>363</td>
<td>321</td>
</tr>
<tr>
<td>(including the piles)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E for Concrete (N/mm²)</td>
<td>25000</td>
<td>26000</td>
</tr>
<tr>
<td>EI for the Raft ($x10^6$ kNm²/m width)</td>
<td>8.8</td>
<td>7.31</td>
</tr>
</tbody>
</table>
Table 2.3/3 Parameters of the Fill Material Considered during the Design

<table>
<thead>
<tr>
<th>$c_u$ (kN/m$^2$)</th>
<th>$m_v$ (m$^2$/MN)</th>
<th>Compressibility</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>25-100</td>
<td>0.2-0.5</td>
<td>medium to high</td>
<td>36-59.2</td>
<td>14-19.5</td>
<td>22-39.7</td>
</tr>
</tbody>
</table>

*The two higher values indicated in Table 1 were attributed to sampling from intact lumps of boulder clay from the fill.*

Table 2.3/4 Parameters of the Limestone Considered During the Design

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>Unconfined Compressive Test (MN/m$^2$)</th>
<th>RQD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The Initial Investigation</td>
<td></td>
</tr>
<tr>
<td>3.9</td>
<td>21.2</td>
<td></td>
</tr>
<tr>
<td>3.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>The Second Investigation</td>
<td></td>
</tr>
<tr>
<td>2.12</td>
<td>from 23 tests</td>
<td>from 6 cores</td>
</tr>
<tr>
<td>2.32</td>
<td>lowest value 2</td>
<td>48-96</td>
</tr>
<tr>
<td>2.67</td>
<td>average 8</td>
<td></td>
</tr>
<tr>
<td>2.70</td>
<td>range 2 to 20</td>
<td></td>
</tr>
<tr>
<td>2.81</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.10</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 2.3/5 Parameters for the Lias Clay Considered During the Design

<table>
<thead>
<tr>
<th>Undrained Shear Strength (kN/m²)</th>
<th>Hand Held Penetrometer</th>
<th>Drained Reload Test</th>
<th>Oedometer E (MN/m²)</th>
<th>Oedometer mₗ (m²/MN)</th>
<th>Oedometer E (MN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The Initial Investigation</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>260</td>
<td>70</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>230</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The Second Investigation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*225</td>
<td>44.4</td>
<td>0.031-0.050</td>
<td>25-35</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>82.1</td>
<td></td>
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</tr>
</tbody>
</table>

* from 13 undrained triaxial compression tests in the 10m below the Limestone

+ the range is taken from 12 tests
2.4. Corby Design Analysis

2.4.1. The Design Model

In view of both the limited time and financial resources the geotechnical consultant decided that in modelling the soil and the structure very considerable simplifications were necessary. The object of the soil-structure interaction analysis was primarily to determine the differential settlements and bending moments to be expected in the raft slabs. Fig.2.4/1 shows the way in which the structure was modelled. A Winkler beam model was used to derive the stresses to be applied to a three dimensional fully flexible foundation on an elastic soil.

The Winkler beam model is a two dimensional beam on a spring model and derives the stresses to be applied to the soil model by assignment of:

1) A stiffness value for the beam equal to that for a 1m wide strip of the foundation slab or an equivalent stiffness value taking into account the whole structure.

2) An equivalent loading regime consistent with that applied to a 1m wide strip of the foundation slab.

3) Initial values for the spring constants representing the modulus of subgrade reaction.

The resultant stresses were applied to the soil model which calculated the displacements and the stresses at specific points within or on the surface of an elastic solid due to the application of any number of rectangular fully flexible uniformly loaded areas.
The model allows for the soil to be divided into a number of horizontal layers, each of which may be assigned a different Young's Modulus and Poisson's Ratio value.

The ratio of applied stress/calculated settlement for a particular point was then used to recalculate the spring stiffness, representing the modulus of subgrade reaction, to be used in the next Winkler beam analysis to recalculate a better stress distribution to be applied to the soil model.

This iterative process was carried out until reasonable agreement between the modulus of subgrade reaction values introduced to the Winkler beam model and those derived from the elastic soil model was obtained.

The initial analysis carried out by the geotechnical consultant used two programs commercially available from Geocomp U.K. Ltd.

RECTS is a program which calculates displacements and stresses at specific points within or on the surface of an elastic solid due to the application of rectangular fully flexible uniformly loaded areas and is thus the soil model. This program was used throughout the analysis.

RETWALL 2 is a program which uses a beam on spring model to analyze retaining walls. With simple modifications to the input data the program was used by the consultant to model a Winkler beam. Two runs were made with this program at the start of the analysis which yielded reasonable results. The limitations of this method became apparent as the beam was stiffened and significant errors arose.

The consultant assumed that this resulted from rounding errors produced during the calculation by the computer for which the program
had been written. Subsequent analysis was carried out using a less sophisticated program, Winkler, written by the specialist geotechnical consultant during the period in which the analysis was carried out. This program modelled the structure by discretising the beam into a series of elements with applied forces and springs at the nodes between the elements. Using finite difference techniques a stiffness matrix was assembled and the displacements normal to the beam were obtained by inversion. Bending moments and contact stresses between the beam were obtained by normal methods.

2.4.2. Design Assumptions

As time was of the essence it was necessary to make a number of assumptions and simplifications. They were described as follows in the geotechnical consultants report to the consultant.

1) It was assumed that the piles had the effect of lowering the basement and to make the analysis simple it was modelled as resting on top of the Lias Clay. This meant that

   a) compression in the pile was ignored.
   b) compression at the pile/limestone contact was ignored.
   c) compression in the limestone was ignored.

2) The ability of the limestone to spread the load applied to it was modelled by increasing the loaded area on the Lias Clay and decreasing the applied stress. The load was assumed to spread through the Limestone at 2 vertically to 1 horizontally giving an area of length +2.7m by breadth +2.7m to which the loads should be applied.
3) The structure was modelled only at the centre lines as an incremental beam 1m wide, part of an infinitely long strip foundation with the length in turn normal to each centre line. This length was represented by 41 points.

4) Analysis was essential before a considerable amount of testing had been completed. The only soils data available were from the oedometer test from which a value of Young's Modulus to a depth of 30m of 25MN/m² was indicated. Limited analysis was also carried out for a value of 35MN/m² indicated by the upper bound of the oedometer testing.

5) To model the Lias Clay outside the depth significantly stressed by the foundation a value of Young's Modulus of 100MN/m² was assigned to a depth of 300m in the Lias Clay. Below this depth Young's Modulus was assigned an infinite value.

6) The design engineer gave the raft stiffness values

Wheat Silo \( EI = 8.87 \times 10^6 \text{kNm}^2/\text{m width} \)
Flour Silo \( EI = 7.31 \times 10^6 \text{kNm}^2/\text{m width} \)

It was considered that the superstructure must increase the overall structural stiffness of the structure although difficult to assess. Values for the combined stiffness of the superstructure and the slab supplied by the design engineer were

Wheat Silo \( E = 11.5 \times 10^6 \text{kNm}^2/\text{m width} \)
Flour Silo \( E = 8.5 \times 10^6 \text{kNm}^2/\text{m width} \).

It has been suggested that "footing analogies provide a tractable solution for the practising engineer, but they bear little resemblance to the piles they attempt to model" (Douglas and Butterfield, 1984).
However in this case the piles did take the structure down to the Limestone and the fill material only contributed load in the form of skin friction. The shortfalls in the analysis were the absence of a method of predicting failure loads in the Limestone and a degree of uncertainty as to whether the Lias Clay would yield around the edge of the proposed loaded area. The consultant did however suggest that yielding of the Lias Clay would redistribute the stresses and thus reduce the bending moments inducing slab differential settlement.

The use of elasticity has recently been questioned (Jardine et al 1986) in that soil-structure interaction computations and interpretation of field measurement may be misleading and that small strain non-linearity and consideration of local failure have important implications in considering soil-structure interaction at working loads. Poulos (1989), however, maintains that at normal working loads (40 to 50% of ultimate) non-linear behaviour of soil generally does not have a substantial influence on pile settlement. Cheung et al (1988) state that empirical rules are adequate for the prediction of the perfectly rigid pile cap. Chow (1989) only considered elastic solutions as adequate field and experimental evidence suggested that the load settlement response of piled foundations was essentially linear; Cooke (1986) stated that the behaviour is elastic until the ultimate bearing capacity is reached.

Poulos (1989) describes various design tools, from simple empirical techniques through boundary element techniques to full three-dimensional finite element techniques, to be used in the analysis of the appropriate problem in terms of both its complexity and the cost of the structure. Both he and other authors, however, qualify the use of any method no matter how simple or complex with the requirement that the appropriate soil parameters must be used (Gezetas et al, (1985), Jardine et al, (1986), Cooke, (1986) and Cheung, (1986)).
The designers view that, considering the time limitations and the assumptions and simplifications listed above, he could make an engineering judgement on the results of his simple model are still in keeping with current thinking as discussed above. Whilst the laboratory techniques for producing local strain measurements, at strains appropriate to those under the foundation, have improved dramatically producing a more realistic value for the elastic modulus (Jardine et al 1984, and Clayton and Khatrush, 1986), the design method is still a useful tool if the limitations of its predictions are realised.

2.4.3. The Design Results

Because of the limited time only the Wheat Silo analysis was completed in time to be of use to the design engineer in assessment of the pile design. Some estimates of the maximum settlements and moments likely in the Flour Silo were available.

Wheat Silo Analysis

Figs.2.4/2 and 2.4/3 show the contact stresses along the short and long centre lines of the Wheat Silo, while Figs.2.4/4 and 2.4/5 show the settlements along the same centre lines, both as predicted from the computer analysis. Figs.2.4/6 and 2.4/7 show the distribution of bending moment per meter width predicted by the analysis.

The results are summarised in Table 2.4/1.

As was mentioned earlier limited analysis was carried out with a higher soil modulus value \( E_s \) of 35MN/m\(^2\) instead of 25MN/m\(^2\). The results of this analysis were that total settlements were decreased to about 135mm without a significant reduction in differential settlement.
Stiffness of the Soil

It was shown that the total settlements showed a linear reduction when increasing Young's Modulus from 25 to 35MN/m². These were the upper and lower bounds of the oedometer tests and the undrained Young's Modulus $E_u$ obtained from the undrained triaxial tests assuming an elastic material with Poisson's ratio equal to 0.25. On theoretical grounds the undrained value $E_u$ should be reduced to obtain the drained modulus $E'$ suitable for determining settlement in the long term. Elastic theory suggests that $E' = 0.83E_u = 0.831/n'$. However, experience, sample disturbance effects and consideration of stress paths and pore pressure coefficients suggest that $1/m_v$ values will lead to overestimates of combined immediate and consolidation settlement.* Fig.2.1/12 shows the values of $1/m_v$, oedometer tests plotted with the approximate values of $E_u$ obtained from the stress-strain curves of the undrained triaxial tests against depth below ground level. It can be seen that most values are between the values used in the analysis of 25 to 35MN/m² and that there is no apparent improvement with depth.

The two drained reload triaxial compression tests show a significant increase in the value of drained Young's Modulus $E'$. Only two specimens were tested, one from a depth of 12.63m below ground level and one from 22.47m below ground level. The values of $E'$ were 44.4MN/m² and 82.1MN/m² respectively. Although these results indicate the expected increase in $E'$ with depth there are not enough data available for the same depths or indeed any intermediate depths. It could be argued that the two results fall within the bounded region suggested by the those results from the oedometer and undrained triaxial tests (Fig. 2.1/12). The object of the drained reload tests was to produce Young's Modulus values unaffected by loosening of fissures or bedding between the sample and the platens.

* See Appendix A
If an average value of 65MN/m² were taken, and the linear response of settlement to increasing the modulus from 25 to 35MN/m² is valid, then the predicted settlements would be reduced to a value of about 70mm. However increasing $E'_1$ to 65MN/m² may have some effect on the analytical model as it is significantly closer to the value of $E_2$ in the model of 100MN/m², which was used to model the soil outside the significantly loaded influence of the structure. Without alteration to $E_2$ the settlements suggested by modulus values derived from the empirical formulae tabulated in Table 2.1/10 are as follows,

a) $220\text{CU Butler (1974)}$ 80mm

b) $400\text{CU Butler (1974)}$ 45mm

c) $E_{u0.1}$ lower bound Jardine (1986) 50mm

d) $E_{u0.1}$ upper bound Jardine (1986) 40mm

External strain measurement was employed for the range 25 to 35MN/m² suggested by the oedometer tests and for the average 65MN/m² suggested by the drained reload test. The increase from 35MN/m² to 65MN/m² is a substantial one and yet internal strain measurement such as that described by Burland and Symes (1982) and Clayton and Khatrush (1986) avoids bedding errors which the reload test hopes to avoid but the drained reload test may, because of the method, result in consolidation. Added to this the effect of sample disturbance may well make the value of 65MN/m² a considerable underestimate of Young's Modulus. Miller (1980) using local strain measurement yielded Young's Modulus values at least 2.5 times higher than those obtained from conventional measurement taken outside the triaxial cell. Further major differences are found when the specimens are loaded from their insitu anisotropic initial stress state rather than from an arbitrary isotropic stress state (Lambe 1967).
The conclusion to be drawn from this is that even the use of 65MN/m² for Young's Modulus $E_1$ may produce extreme overestimates of settlement despite laboratory tests on good quality core.

Had more time been available to the geotechnical consultant a more comprehensive range of laboratory testing could have been completed for the oedometer and drained reload tests. More confidence could then have been applied to the effect changing the stiffness profile had on the contact stress, bending moment, total settlement and differential settlement.

**Stiffness of the Structure**

The designer had considerable problems in assessing the contribution the superstructure would make to the overall stiffness of the structure. For the Wheat Silo analysis the conservative approach of only including the raft stiffness of $8.87 \times 10^6$ kNm²/m width was applied. Analysis was undertaken, by the geotechnical consultant, for the variation of differential settlement and moment with increasing $EI$ and the results are shown in Fig.2.4/B and Fig.2.4/9 respectively for the short centre-line.

A conservative estimate of the combined stiffness was thought to be $11.5 \times 10^6$ kNm²/m width which has the result of reducing the differential settlement from 7.7 to 6.0mm. It was estimated that the combined stiffness might reach a maximum value of $10^8$ kNm²/m width (Meyerhof 1953) which as can be seen from the plot would reduce the predicted differential settlement to 1mm. The difficulty in assessing the overall stiffness arises due to the discontinuity between foundation slab and the very stiff cell construction of the 15 wheat bins. It has been shown that in the case of a sugar floor, again separated by columns from the raft slab, very little additional stiffness is gained as the columns try to form pinned joints at both top and bottom, when settlement is large, due to lack of lateral stability (Burland and
Davidson 1976). This lack of lateral stability is thought to be caused because the sugar floor is not connected to the silo walls. This situation does not arise in the case of the Wheat Silo as the grain bin floors are jointed to the walls forming a box construction. This is further stiffened by the external columns being integral with the wall forming pilasters and by the grain bins stiffening the grain floor. Hence it is felt that in this case the columns could add to the combined stiffness of the structure.

Although the design approach was thought to be conservative it must be noted that in section 2.3.4 the designers gave a value of Young’s Modulus of 25kN/mm² for the concrete used in the Wheat and Flour Silos. This value is in agreement with CP110 (1972) 24.2.2. as this is the value assigned for 25N/mm² concrete but for short term loading. The live load in this structure due to its very nature is a real load, and although not constant, is a long term load of substantial value. BS5400: Part4 (1984), CP110 (1972) A2.2. and the Handbook on the Unified Code for Structural Concrete (1974) state that due to creep the effective modulus is reduced. In the unified handbook the simplified correction is dependent upon the age of the structure when it is loaded and the duration of the load. The load at present has been maintained for between 100 to 1000 days which will have had the effect of reducing the effective Young’s Modulus by a factor of 2 to 3 times. Thus in Fig.2.4/8 it can be seen that this will have a significant effect on the differential settlement depending upon the overall stiffness value used in the computation.

**Corner Stresses**

The analysis has shown that for the Wheat Silo the soil/structure contact stress is 223kN/m² at the centre of short centreline and that the edge contact stress is 680kN/m² giving a ratio of 1:3. It is normally expected that corner stresses are higher than edge stresses and thus the corner stresses would be predicted as greater than
680kN/m². With the undrained shear strength for the Lias Clay averaging about 225kN/m² it is unlikely that values higher than 700kN/m² could be sustained without local yield occurring. In the case of the Wheat Silo the higher edge stress effect is predicted to involve the outer 2m of the slab, so that if local yield into the limestone occurs it will have the effect of redistributing the contact stresses and may help to reduce the bending moments theoretically applied by the founding strata to the structure. Whitaker (1957,1960) showed that the load distribution, for a pile spacing/diameter ratio of 2, would be 5 times at the corner and about twice at the edge and that, for a pile spacing/diameter ratio of 4, it would be 4 times at the corner and twice at the edge.

Indications from monitored structures are as follows:

Cooke et al (1981) for a pile spacing/diameter ratio of 3.55 showed the load carried by a corner pile to be 2.2 times that carried by an internal pile and that an edge pile carried approximately 1.7 times that of an internal pile.

Hight and Green (1976) for a pile spacing/diameter ratio of 3, on a more complicated structure incorporating a central core, showed that the measured pile loads expressed as a proportion of the mean pile load varied from 0.8 between the core and 1.9 at the corner.

The problem is how to equate these pile load distributions to the analysis for this structure. The above sited cases and others, Furley and Curtis (1981) and Hooper (1979), compare pile loads and raft contact pressures for piled raft foundations with the piles loaded on London Clay.
The result of the analysis showing these high edge and corner contact stresses was to insert an extra three piles under each external and corner column to give the pile arrangement shown in Fig. 2.1/1.

More recent thinking concurs with the above. Many structures are highly rigid by the time of completion and in these cases the load distribution depends particularly on the number of piles and their spacing (Cooke 1986). For most common spacings Cooke suggests that corner piles could be expected to carry at least twice the load on the interior piles and edge piles at least one and a half times the load on interior piles.
### Table 2.4/1 Summary of the Wheat Silo Analysis

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<tr>
<th></th>
<th>Centreline</th>
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</thead>
<tbody>
<tr>
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<td>Short</td>
<td>Long</td>
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<td>Edge Soil/Structure</td>
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<td>contact stress (kN/m²)</td>
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<td>660</td>
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<td>Centre soil/structure</td>
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</tr>
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<td>contact stress (kN/m²)</td>
<td>223</td>
<td>249</td>
</tr>
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<td>Maximum settlement (mm)</td>
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<td>194</td>
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<tr>
<td>Differential settlement (mm)</td>
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<td>33.8</td>
</tr>
<tr>
<td>Maximum Bending Moment (kN/m width)</td>
<td>1970</td>
<td>3430</td>
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</table>

### Table 2.4/2 Summary of the Flour Silo Analysis

<table>
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<th>LONG CENTRE-LINE</th>
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<td>Max settlement (mm)</td>
<td>150</td>
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<tr>
<td>Differential settlement (mm)</td>
<td>25-30</td>
</tr>
<tr>
<td>Max bending moment (kN/m width)</td>
<td>2500</td>
</tr>
</tbody>
</table>
THE STRUCTURE

533mm ø piles at 1.5m c/c both ways.

Upper Lias Clay

THE ANALYTICAL MODEL

Figure 2.4/1 The Model used in the analysis
Figure 2.4/2 Stress Profile for the Wheat Silo Short Centreline
Figure 2.4/3 Stress Profile for the Wheat Silo Long Centreline
Figure 2.4/4 Settlement Profile for the Wheat Silo Short Centreline
Wheat Silo length + 2.7m at Top of Lias Clay

Figure 2.4/5 Settlement Profile for the Wheat Silo Long Centreline
Figure 2.4/6 Bending Moment Diagram for the Wheat Silo Short Centreline

Wheat Silo Width + 2.7m at Top of Lias Clay
Figure 2.4/7 Bending Moment Diagram for the Wheat Silo Short Centreline

Wheat Silo Length + 2.7m at Top of Lias Clay
Figure 2.4/8 Variation in Differential Settlement across the Slab due to the Assumption Made on Structural Stiffness of the Wheat Silo
Figure 2.4/9 Variation in ending Moment across the Slab due to the Assumption Made on Structural Stiffness of the Wheat Silo
2.5. Scope of the Instrumentation at Corby

The purpose of the instrumentation was to monitor the progress of settlement under load to confirm the calculated settlements had been over-estimated. Also, although the predicted settlements were not particularly large, the structures are of interest as they consist of a number of relatively rigid slabs which are not structurally rigidly connected. The design predicted that there was little differential settlement expected within each slab block and that the majority of the movement was expected to take place at the junction between the blocks. The confirmation of the design methods predictions would provide a documented case record on which to base future design.

Additional benefits to the consultant and to the client were;

a) a knowledge of the rate of settlement from the long term observation would allow an assessment of the need for maintenance at construction joints, to floor finishes and in the extreme to machinery which spans the blocks.

b) settlement observations would allow sensible decisions to be made during the initial loading of the silos.

The location of the BRE settlement stations (Cheney 1974) is shown in Fig.2.5/1. The object was to monitor each loaded individual slab, with a greater number of stations to monitor the slabs carrying the greatest loads. At the insistence of the client's representative the permanent levelling stations were not located and fixed until the structure was complete. Thus during construction some temporary monitoring stations were established on site by university staff and were monitored during the construction by the Engineer's site staff.
When the structure was complete the exact location for each station was agreed so as to avoid interference with plant and equipment. Additional considerations taken into account during the siting of the stations were;

a) inter-station visibility,

b) continuity around the slab.

The latter was desirable as it completed the round of levelling without the need for temporary bench marks such as those provided by a crows foot.

The stations were fixed in position before the application of the quite considerable live load. This operation required tying up the temporary monitoring and the newly installed BRE settlement stations. As appears usual in the case of a large construction, monitoring by site staff is of secondary importance. As a result most of the initial temporary monitoring stations were lost in the enthusiasm of completing the finishes. A programme of remedial measures were carried out by the site staff to establish new stations and to relate them to the original stations wherever possible. The final outcome of this series of events was that the levels produced were of little use for the purpose of an accurate settlement record which could, with some confidence, be guaranteed to be within 1mm (Table 2.3/1). To gain an idea of the settlement at the end of construction back analysis was done from the results obtained of settlement under the influence of dead and live load, this is described in section 2.9

**BRE Settlement Bolt and Installation Details**

The settlement sockets and levelling bolt are based on a Building Research Station design and have been described by Cheney (1974).
As has already been mentioned earlier, the levelling stations were installed after the structure was complete at the request of the client. As a result the walls and columns had to be drilled. The holes were drilled oversize to enable a mortar surround to be placed to fix the settlement socket in position. The settlement sockets incorporate a perspex screw cap which minimises the visual disturbance. These have, in most cases, been painted during decoration of the walls and columns which further reduce the visual disturbance. The caps are removed with a key to install the levelling bolt during levelling surveys.

Levelling was performed using a Zeiss Ni 007 automatic precise level in conjunction with an Invar strip levelling staff.
Figure 2.5/1 Plan of the Mill Complex Showing the Position of the BRE Levelling Sockets
2.6. Measurement Procedures

The objective of the exercise was to refer all the settlement stations indicated in Fig.2.5/1 to the datum on the electricity sub station shown on Fig.2.1/3. As there were a number of difficulties in this exercise the following sections will discuss the route taken and outline the reasoning behind the decisions.

The Datum

The datum was located in the brick wall of the electricity substation building. The very light structure of this building was founded on the test piles that had been driven to the limestone bed. It was considered that the load as a function of piled raft slab load bearing capacity was negligible and that the building should not settle as it is some 50m from the main mill complex and on the opposite side of the road to the offices indicated on Fig.2.1/3. The first levelling station, and the one to which the datum was transferred, M02 is situated on the wall of the Tempering room Fig.2.1/3 and Fig.2.5/1.

This operation was carried out once only on each visit as the conditions did not allow the closure of the exercise back to this station. The two major reasons for this were as follows;

a) The site was particularly open. The only large structures of any note in the visible distance was the mill complex. This coupled with the local weather conditions meant that it could be virtually guaranteed that there would be significant turbulence in the area. Thus by ensuring a good transfer at the start of the exercise the procedure was not required on a second occasion at the end of the day.
b) The site is a particularly busy one. Picking a structure outside the sphere of influence of the highly loaded areas resulted in a location on the opposite side of the complex's access road close to the weigh bridge. This causes a number of interesting problems when precise levelling and is as a result better only undertaken once when you have the attention of lorry drivers on that one particular occasion.

To ensure the datum transfer three independent levelling readings were taken. After each transfer of level the level was re-positioned to gain another collimation thus negating the desire to read the same values on the staff.

The major points to be aware of whilst carrying out the operation of transferring the datum from the TBM to M02 were as follows;

1) after transportation in the car the level had to be set up on its legs and allowed to stand for at least 15 minutes to allow the temperature of the instrument to equilibrate. Close attention had to paid to the instrument on those occasions when it was particularly windy.

2) the level was set up as close the mid point between the TBM and M02 as possible so as to reduce collimation errors.

3) the site was nearly always very windy and thus the utmost patience had to be exercised by both the surveyor and the chainperson. The automatic collimation of the machine meant that any vibration induced by the wind resulted in flutter of the compensator and hence the reading was not taken.
4) the presence of the lorries not only results in the loss of the sight line but also results in an inability to take the reading due to heat hazes resulting from the proximity of the engines even when the sight line was not obstructed. It was imperative that the lorries left a sufficient gap to stop this problem, difficult to explain when the driver can see he is not obstructing your sight line.

5) the level was set up on soil and hence the greatest of care had to be taken to ensure the legs were firmly set and that as little movement as possible took place next to the legs as the weight of the operator can be a contributory factor to a change in collimation if the ground is soft.

Every care possible must be taken with the above points for the three independent sets of readings. These readings can then be readily checked quickly to assess the value of the reduced level for M02 and if it agrees for all three sets then the levelling exercise can proceed. It must be noted that the temptation to reduce the levels after each set of readings must be ignored as the psychological tendency to read the desired result and not the actual reading must be avoided.

**Levelling on the Outside of the Structure**

The external exercise had the following points to note. Points are made which have relevance to any external precise levelling, most of which are mentioned elsewhere (MacLeod and Paul 1984). Also points are made which have particular reference to Corby.

a) The legs are sited on tarmac which in hot weather will settle.
b) The external levelling bolts are all positioned on the external surface of a high structure. The prevailing wind conditions mean that these stations are all in regions of high turbulent air and thus care and patience must be exercised to obtain a good closure back onto M02.

c) The levelling stations M05 and M06 have significantly different reduced levels and the greatest of care needs to be exercised in the positioning of the level and the collimation of the level such that a TBM need not be used. The position to achieve this is on the ramp up to the Wheat Silo tipping bay and it is imperative that lorries be kept at a good distance away whilst this reading is taken to avoid them depressing the ground.

d) The level station M27 requires a change point other than a settlement bolt. A crows foot had to be used and this required careful positioning of the crows foot and diligent protection of it whilst the level was moved and the collimation re-established.

Care in carrying out the above points and closure back onto M02 ensured not only that the external level stations were read as accurately as possible but also that the levelling procedure could then be carried on independently for the internal settlement stations.

The Internal Levelling procedures

The internal exercise has a number of points which are of general interest when precise levelling inside a structure, these are outlined first followed by the particular points specific to this exercise.

1) The initial setting out of the stations requires careful thought planning and co-operation between all the parties.
involved. The primary items that have to borne in mind are that the surveyor requires a defined line of sight and that the client requires a structure which is either functional or aesthetically acceptable or in some cases both. If machinery is to be installed in the structure then the details of this must be known such that the lines of sight and the stations are not obscured, even then amendments to the machinery may result in the loss of a station if the stations are to be inserted during the construction sequence.

2) Installation during construction means that the station is solid as it can be tied in with the reinforcement or brickwork during the construction process.

3) Installation after construction and particularly after installation of any machinery means that the sight lines can be guaranteed baring any future alterations but requires the station to be grouted in.

4) Installation after construction requires a hole to be made. This can cause problems with both reinforced concrete and brickwork in that the former needs either to be drilled around any reinforcement or to cut through it and in the latter displacement of brickwork may be encountered.

5) Care must be taken to ensure that the perspex cap can be inserted or taken out at a later date and that the socket is protected from grout intrusion.

6) Transfer of the datum from external sources means a change of environment for the level. Once the level has been
moved inside it must be given time to reach ambient temperature.

7) Account must be made for the type of floor finish. Carpet will have obvious settlement implications and top quality floor surfaces may require the level legs to be restrained against collapse. In both case movement around the level must be undertaken carefully but in the later case particular care must be taken to avoid lateral sliding.

8) If heavy industrial machinery is running then the possibility of vibration of both the floor and columns must be appreciated and great patience exercised in taking the readings.

9) If noise is also present then provision for communication must be made so that all those involved are aware of the current requirements.

10) If internal compartmenting is encountered then it is wise to close the levelling in each compartment such that error trapping, correction and rectification are as easy as possible.

The internal route and the particular problems associated with each section may be followed on Fig.2.5/1 and are as follows:

1) of paramount importance was the transfer of the datum from M02 (external) to M28 (internal). This operation took place through a roller door with the level positioned externally. Patience and care had to be exercised as the level position was exposed to high turbulence and M28, the only station visible, was subject to vibration from plant above.
2) the level had to be allowed to reach ambient indoor temperature.

3) the level had to be set up with the use of a chain. The chain is three chains connected to one another centrally and on the other end is a cup into which the feet of the legs fit. The floor was highly polished tiles so care still had to be exercised so as not to disturb the level and the chains laterally.

4) the first loop was M28 to M35 closing back on M28. M36 was lost behind machinery.

5) the datum then required to be transferred from M35 to M37 through double fire doors.

6) the second loop is M37 to M43 closing on M37. The levelling is undertaken in a corridor to M40 and then through fire doors into the main warehouse for M41 to M43.

7) the third loop is from M37 to M44 to M47 closing back on M37. The transfer from M37 to M47 requires a crowsfoot to be used in a busy stairwell whilst negotiating two fire doors. The transfer also moves from indoors to outdoors to read M44 to M47 and indoors when closing back on M37.

8) the fourth loop is M28 to M07 to M12 closing on M07. The initial reading is important as M28 is usually vibrating.

9) the fifth loop is M12 to M13, M13 to M26 closing on M13. A check is made on M27 by moving outside through a fire escape. The transfer from M12 to M13 is via the Pellet Loading Bay. The exercise requires the crowsfoot to be
used as a TBM in the loading bay whilst negotiating two doorways.

Noise and vibration are experienced on loops one, four and five and vibration may also be a problem on loop two as the corridor and the specific area of the warehouse covered by stations M41 to M43 are under the Flour Silo.

The precise levelling was undertaken to monitor movement at a number of locations around the site. The overall objective with this type of exercise is to know the movement of any of the stations to within 1mm. To achieve this level of accuracy each loop on the exercise should close to better than 0.5mm. On this site the closure was allowed to be this much due to the site conditions of wind and vibration. The author has found that, under ideal conditions with a similar number of stations, closures are commonly better than 0.1mm.

It still remains, however, that the major source of error in the levelling exercise is the discipline of both the chainperson and the surveyor. The importance of the positioning, maintaining and cleanliness of the staff are more important than the ability to read the level. The discipline of checking the alignment of the staff through the telescope is necessary as is any check facilitated by the levelling method. In the case of the Zeiss the difference in the micrometer readings should be 5. If it is not then the verticality of the staff must be suspect and no matter how long it takes the readings must be retaken.
2.7 The Corby Mill Complex Live Loading

The dead loads and live loads used in any analysis were

1) the design dead loads and live wind and incidental slab loads, without safety factors, supplied by the design engineer

2) the live loads for the silos, without safety factors, supplied by the design engineer

3) for any subsequent back analysis the live loads for the silos supplied by the client from the stock control computer printout.

Dead Loads

The dead loads, as supplied by the designer, were given in terms of column loading with an additional value added for the dead load of the foundation raft slab located over the group of piles concerned with the column.

If the Wheat silo is analyzed on its own, ignoring the elevator and pre-clean area, there were four types of column loadings as indicated by the design engineer. These are summarised below in Table 2.7/1.

However although the Wheat Silo was slipformed, the foundation slab to the elevator and pre-clean area is a structural continuation of the Wheat Silo Foundation slab. There is a substantial wall dropping it to a lower level Fig.2.2/2. The elevator tower will influence the dead and live loads as the external and corner columns associated with this part of the structure will form a party wall between the Wheat Silo and the Elevator and Pre-clean areas. This will have the effect of increasing the type of columns by two. Two of the external
columns, on the short side, will now be internal wall columns and the two corner columns will be wall junction columns. These two types of column will have a different loading from any of the above tabled.

**Live Loads**

The live bin loads and incidental live loads used by the designer are in the same form as those given for the dead loads and are summarised in Table 2.7/2.

The curtain wall has the same effect on the dead loads and the live loads and thus the same proviso applies.

The true live load provided by the contents of the structure is well known at any time. Electronic instrumentation records the inflow and outflow of the grain to each bin, and a central computer keeps up-to-the-minute records of the stock in each bin. The loads are known to an accuracy of 100kg.

Fig.2.7/1 shows the bin arrangement for the Wheat Silo along with the position of the BRE settlement stations located in the walls and columns below the bins. Tables 2.7/3 and 2.7/4 show the bin loadings on the dates that levelling surveys were undertaken. Obviously there will be variation, daily deliveries and usage in the mill, but it is thought that only rarely will the bin loadings on the printout for that day be unrepresentative of the recent loading history for that bin.
Table 2.7/1 Live Loadings on the Four Column Types

<table>
<thead>
<tr>
<th></th>
<th>Internal Column</th>
<th>External Column Short Side</th>
<th>External Column Long Side</th>
<th>Corner Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load from the Structure</td>
<td>2051</td>
<td>1831</td>
<td>1831</td>
<td>1351</td>
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<tr>
<td>Dead Load from the Raft</td>
<td>1087</td>
<td>544</td>
<td>544</td>
<td>272</td>
</tr>
<tr>
<td>Total Dead Load to Each Pile Group</td>
<td>3138</td>
<td>2375</td>
<td>2375</td>
<td>1623</td>
</tr>
</tbody>
</table>

All loads are in kN

Table 2.7/2 Dead Loadings on the Four Column Types

<table>
<thead>
<tr>
<th></th>
<th>Internal Column</th>
<th>External Column Short Side</th>
<th>External Column Long Side</th>
<th>Corner Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Bin Load</td>
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<td>2964</td>
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<tr>
<td>Wind</td>
<td>101</td>
<td>51</td>
<td>303</td>
<td>152</td>
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<tr>
<td>Live Load on the Raft</td>
<td>325</td>
<td>163</td>
<td>163</td>
<td>82</td>
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</tbody>
</table>

All loads are in kN
### Table 2.7/3 Wheat Silo Bin Loadings (Bins 1 to 8) All Loads in kN

<table>
<thead>
<tr>
<th>Date</th>
<th>Elapsed Days</th>
<th>Wheat Silo Bin Number</th>
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<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
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<th>8</th>
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<td></td>
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<td>20/11</td>
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<td>502.1</td>
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<td>192.0</td>
<td>209.5</td>
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</tr>
<tr>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>27/1</td>
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<td>263.9</td>
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<td>138.1</td>
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<td>343.6</td>
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<td>349.6</td>
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### Table 2.7/4 Wheat Silo Bin Loadings (Bins 9 to 15) All Loads in kN

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<td>30/1</td>
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<td>298.7</td>
<td>414.4</td>
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<td>142.9</td>
<td>222.9</td>
<td>527.4</td>
<td>518.9</td>
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</table>
Figure 2.7/1 Plan of the Wheat Silo Showing Bin Locations and BRE

Settlement Socket Locations
2.8 Pile Heave

Preboring has commonly been quoted as a means of overcoming uplift and yet the piling contractor maintained that preboring to any depth was not necessary. In some cases it has been reported that preboring to reduce heave and uplift can be beneficial but that the cost may be prohibitive (Healey and Weltman 1980). In other cases a number of site records have shown that preboring has had little effect on the subsequent heave of the piles (Cole 1972). Even low displacement "H" piles, driven to overcome heave, have behaved as high displacement piles when the soil conditions have plugged the "H" section (Thorburn and Thorburn 1977). It would appear that there is no hard and fast rule and that the solution to pile heave is particularly site dependent. As a result of the uncertainty of the effect of preboring it has been suggested that it may be more economic to re-drive the piles (Healy and Wheltman 1980). This philosophy must be questioned for the case of closely spaced piles such as those under the Mill Complex.

Initially, up to the first pile load test, the site supervisory staff were in agreement with the piling contractor. Presumably this was the case as preboring would have slowed the site works. By the time the designer insisted on preboring, due to the volume of soil that required displacement to accommodate the piles, a large number of piles had been driven. The decision to prebore to 6m looked justified after the first pile load test. The test had shown the pile to have heaved away from its seating in the rock stratum. However subsequent re-tapping confirmed Cole's findings for one of his sites (Cole 1972) that preboring to 6m had not eliminated heave. It was also confirmed that preboring to 6m had not made any appreciable difference to the amount of re-tapping required to reseat the piles. A system not tried at Corby was that of multi-tubing. Multi-tubing has been found to be effective in some instances (Healy and Wheltman 1980, Thorburn and
Thorburn 1977 and Cole 1972) and has been found to be necessary in some others where collapse of the prebore has occurred (Cole 1972).

The volume of soil displacement under the complex must have resulted in some lateral displacement of the piles although there appears to be no mention of its occurrence at Corby. Hagerty and Peck (1971) have reported some lateral movements of piles and the long term nature of the displacements. Not only were individual piles shown to move as a result of the piling of an adjacent pile but clusters of piles were shown to move considerable amounts in relation to one another. Within 11 days no cluster had moved more than 25mm in relation to another. At 58 to 74 days this displacement had increased to between 380 and 610mm. Later surveys showed the clusters to be moving at 50 days after all piling in the area had been completed.

The sequence of piling is particularly important for large numbers of closely spaced piles where access may not be possible for re-driving (Healy and Weltman, 1980, Young and Thorburn, 1981, Hegerty and Peck, 1971 and Cole 1972). Healy and Weltman (1980) suggest that where groups are driven it is preferable to progress outwards from the centre. They suggest this is applicable up the 12D. This recommendation appears conservative if the predicted effect on an adjacent pile is calculated after Cole’s method (1972). The prediction chart in Cole’s paper does not plot a heave effect for diameters greater than 10D. In Fig. 2.8/1 the heave effect on each pile in a cluster of five piles is discussed with the heaves calculated from Cole’s chart Figure 2.8/2. In each case the central pile (pile 1) is driven first followed by the piles in sequence. Table 2.8/1 shows the effect on each pile as the spacing increases.
Methods of Predicting Heave

In each case a reported method for predicting pile heave will be compared with the actual heave observed for pile 261. This pile was the first to undergo the pile load test and resulted in a settlement of some 60mm before the pile was able to take load other than that attributable to skin friction.

The method after Hagerty and Peck (1971) is examined first.

Mean Rise in ground level in area of pile 261 Hagerty & Peck.(71)

\[
\text{Mean Rise} = 0.5 \times \frac{\text{volume of the pile installed}}{\text{surface area contained by group}}
\]

Single Pile Volume = \(\frac{\pi \times 533^2 \times 10.1}{4}\) m³

Surface area is 2.72 diameters x 3.75 diameters

Therefore Mean Rise in ground level is

\[
\frac{1}{2} \times \frac{\pi \times 0.533^2 \times 0.5 \times 10.1}{4 \times 2.72 \times 0.533 \times 3.75 \times 0.533}
\]

\[
= \frac{\pi \times 10.1}{8 \times 2.72 \times 3.75} = 0.389 \text{m}
\]

Hagerty & Peck now postulate that in a uniformly heaving mass of clay the upward movement would vary linearly with distance above the base of the clay. Inextensible vertical piles embedded in the clay would be lifted by the relative rise of the soil with respect to the upper part of the pile, but the rise of the lower part of the pile would exceed that of the surrounding soil at that level. Therefore, the lower half of the pile would be acted on by downward forces tending to reduce the total uplift of the pile.

Hagerty & Peck (1971) seem to suggest that in a uniform clay half the pile holds itself down and thus the pile heave should be half of the mean rise in soil.
Therefore Pile Heave = 0.5 \times 0.389 = 0.195m

The observed heave is about 60mm

Hagerty & Peck state that in one of their case records inextensible piles were effectively held down by a weathered rock zone into which the tips of the piles penetrated against the upward pull of heaving overlying silty clays and clayey silts.

Their other proviso is that the piles should be driven in a rather regular manner from one side on the foundation to the opposite side.

If the piles were prebored to 6m

\[
\text{ground heave} = \frac{1}{2} \times \frac{\pi \times 4.1 \times 0.5332}{4 \times 2.72 \times 0.533 \times 3.75 \times 0.533} = 0.158m \text{ at the surface}
\]

Thus pile heave = \frac{0.158}{2} = 0.079m

Preboring to 9m soil heave = 0.042m
Thus pile heave = 0.021m

Cole's Method

The chart after Cole (1972) reproduced as Fig.2.8/2. has been used. The piles around pile 261 are shown in terms of the spacing in pile diameters in Fig.2.8/3. The displacements according to Cole's chart are set out below.

<table>
<thead>
<tr>
<th>Pile Configuration</th>
<th>Spacing (D)</th>
<th>Heave</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 piles at 3.75 D</td>
<td>2x8.5</td>
<td>17.0mm</td>
</tr>
<tr>
<td>2 piles at 4.63 D</td>
<td>2x5.7</td>
<td>11.4mm</td>
</tr>
<tr>
<td>2 piles at 6.61 D</td>
<td>2x2.6</td>
<td>5.2mm</td>
</tr>
<tr>
<td>1 pile at 2.72 D</td>
<td>1x13.5</td>
<td>13.5mm</td>
</tr>
<tr>
<td>1 pile at 5.44 D</td>
<td>1x4.0</td>
<td>4.0mm</td>
</tr>
</tbody>
</table>

Total Heave for the piles driven in rows = 51.1mm
Pile heaves in the closely spaced Wheat Silo where the pile spacing to diameter ratios are 1.99 and 2.81 for the edge groups are as follows.

The lowest value of heave predicted for a pile driven first in the group would be 68mm. The worst case dependent upon the sequence of driving was 106mm.

Internally in a column group, where the pile spacing to diameter ratio is 2.81, the lowest value of heave predicted for a pile driven first in the group would be 64mm. For the worst driving sequence the pile heave would be 84mm.

It should be remembered that, in all the cases sited above, the heave is for the first pile driven and that all subsequent piles in the group will heave less and that the last pile driven will not heave at all. Thus it is quite readily evident that differential settlement is possible not only for the piles under the column location, but, if they are driven in a different order, from column location to column location. This has been observed elsewhere (Cole(1972), Hagerty & Peck(1971)), in case records as well as re-driving records.

Young & Thorburn (1981) suggest that re-driving should not be attempted until all piles have been installed and that they often require considerable energy input. If the pile re-drives easily the possibility and effect of tension failure should be considered.

In conclusion Hagerty & Peck's (1971) method overestimates the heaves measured (Table 2.2/2) and in particular for the pile 261 for which an observed settlement of 60mm is recorded. It has been suggested by Hagerty and Peck (1971) that piles socketed into rock may initially resist heave. This explanation was put forward as the potential reason for their method over predicting the measured heave in one of their case records. Cole's method predicted the observed settlement well, 51mm, against the observed 60mm. This assumes that pile 261 was
driven in the least favourable manner. The above assumes that full heave recovery is obtained by re-tapping and that the toe again reseats in the rock. The re-tap records show that preboring to 6m did not seem to affect the amount of heave experienced by the pile in comparison to others that had not been prebored. It is however clear that a substantial amount of material would have had to be displaced for the piles that were not prebored and that this may have resulted in lateral movement which was not monitored. With such closely spaced piles it must be good practice to drive, monitor, re-tap and re-monitor if these potentially large horizontal and vertical displacements are to be avoided.

The piling contractor supposedly proved that pre-boring would not be necessary by piling a straight line of three piles. It would appear that this practice should not be used to prove that heave will not be a problem. If piles are to be used to provide information on the likely behaviour of the working piles then two requirements must be fulfilled from the experience on this site.

1) The piles should be the same diameter as the working piles.

2) The spacing must be representative of the spacing under the foundation.

Additionally it would be prudent to look for lateral movement of the piles if heave is not detected on closely spaced clusters of piles.
Table 2.8/1 The Heave Effect on Piles (Figure 2.8/1) due to Driving Pile(s) and the Effect of Increasing Pile Spacing

<table>
<thead>
<tr>
<th>Pile Spacing in Diameters</th>
<th>Heave Effect Due To Driving the Next Pile(s) in Sequence (mm)</th>
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<td></td>
<td>2.5</td>
</tr>
<tr>
<td>Heave on Pile 1</td>
<td>60</td>
</tr>
<tr>
<td>Heave on Pile 2</td>
<td>42</td>
</tr>
<tr>
<td>Heave on Pile 3</td>
<td>27</td>
</tr>
<tr>
<td>Heave on Pile 4</td>
<td>15</td>
</tr>
</tbody>
</table>
Figure 2.8/1 Sequence of Driving a Cluster of 5 Piles

Figure 2.8/2 Uplift of a Pile Due to Driving One Adjacent Pile (after Cole 1972)
Figure 2.8/3  The Piling Around Pile 261
2.9 Analysis of Results

2.9.1 General Principles
As the construction of the Mill Complex progressed in the early stages, the BRE settlement sockets were installed. During the remainder of the construction phase the site staff monitored the change in level due to the increasing construction loads. As is often the case with these tasks, they are of very low priority when compared with the construction process, the operation was not carried out well. This may have resulted because of poor explanation of the value of the readings. However, the trends of the levelling were not checked so that when an analysis was undertaken it was not possible to rely upon the datum levels to within plus or minus 5mm. This was approximately the magnitude of the settlement expected during the construction phase.

Without level data to see how the rafts were behaving under load the client was persuaded to leave a constant live load in the Wheat Silo bins when the Mill became operational. The mill was levelled before the live load addition, during the loading, at full load and for the few days after the addition of the full load was added. Once the levelling showed no detrimental affect the client insisted the mill became operational. Knowing the live load addition and the settlement due to the live load, an exercise was conducted to estimate the settlement due to the live load addition by back analysis. The calculated value was added to the settlement monitored by precise levelling undertaken by the University.

The analysis undertaken by the geotechnical consultant was on a layered soil assuming $E_1$ 0-30m=25MN/m² and $E_2$ 30-300m=100MN/m² which predicted a settlement of 170mm. Analysis of the time settlement relationships allowed a simple assessment of the primary consolidation and the time for 90% consolidation to be made from the oedometer laboratory data.
Using Terzaghi's theory of one dimensional consolidation to predict $t_90$, $t_70$ and $t_50$ the length of the drainage path was assessed. There are a number of possibilities of which three are considered below.

1) The soil will drain to the Lias Limestone junction and downwards to the Lias Clay at the limit of the sphere of influence equal to the width of the foundation, at a depth of 15.9m. Thus the model will assume a double drainage with a path length of 7.95m. This model will predict the shortest time for primary consolidation and the smallest value of primary settlement.

2) The soil will drain laterally to the Lias Clay outside the sphere of influence of the stress bulb. Nominally this results in a drainage path approximately equal to half the width of the foundation. The oedometer test cannot give values of $c_v$ and $m_v$ but it was considered that in this case the values of $c_v$ and $m_v$ were appropriate. The detailed borehole log of the second investigation indicated that the Lias Clay was an intact material with no apparent fissuring or fabric to indicate that lateral drainage would be substantially greater than vertical drainage. The assumption made here will give the most conservative figures.

3) The soil will drain to the Lias Limestone junction and the Lias Clay at the sphere of influence of the stress bowl was taken as impermeable. This assumed single drainage with a path length of 15.9m. This produced values for $t_90$ and an assessment of the primary consolidation within the bounds of those predicted in 2) above.
The following calculations were carried out using the principles outlined in 2) as they gave the largest value of settlement that might be predicted by this simple method.

The following sections look at the response of the structure to the loading history and endeavour to calculate appropriate values of parameters to fit the model. These values will then be considered as to whether they are appropriate for the Lias Clay and or whether the use of such a simple model is viable to predict settlement accurately as opposed to only giving an indication of the likely settlements.

The problems in back analyzing such a structure are that the live loads are not only very large but that they are also continually variable. However, because of the computer monitoring it is possible to take a sample of the load history. It must be remembered that it is only a sample because, being a working mill, if the loads were sampled one hour later there may be significant changes in the bin loading. The rate at which the structure responds to these load changes is not known and thus the following assessment can only be a guide. An attempt has been made to correct the levelling data to give a time settlement response for a constant live load.

The first section takes the response of the Wheat Silo to its first live load and then corrects subsequent levels by ratioing the initial response to the proportion of the load that is in the bin at the time of levelling. The calculation is simple, it assumes the system is elastic with full strain recovery, that the structure responds to the loads instantly and that the columns take a quarter of the load from each bin it supports.

The results of the above were then plotted as log(time)-settlement curves and the straight line portion drawn (Figs. 2.9/1 to 2.9/4). Values of $T_v$ for $U_{50}$ and $U_{70}$ were taken from the corrected plot of Fox's solution (1948) after Janbu, Bjerrum and Kjaernsli (1956) and
the times calculated. The settlement response for 50 and 70% consolidation were taken from the above plots and were factored to give estimates for the primary consolidation.

The above values of primary consolidation were then used to determine the combination of modulus values used in the consultants model for it to have predicted the estimates of primary consolidation.

2.9.2 Back-Analysis for the Construction Settlement on the Wheat Silo.

The references to bin numbers and to monitoring stations should be read in conjunction with Fig.2.7/1.

To carry out analysis including bins W1 to W6 was considered to be unwise as these may be too greatly influenced by the Elevator and Pre-clean area as well as the Pellet Outload to the east of the slab.

The dead load settlement for the Wheat Silo slab was approximated as the average of the above calculations

\[ \frac{5.81 + 4.64 + 4.00}{3} = 4.82 \text{mm} \]

Thus 4.8mm was added to all the levels to approximate the total settlement of each level station.

2.9.3 Assessment of the Elastic Shortening of the Piles

The foundation was modelled ignoring the lightly loaded area of slab at the southern end of the Wheat Silo, the Elevator and Pre-Clean Area. The last line of piles at the southern extremity of the Silo slab are at the lower level of the Elevator and Pre-clean Slab some 4.63m below the Silo slab. As a result of this there will be a difference in elastic shortening between this line of piles and those
commonly under the Silo slab. This difference in shortening will be looked at in the context of the settlement profile of the slab.

The assessment has again been made for the loading data available from the 27/1/83 when the live load was built up uniformly to monitor the structure's behaviour as it was first filled with grain.

2.9.4 Assessment of the Long Term Young's Modulus for Concrete

For 20MN/m² concrete E is approximately 25kN/mm² (CP110)
From CP110 A2.2. A3 and The Handbook on the Unified Code for Structural Concrete and taking $\phi$1 for greater than 365 days = 0.96

Effective Long Term $E$ 12.5kN/m²
Difference in Length of the Piles 4.63m
Difference in Elastic Shortening 1.05mm

Allowance was made for the curtain wall which drops the Silo slab to the lower Elevator and Pre-clean level. The elastic shortening for this section of wall was about 0.1mm which has been deducted to give the above value.

The difference in elastic shortening is of the same order as the difference between successive lines of monitored stations working in a Northerly direction. This can be seen if Fig.2.7/1 is studied in conjunction with the Table 2.9/3. It might be suggested that the slab is stiff enough to be tilted at the angle indicated by the difference in elastic shortening but it is thought unlikely.

It is believed that the more lightly loaded Elevator and Pre-clean Slab is taking load from the southern end of the Wheat Silo Slab due to stress redistribution as this lower slab resists the settlement of the main slab. This will produce a decreasing stress profile towards
the Elevator and Pre-clean area and is thought to be the more likely solution.

2.9.5 Assessment of the Primary Consolidation.

Assessment of $t_{90}$ for the Wheat Silo.

Using Terzaghi's theory of one dimensional consolidation.

$$t_{90} = \frac{T_v \cdot H^2}{C_v}$$

$T_v$ range is from 0.84 to 1.0 dependent upon the $r_p$ ratio (Janbu, Bjerrum and Kjaernsli 1956).

Taking lateral drainage the drainage path length is half the width of the structure = $\frac{15.9}{2} = 7.95$m

$C_v$ - the average value of eleven results was taken from the second compression stage of the oedometer tests: mean value 0.75m$^2$/year.

Taking the mean $C_v$, $t_{90}$ ranges from 71 to 85 years. Assessment for $t_{90}$ taking the extremes of $C_v$ and the $r_p$ ratio into account gives a maximum range of 40 to 200 years for 90% consolidation. Again, as was pointed out earlier this is the most conservative estimate and will result in the largest settlement.

The Settlement response for a Continual Full Live Load.

An approximation of the response of the Wheat Silo Slab to a continual live load was made. An example is given for level station M17. Level stations M16, M19 and M20 were calculated in the same manor.
The bins associated with M17 are W1, W2, W4, W5 (Fig. 2.7/1).
Thus the load associated with M17 on 27/1/83 is (Tables 2.7/3 and 2.7/4)
\[(530.0 + 530.0 + 508.8 + 530.0) = 524.7 \text{ tonne}\]
The difference in settlement due to the addition of the live load
\[= 11.5 - 8.0 = 3.5 \text{mm}\]

Load on column 3/3/83 = 264.8 tonne

Load difference = 524.7 - 264.8 = 259.9 tonne

Difference in settlement if load on 3/3/83 were 524.7 tonne =

\[
\frac{\text{Load Diff} \times \text{Initial settlement diff due to live load}}{\text{Maximum Load}}
\]
\[= \frac{259.9 \times 3.5}{524.7} = 1.7 \text{mm}\]

Thus the adjusted settlement = 11.5 + 1.7 = 13.2mm

This process is repeated to produce Table 2.9/4.

These results are plotted on Figs. 2.9/1 to 2.9/4 for stations M16, M17, M19 and M20 respectively.

This simple analysis assumes elasticity in the Lias Clay and that there is full strain recovery in the system. It was thought to give the highest possible settlements.
Assessment of the Primary Consolidation from the Log Time Plots.

The following procedure was adopted:

1) An assessment of the straight line portion was made on the Log-(Time) Plots (Figs. 2.9/1 to 2.9/4).

2) The range of $T_v$ due to the $r_p$ ratio was determined for 50 and 70% consolidation using the plot after Janbu, Bjerrum and Kjaernsli (1956).
   - $U_{50} - T_v$ range 0.09 to 0.30
   - $U_{70} - T_v$ range 0.27 to 0.50

3) The time range for 50 and 70% consolidation was calculated:
   - $t_{50}$ ranges from 2775 to 9252 days
   - $t_{70}$ ranges from 8327 to 15420 days

4) The settlement at 50 and 70% consolidation was read from Figs. 2.9/1 to 2.9/4 taking off the 10mm due to the settlement up to the time of the live load addition.

5) The settlement differences were ratioed up to give ranges for 100% consolidation. Table 2.9/5 summarises the estimates obtained for stations M16, M17, M19 and M20.

Hence the likely range of primary consolidation ranges between 30 and 50mm. Taking into account the tilt of the structure, the likely maximum primary consolidation for the Northern end of the structure might be 60mm, with an average of say 50mm.
Assessment of Young's Modulus Appropriate for the Model.

The above value for primary consolidation was then used to determine the effect on the modulus values used in the consultants two layer elastic system.

The stiffness used in the model to analyze the structure were as follows:

\[
\begin{align*}
E_1 & \ (0 \ to \ 30\text{m}) \ = \ 25\text{MN/m}^2 \\
E_2 & \ (30 \ to \ 300\text{m}) \ = \ 100\text{MN/m}^2 
\end{align*}
\]

The modulus suggested by a settlement of 50mm in the model would be 90MN/m². Due to the ratioing effect shown in the above simple exercise this also implies a value for \( E_2 \) of 360MN/m².

Table 2.9/6 shows the values of \( E_1 \) and \( E_2 \) required, in the simple analysis, to produce the same settlement.

The model used to analyze the structure shows the same dependency upon the ratio of \( E_1 \) and \( E_2 \). Thus it is implied from the above table that the soil modulus is likely to be between 80 and 116MN/m² dependent upon how the stress distribution and hence settlement distribution occurs with depth. This value compares well with the values of modulus indicated in Table 2.1/10 and Figure 2.1/13 which would indicate the following values,

a) \( 100\text{MN/m}^2 \) for Butler's relationship \( 400C_u \ (1974) \)
b) \( 95\text{MN/m}^2 \) for lower bound value \( E_{u0.1} \) (Jardine et al 1986)
c) \( 117\text{MN/m}^2 \) for upper bound value \( E_{u0.1} \) (Jardine et al 1986)

The empirical relationship of \( 220C_u \) normally associated with design in London Clay suggests a lower value of modulus \( 56\text{MN/m}^2 \).
Table 2.9/1 Analysis of Bins 7 to 15 in the Wheat Silo

<table>
<thead>
<tr>
<th>Bin Numbers</th>
<th>13-15</th>
<th>10-15</th>
<th>7-15</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load (k)</td>
<td>19022</td>
<td>30048</td>
<td>41074</td>
</tr>
<tr>
<td>Live Load (k)</td>
<td>20137</td>
<td>36154</td>
<td>49185</td>
</tr>
<tr>
<td>Average Settlement (mm)</td>
<td>6.15</td>
<td>5.58</td>
<td>5.00</td>
</tr>
<tr>
<td>Dead Load Estimate (mm)</td>
<td>5.81</td>
<td>4.64</td>
<td>4.00</td>
</tr>
</tbody>
</table>

Live Load was taken as the Recorded Bin Loading + Incidental Live Wind and Slab Loads.

The estimate of settlement due to the Dead load application was the settlement due to the live load ratioed by the dead load to the live load.

Table 2.9/2 Parameters Used in the Calculation of Elastic Shortening of the Piles under the Wheat Silo

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Bin Loading</td>
<td>68837kN</td>
</tr>
<tr>
<td>Assessment of Incidental Live Loads</td>
<td>2976kN</td>
</tr>
<tr>
<td>(Wind and Slab Live Loads)</td>
<td></td>
</tr>
<tr>
<td>Dead Load of the Structure</td>
<td>60096kN</td>
</tr>
<tr>
<td>Total Load</td>
<td>131909kN</td>
</tr>
<tr>
<td>Number of Piles under Slab</td>
<td>190</td>
</tr>
<tr>
<td>Diameter of the Piles</td>
<td>533mm</td>
</tr>
</tbody>
</table>
### Table 2.9/3 Average Settlement along the Length of the Wheat Silo for the 27/1/1983

<table>
<thead>
<tr>
<th>Long Section</th>
<th>Average Settlement (mm)</th>
<th>Difference Between Sections (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Station M6, M5</td>
<td>4.30</td>
<td>0.93</td>
</tr>
<tr>
<td>Station M13, M18, M26</td>
<td>5.23</td>
<td>1.47</td>
</tr>
<tr>
<td>Station M17</td>
<td>6.70</td>
<td>0.43</td>
</tr>
<tr>
<td>Station M14, M16, M25</td>
<td>7.13</td>
<td>1.00</td>
</tr>
<tr>
<td>Station M15, M19, M24</td>
<td>8.13</td>
<td>0.67</td>
</tr>
<tr>
<td>Station M20</td>
<td>8.80</td>
<td>0.77</td>
</tr>
<tr>
<td>Station M22, M21, M23</td>
<td>9.57</td>
<td></td>
</tr>
</tbody>
</table>

### Table 2.9/4 Adjustment of the Measured Wheat Silo Settlements to Allow for a Continual Full Live Load

<table>
<thead>
<tr>
<th>Date</th>
<th>Load (tonne)</th>
<th>Measured Settlement (mm)</th>
<th>Adjusted Settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>27/1/83</td>
<td>524.7</td>
<td>11.5</td>
<td>11.5</td>
</tr>
<tr>
<td>3/3</td>
<td>264.8</td>
<td>11.5</td>
<td>13.2</td>
</tr>
<tr>
<td>12/5</td>
<td>101.3</td>
<td>11.6</td>
<td>14.4</td>
</tr>
<tr>
<td>28/6</td>
<td>254.2</td>
<td>11.3</td>
<td>13.1</td>
</tr>
<tr>
<td>4/10</td>
<td>417.9</td>
<td>12.1</td>
<td>12.8</td>
</tr>
<tr>
<td>17/1/84</td>
<td>340.1</td>
<td>12.9</td>
<td>14.1</td>
</tr>
<tr>
<td>3/4</td>
<td>386.6</td>
<td>17.1</td>
<td>18.0</td>
</tr>
<tr>
<td>5/6</td>
<td>352.2</td>
<td>12.9</td>
<td>14.2</td>
</tr>
<tr>
<td>10/10</td>
<td>315.9</td>
<td>16.0</td>
<td>17.4</td>
</tr>
<tr>
<td>30/1/85</td>
<td>390.3</td>
<td>16.8</td>
<td>17.7</td>
</tr>
</tbody>
</table>
Table 2.9/5  Estimated Consolidation Settlement from Estimates for $d_{50}$ and $d_{70}$

<table>
<thead>
<tr>
<th>Station</th>
<th>Settlement</th>
<th>$d_{50}$</th>
<th>$d_{100}$</th>
<th>$d_{70}$</th>
<th>$d_{100}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M16</td>
<td></td>
<td>12</td>
<td>35</td>
<td>15</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>40</td>
<td>17</td>
<td>34</td>
</tr>
<tr>
<td>M17</td>
<td></td>
<td>13</td>
<td>36</td>
<td>15</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>40</td>
<td>17</td>
<td>40</td>
</tr>
<tr>
<td>M19</td>
<td></td>
<td>18</td>
<td>46</td>
<td>21</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>44</td>
<td>22</td>
<td>41</td>
</tr>
<tr>
<td>M20</td>
<td></td>
<td>22</td>
<td>54</td>
<td>24</td>
<td>44</td>
</tr>
</tbody>
</table>

Table 2.9/6  Ratios of $E_1$ and $E_2$ to be used in the Consultant's Two Layer Analysis to give a Settlement Prediction of 50mm

<table>
<thead>
<tr>
<th>$E_1$ (0 to 30m)</th>
<th>$E_2$ (30 to 300m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(MN/m²)</td>
<td>(MN/m²)</td>
</tr>
<tr>
<td>90</td>
<td>360</td>
</tr>
<tr>
<td>125</td>
<td>100</td>
</tr>
<tr>
<td>116</td>
<td>116</td>
</tr>
<tr>
<td>80 rigid base at 30m</td>
<td>360</td>
</tr>
</tbody>
</table>
Log Time from Completion of Construction (days)

Figure 2.9/1 Log Time against Settlement for Station M16 Showing

Settlement (*) and Settlement Corrected for a Full Live Load (+)
Figure 2.9/2  Log Time against Settlement for Station M17 Showing

Settlement (x) and Settlement Corrected for a Full Live Load (+)
Figure 2.9/3 Log Time against Settlement for Station M19 Showing

Settlement (*) and Settlement Corrected for a Full Live Load (+)
2.10 Discussion.

The results of the initial site investigation highlight the difficulties that arise for a design engineer in specifying the parameters that will be required for a specific foundation design at a specific location.

The report described the work carried out and discussed the findings with a view to founding 100tonne piles and the likely settlement of the fill yet the investigation did not carry out the necessary sampling. Subsequently two of the holes were redrilled by rotary methods to indicate the thickness of the limestone but still no adequate sampling was undertaken for the equally critical founding stratum, the Lias Clay.

When time and money are of the essence it may be difficult to justify the expense of an all embracing site investigation when a less intensive and cheaper investigation would have proved sufficient. The need for liaison between the design engineer and the site investigation firm is evident particularly if the design engineer does not have the required geotechnical expertise. In such a case the appointment of the site investigation firm should take on an added dimension. Not only should the firm carry out the work for which they have tendered, but during the desk study and site work they should also update the design engineers' information on the ground conditions in line with the current design concept. Equally the design engineer should update the site investigation firm's brief should changes in the foundation design occur. This two way process should then enable sensible decision making on any additional works necessary to produce the appropriate parameters for that design. It was highlighted by Burland Broms and de Mello (1977), in their state-of-the-art report, that the nature of the ground to be investigated should, in nearly all cases, be well documented. Thus, at the desk study stage, the general parameters for the strata to be investigated should be
known. The purpose of the ground investigation and subsequent laboratory testing should then be to refine those parameters or highlight differences from those generally expected. The design can then be altered to suit having already demonstrated to the design engineer the likely ground conditions and parameters that will affect his proposed design.

It is evident that this process did not take place until the report was submitted. At this stage the report indicated that should piles be required, analysis of the Lias Clay for bearing capacity and settlement characteristics should be undertaken.

This work was outside the site investigation firm's brief but its importance should have been realised at the desk study stage, and if not then, as soon as the thickness of the limestone was proved marginal. The problem should have been highlighted to enable a sensible decision to be made to avoid the expense of an additional investigation. It became evident, when a geotechnical consultant was engaged to advise on the likely settlements, that the sampling was not sufficient for either the Limestone or the Lias Clay. Additionally the depth of the boreholes needed to be increased to check the parameters of the Lias Clay in the region of significant stress caused by the proposed foundation's stress bowl.

On completion of the additional investigation it was shown that the sampling techniques employed in the initial investigation were inadequate. The results of the initial routine investigation show that light percussion boring cannot estimate the thickness of thin rock layers with adequate accuracy. Furthermore coring with air flush and double tube swivel-type corebarrels failed to provide satisfactory estimates of the undrained shear strength of a very stiff to hard clay. The use of bentonite mud flush, mylar linings and large diameter corebarrels in the second investigation was sufficient to produce very good quality samples for laboratory testing.
A time delay in the contract was not possible, thus the additional sampling and resultant analysis were undertaken during the piling contract. It became evident that it was necessary to undertake more than usual pile load testing as the additional investigation was unable to confirm the pile design. Even though the piling contractor was confident of his system based on similar ground conditions, for the reasons given in section 2.2, the design analysis showed that the edge piles were at a stress level close to that which would cause local yielding. It is known that the corner pile stresses are higher than those at the edge (for example Whitaker 1957 and 1960). It would, from the above, seem reasonable to assume that the piles in the corner of the structure would be loaded in excess of that caused local yielding in the Lias Clay. However in was felt that local yielding of the piles into the top of the Limestone might reduce bending moments theoretically applied by the founding strata to the structure providing the limestone was thick enough to avoid punching failure.

The results from the pile test indicated that;

1) pile heave was a problem and that action had to be taken to retap all the piles. The piling contractor, with his local knowledge, had confidently predicted that pile heave would not be a problem. There would have been serious consequences had he also been wrong about the adequacy of the rock thickness, the unconfined compressive strength method of predicting end bearing resistance being conservative, clays beneath limestones being very stiff and negative skin friction being grossly overestimated by geotechnical engineers.

2) the piles could carry 1.5 times their capacity satisfactorily. Maximum settlements under 165 tonne ranged from 5.5 to 8.0mm and residual settlements after unloading between 0.4 and 1.7mm.
3) due to uncertainties regarding shedding load by a relatively rigid structure to the outer piles of the group, three piles were tested to 2.5 times their nominal capacity. These piles behaved well with maximum settlements under 250 tonne ranging from 8.8 to 10.8mm and residual settlements after unloading between 1.4 and 1.7mm. 

Two of the three pile load tests to 2.5 times working load showed a slight acceleration of settlements between loads of 235 and 250 tonne perhaps indicating pile failure was not far off. If this is the case the failure toe stress would be just in excess of 8.1MN/m² indicating the rock sustained stress estimated at 3 times its uniaxial unconfined compressive strength (Clayton et al 1984). The rock mass was by no means intact and yet the piles behaved well under 250 tonne which confirmed the validity of the pile design where the laboratory tests could not.

The pile construction process showed that pile heave will be a serious problem when closely spaced piles are used to support structures sensitive to differential settlement. In this case the fill is reasonably homogeneous and penetration into a high strength stratum did not occur. Despite this, and the fact that the volumetric displacement ratio was of the order of 6 to 10 times the critical limit suggested by Brierley and Thompson (1972), the observed pile heaves were very much less than would be predicted by Hagerty & Peck's approach (1971). On the other hand Cole's method (1972) predicts pile heave of between 40 and 50mm for pile No. 261 (Fig.2.2/5). The predicted range is dependent upon the driving sequence of pile 261 in relation to the surrounding piles. The range of 40 to 50mm for settlement predicted by Cole's method is in good agreement with the observed initial settlement in Fig.2.1/5 of 45mm before the bearing capacity improved during the pile load test. For more closely spaced pile groups beneath the wheat silo, heaves of the order of 65 to 105mm are estimated by this method. During the re-driving of the piles to a
set levelling indicated that 82% of the piles settled less than 65mm, the lower bound indicated by Cole's method and 96% of the piles settled less than 105mm. This overprediction of heave maybe due to the pre-boring to 6m which had removed some of the soil to be potentially displaced by the pile.

Partial pre-boring was not enough to prevent pile heave. In hindsight it was felt that pre-boring should have taken place to within 0.5m 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. It is equally clear that if pre-boring is not carried out the re-driving may give rise to a second phase of pile heave. Therefore a combination of pre-boring, monitoring and re-driving is essential when closely spaced piles are to be driven through clay if multi-tube systems are not to be used.

The first pile load tests indicated shaft adhesion contributing about 65 tonne to the pile capacity, at a settlement of 2 to 3mm, about 0.5 to 1% of the pile diameter. These figures are in accordance with full mobilisation of the undrained shear strength of the made ground, at displacements common with those observed elsewhere (Whitaker & Cooke (1966)).

The observation also brought into question the validity of the applied load in the pile test being applied to the rockhead. A pile load test to 150 tonne might only apply 85 tonne at rockhead with 65 tonne being taken by the shaft. However if the pile had already settled much more than the 2 to 3mm needed to fully mobilise the skin friction the load from the pile test could be fully transmitted to rockhead. It was considered, however, that the pile load tests to 2.5 times nominal working load had stressed the rock to nearly twice their working load if all the shaft adhesion had been mobilised. When the piles are working they will also attract negative skin friction, however,
consideration of the volume of the soil between the piles in the group, and dependent upon assumptions made this might add 15 to 30 tonne per pile in their working condition. Reddaway and Elson (1982) on instrumented piles showed that load tests carried out on the short term showed markedly different load distribution along the pile shaft from that in service and questioned the usefulness. They suggest that load tests on a firm stratum beneath loose and soft alluvium overburden should be interpreted cautiously and that ideally the piles should be sleeved. The latter recommendation concurs with the use of the multitube system to overcome pile heave (Healy and Wheltman 1980, Thorburn and Thorburn 1977 and Cole 1972).

The total settlements to date, including an allowance of 4.8mm for settlement under dead load during construction, are shown in Fig.2.10/1. Even though the back calculation was carried out for the wheat silo, which is a more heavily loaded area than most of the complex, it was considered as a fair assessment of the dead load settlement for the whole complex. This is justified on two counts. First the levels taken by the site staff showed no marked difference in inter-slab differential settlement. Indeed the levels ranged, without any apparent trends, between +5 and -5mm across the whole complex. Secondly the complex interconnections showed no signs of distress at any location.

The dead load allowance was based on the settlement in the wheat silo due to a live load influence. Obviously this could not take into account any creep effects or fully take into account any primary consolidation effects as the influence was measured over a short time period as the bins were filled to near capacity before the plant became fully operational. It was not possible to leave the bins fully loaded to take these effects into account as the client obviously wanted the plant fully operational once the levelling indicated no detrimental effects on the wheat silo. However consideration of how the dead load and live load were applied led to the assumption that
the effects of the time periods the loads were applied balanced the
effects out. The live load was applied rapidly where as the dead load
was applied relatively slowly as construction progressed which would
allow some consolidation to take place. At worst, taking the above
into account, it was thought the dead load assessment might be up to
50% in error which would alter the total settlement values by up to
2.4mm. Overall it was considered the best method of assessing the
dead load settlement due to the range and unreliability of the site
staffs levelling results.

The progression of the measured settlement, including the dead load
allowance, at a number of timed intervals throughout the monitoring
exercise are shown for various sections in Figs.2.10/2 to 2.10/10.
These should be studied in conjunction with Fig.2.5/1, showing the
location of the BRE settlement stations, to appreciable the sections
referred to. The five time intervals plotted are;

1) Dec 82 about 1 month after plant installation and just
   prior to filling the wheat silo bins,

2) late December 1982 whilst the bins were being filled,

3) late January 1983 when the bins were close to maximum
   capacity prior to the plant becoming operational,

4) May 1984 500 days into monitoring,

5) January 1985 800 days into monitoring,

Figs.2.10/3 to 2.10/5 show the settlement profiles for the above five
time periods for three longitudinal sections along the wheat silo. It
would appear that for the first two time periods settlement along the
length of the slab was uniform. In all three sections for the second
time period, late January 1983, the uniform settlement was about 9mm.
Once the bins were full and the plant became operational all three
sections showed different differential settlements away from the
Elevator and Pre Clean Area. Fig.2.10/5 shows the situation for the three sections at January 1985. Based upon oedometer and undrained triaxial values of Young's Modulus and the stiffness of the slab above the design predicted a maximum settlement on the longitudinal centre line (Table 2.3/1) of 194mm with a differential settlement from the edge to the centre of the slab of 34mm. These figures can be directly compared with the situation to date (January 1985) plotted in Figure 2.10/3 which shows a maximum settlement of 21mm and a differential settlement from one end of the slab to the other of about 10mm. The drained reload modulus value reduces the predicted value of total settlement to about 70mm, but the triaxial method is at strain levels higher than the 0.1% strain of the insitu soil under load (Jardine et al 1986 and Poulos 1989). Jardine et al (1984) point out that conventional external measurement of displacement, such as those employed in the site investigations, contain errors which are frequently so large that the use in determining the soil stiffness at working levels is invalid. Most triaxial tests give apparent soil stiffnesses far lower than those inferred from field behaviour (Jardine et al 1984) and the use of non-linearity suggests that the inferred field value is also low (Jardine et al 1986).

Additionally Figs.2.10/6 to 2.10/9 show the progression of four transverse wheat silo slab sections for the same five time periods. Again the uniformity of the settlement is apparent prior to the bins being filled. The situation in January 1985 is shown in Fig.2.10/10 where the transverse section through the Elevator and Pre-clean area has been added. The maximum differential settlement measured to date is approximately 2mm from edge to edge of the slab. As above the maximum total settlement is 21mm.

Looking at Fig.2.10/10 in closer detail the following can be seen:
1) The section through M26, M18, M13 shows a slight sagging mode with a slight tilt downwards in a westerly direction.

2) The sections through M25, M16, M14 and M24, M19, M15 show no signs of a hogging or sagging tendency, but shows a slight tilt downwards in an easterly direction.

3) The section through M23, M21, M22 shows a slight hogging tendency, indeed this tendency appears at present to be decreasing. Again the slab shows a slight tilt downwards in an easterly direction.

The conclusion of these observations is that the slab has been put into torsion along its length.

Table 2.3/1 shows the total and differential settlements for the range of calculations done, primarily altering the stiffness of the structure.

The lack of any apparent bending in the slab in any of the slab sections would suggest that the stiffness of the structure used in the main analysis was extremely conservative.

Unfortunately, due to the time available, analysis for the long centre line for the wheat silo was restricted to one run using a value of 8.87x10^6 kNm^2/m run which is the stiffness of the slab only. Limited analysis was undertaken for the transverse centre line using a range of stiffness to an upper bound suggested by Meyerhof's paper (1957). These were only undertaken with a soil modulus of 25 MN/m^2, but, taking into account the one run done with a soil modulus of 35 MN/m^2 with a slab stiffness of 8.87x10^6 KNm^2/m run, Fig.2.4/8 might suggest a value of 40x10^6 kNm^2/m run due to the magnitude of bending apparent on one section in Fig.2.10/10.
The magnitude of the total settlement measured to date, 21mm, would suggest that the value of modulus used in the analysis was extremely conservative. The one run undertaken with a value of 35 MN/m² (as discussed in section 2.5 suggested by the oedometer testing), did substantially reduce the total predicted settlement from 180mm to 137mm for the short centreline analysis (Table 2.3/1). This figure is still much larger than the 21mm to date or the predicted 40-50mm for the worst estimate of primary consolidation and suggests that the use of the average value of the drained reload triaxial tests, 65 MN/m², would still be a low value as the total settlement predicted from this value is of the order of 70mm. The back analyzed value of modulus of 90MN/m², which gave the right order of settlement for the method of analysis, may also be low if non-linearity is taken into account (Jardine et al 1986).

The tilting away from the Elevator and Preclean area once the bins were loaded is thought to be a combination of two factors. Firstly the obvious cause is due to the uneven live loading of the wheat silo with virtually no live load in the elevator and pre clean area. Secondly if the stiff slab does redistribute the stresses, such that the elevator and pre clean area does take a substantial amount of the live load, the elastic shortening in the shorter piles in this area is about 1mm less for a similar pile load.

The foundation design for the wheat silo obviously had its limitations. It did not take into account the loaded area adjacent to the structure nor could it take into account the deeper foundation in the elevator and pre-clean area of the slab. The latter is particularly highlighted by the apparent uniform settlement under a relatively uniform dead load during the initial, dead load only monitoring. Once the uneven live load was applied in the bins the slab tilted away from the elevator and pre clean area as discussed above. It is of interest to note that the difference in elastic shortening predicted in section 2.9.3 is of the same order as the
difference in average level of M6, M18, M13 situated over a certain wall on the shorter piles and M17, the first station over the longer pile. The small amount of torsion observed along the length of the slab is assumed to be a result of the adjacent structural slabs on the eastern edge of the wheat silo and the deeper foundation of the elevator and pre clean area, neither of which were taken into account during the modelling for settlement predictions.

The predictions made from the design analysis must be looked at in the context of the design parameters. The values for the stiffness of the structure and Young's Modulus for the soil, discussed above, directly affect the predictions for total and differential settlements. The problem still remains for the design engineer to obtain a realistic value for the overall stiffness of the structure. In choosing too high a value, showing just acceptable differential settlement predictions, the complicated mechanisms involved in such a structure as the one discussed may result in failures of crucial areas resulting in the dramatic loss of overall stiffness. In this case the assessment was known to be conservative and it was accepted that even if the remainder of the structure apart from the slab contributed no part to the stiffness the differential settlement predicted was 7.7mm against the 5mm required. Added to this the limited analysis increasing the stiffness shown in Fig.2.4/8 and Table 2.4/1 gave the added confidence that even a slightly greater actual overall stiffness would reduce the differential settlement to within the design requirement of 5mm, without significantly altering the maximum bending moment to be taken by the structure, Fig.2.4/9. The design hinged on the ability of the limestone stratum to transmit the load through to the Lias Clay. When the decisions were made the confidence to carry the piling contract out was based on the limited knowledge obtained from the initial investigation and the confidence of the piling contractor using this system in similar local conditions as discussed in section 2.3. This was particularly relevant in that he stated the unconfined compressive strength method of predicting the safe end
bearing resistance of piles is grossly conservative. This proved to be the case as it was only on the completion of the pile load tests that the limestone bearing stratum was shown to be of sufficient quality and thickness to take the design load. The results of the additional investigation, carried out during the piling contract, had been unable to confirm the design assumption, and had thus recommended additional pile tests. The value of Young's modulus, used in the design, for the Lias clay proved to be very low and thus the value of total settlement predicted was large. This was again due to the time available for testing. With more time to complete more drained reload tests a more realistic value might have been used with confidence. Tests undertaken today using local strain measurement and computer controlled testing, with the good quality core, would have produced very good soil stiffness values. Instead the upper and lower bounds suggested from the oedometer results were used. However, it was pointed out to the design engineer that in the light of the drained reload testing, and with engineering knowledge of the Lias Clay limited analysis had shown the value of total settlement to be about 70mm (a third of that predicted using the value obtained from the oedometer's value for E) but that the design was not carried out using the value from the Drained Reload test due to the uncertainty of making predictions based on two results. The design predictions for total and differential settlement were thus carried out using two very conservative parameters and the maximum design live load. In the case of a grain silo these are very real large live loads and not some theoretically possibly value as in the case of say an office block. Even so the situation of maximum live load is unlikely to occur during the operational life of the structure, nor could it be termed as a udl over the whole slab, the basis of the design. The back-calculated dead load settlement calculation was carried out from an early period in the structures history, when, at the instance of the design engineer, the live load in the bins was brought up to as near maximum capacity as was practically feasible. Once no detrimental effects were evident the plant became operational. It is at this one period,
when the bins were as equally loaded as possible, that the structure has had the influence of what could be termed as a udl acting upon it. Since this period the bins have been by no means equally loaded, Tables 2.7/3 and 4. However it must also be noted that even though the live loading has been unequal (and in many instances reduced from the previous reading) the structure as a whole has continued to settle Figs.2.10/11 to 2.10/13.

In the unequal loading condition, mentioned above, a similar problem exists in assessing the load sharing ability of the bin floor, as occurs in assessing the load sharing capabilities of the foundation slab. With a very stiff structure, and the bin floor supported on walls and columns, how does the load sharing take place down the bin walls to the bin floor slab? Again load sharing is to be expected at the bin floor with these loads being transmitted through the walls and columns to the foundation slab where again a redistribution of loads is to be expected to the piles. The major problem in analyzing the unequal loading condition and its effects would be the determination and understanding of these mechanisms.

The complications involved with these mechanisms are highlighted pictorially in Figs.2.10/11, 2.10/12 and 2.10/13 where settlement and column load are plotted against time. It can be quite readily seen that the magnitude of the settlement bears no relationship to the magnitude of the assumed live load down the column. The column load was assumed to be the sum of the quarter of the load in each of the bins the column supports (Fig.2.7/1 and Table 2.9/1). Thus M20 (Fig.2.10/11), an internal column, under maximum live load will carry twice the load of the external edge column M21 (Fig.2.10/12) and four times the load of the corner column M22 (Fig.2.10/13).

From this simple statement it can be seen that a great deal of redistribution of stress must have to be undergone at bin floor level and slab level for the reported ratios of pile load to apply (Whitaker...
Cooke (84)). For these ratios to apply a complete reversal of loading
must be undertaken as an internal pile takes half the load of an edge
pile and a quarter the load of a corner pile. Due to this quite
substantial load redistribution within the very stiff slab, a
substantial load change over a column will depend on the loading state
adjacent columns for the overall slab settlement trend to be reflected
in the settlement change measured at that column. It is quite
possible for the column position to show a decrease in settlement even
though the column appears to have an increased load from the vast
settlement reading.

Of paramount importance, when assessing a grade A prediction (Lambe
1973), is to determine whether the method of analysis and the
selection of appropriate parameters has predicted correctly the
required results. In this case the most crucial parameter to assess
is the value of primary consolidation and its attending differential
settlement. An assessment of how long the primary consolidation will
take, if it had not already taken place, had to be made. Plots of
settlement against Log(Time) (Figs.2.9/1 to 2.9/4) would lead one to
believe that primary consolidation is still taking place.

To assess how long this process is likely to occur a simplistic
approach was adapted. Both Rowe (1972) and Lambe (1973) in their
consecutive Rankine Lectures gave case records showing that laboratory
values of \( c_v \) determined from small samples drained vertically in the
oedometer, may well be factors of hundreds different from the back
calculated values assessing a structures actual behaviour due to
fabric and fissuring facilitating lateral drainage. However, with the
benefit of detailed borehole logs of the second investigation an
assessment of these effects could be made. The logging indicated that
the Lias Clay was an intact material, retrieval from the corebarrel
was in long lengths. The core was split randomly in many positions
and at no point was there any visible indication of fabric or
fissuring to indicate that lateral drainage in the Lias might be substantially greater than vertical drainage. On a micro scale the clay platelets show preferred horizontal layering which would allow only a marginally preferred passage to water horizontally.

The logging also indicated comparatively thin horizons of silty sandstone. On a visual assessment and because of their thickness, these were not considered to be preferred drainage paths. It was thus assumed that the laboratory determined values of $c_v$ would be representative both horizontally and vertically.

A simplified assessment was made for $t_{90}$ from Terzaghi's theory of one dimensional consolidation and the chart after Janbu, Bjerrum and Kjaernsti (1956) but with the assumption of lateral drainage. A complicated 3-D finite analysis assessment after say Christian (1977) was not thought appropriate for a number of reasons.

a) It is a major undertaking and was considered outside the scope of this report.

b) There may well be problems in applying parameters which were not determined with this exercise in mind.

c) The uncertainties that are evident concerning the appropriate loading regime of the structure.

d) As long as the shortcomings of the simplistic method are realised an indication of the worst case is all that is required.

Using the mean $c_v$ value from the 11 oedometer tests, undertaken for the second investigation, values of $t_{90}$ range from 71 to 85 years dependent upon which $r_p$ ratio is considered appropriate. The value of
t_{90} on the maximum and minimum values of c_{v} taking the r_{p} ratio appropriate give a maximum range of 40 to 200 years.

It can be seen from the above that although the method may be inaccurate all the indications are that the primary consolidation is only 50 to 70% complete.

To assess the likely primary consolidation use was made of the Log(Time) v Settlement plots Figs.2.9/1 to 2.9/4. To make an assessment of the settlement due to full live load a simple correction was made assuming that the Lias Clay shows an elastic response and that full strain recovery occurs in the system. It was also hoped that this might smooth the plots out.

The results of these calculations are shown as the solid line on Figs.2.9/1 to 2.9/4. Calculating the time for 50 and 70% of primary consolidation it can be seen from Figs.2.9/1 to 2.9/4 that the predicted bound of settlements can be made for 50 and 70% settlement. These may then be ratioed up to give an indication of the total primary consolidation. From this, the likely range of full primary settlement predicted from Figs.2.9/1 to 2.9/4 is between 30 and 55mm. Overall, considering the structure is tilting, a maximum possible primary settlement of about 60mm might be predicted for the North End of the structure, with an average predicted settlement of say 50mm.

This being the case the value of 70mm predicted from the average value of the drained reload modulus (65MN/m^2) might be considered as conservative. The modulus suggested from a maximum settlement of about 50mm would be about 90MN/m^2 for use in the computer model.

The difficulty in assessing an average value of modulus from the two drained reload tests is evident when the results are so different (44 and 82MN/m^2). It may be that these are representative of an increasing modulus with depth. On the other hand either result may be
spuriously high or low. The relevance of this test may also be questioned as the specimens may be consolidated, but it has basis on empirical grounds in that it was generally accepted to give values of the right order where as those from the oedometer do not. Consideration of the strain level under the foundation suggests that the value from the drained reload test undertaken in the triaxial cell is not appropriate (Jardine et al 1984, Jardine et al 1986). Provided that good quality undisturbed samples are tested, local strain measurement provides an E value for the appropriate strain level.

The model made the following assumption for Young's Modulus; 0-30m 25MN/m² (E₁) and 30-300m 100MN/m² (E₂). The object of inputting the second modulus as a larger value was to model the soil as being very much stiffer outside the region of significant stress. To study the affect of changing these values a simple elastic exercise was undertaken. This analysis suggests, as expected, that the values of E₁ and E₂ combined in a fixed ratio are directly proportional to settlement. Thus changing the E₁ value to the suggested 90MN/m² also implies that E₂ should be increased in the same ratio, that is E₂ increases from 100MN/m² to 360MN/m².

As the model, by which the structures predicted settlements were determined, also shows this settlement dependency on E this ratioing affect also applies.

Thus the suggested value of modulus from the back analysis may not be representative of the actual modulus of the soil. The above simple analogy would suggest that the actual soil modulus is likely to be between 80 and 116MN/m² dependent upon how the stress distribution and hence settlement distribution actually occurs with depth. Thus, in the model used, if a value of modulus determined in the laboratory is deemed representative from 0-30m, the appropriate value from 30-300m needs also to be representative for the model to predict accurately. Thus applying a larger number arbitrarily to represent the soil as
being stiffer will give an incorrect prediction unless it is the correct ratio to the modulus value used to represent the stiffness at 0-30m, even though the modulus assigned to $E_2$ is actually that of the soil.

With the indications showing that primary consolidation is indeed going to take a very long time and if the future analysis indicates this to be so it might have interesting design implications for so-called temporary structures. If a structure is to be constructed for a short term need and that structure is critical for total and differential settlement then the design could be shown to take into account only part of the primary consolidation. Hence this record might enable short term structures to be designed more economically in the locality by taking this into account.

The design was based on the foundation being piles at 1.5m centres, thus producing a udl on the Lias clay once the end bearing load of the piles had been transmitted through the limestone. In reality this was not the case as can be seen in Fig.2.2/1. The pile spacing was originally at 1.5m centres only under the column locations, with a general spacing of 2.2m between the groups. The situation was further changed after the foundation design analysis in that three piles were added to each external column location. This was undertaken in an effort to reduce and redistribute the high edge and corner stresses predicted by the analysis. As time was not available further analysis was not possible. The analysis would have been able to model unequal pile spacing by adjusting the spring location so had the correct pile spacings been known it would have been possible to model a section through the slab centreline using the appropriate pile spacing.

The net effect of the above discussion on the design prediction, shown in Figs.2.10/5 to 2.10/10 and Table 2.3/1, for settlements for the wheat silo were;
1) the longitudinal deflection profile shows tilting away from the elevator and pre-clean area. Maximum settlement is 21mm with 10mm differential settlement from one end of the slab to the other. The design analysis predicted the slab to be in a sagging mode with a maximum settlement at the centre of the slab section of 194.3mm and a differential settlement, from the centre to the edge of the slab of 6.8mm. EI for the structure 8.86KNm²/m run, Young's modulus for the Lias Clay 25MN/m².

2) the transverse deflection profiles show a small amount of tilting in both directions, i.e. torsion. This is shown in the profiles in Fig.2.10/10. Working from the elevator and pre-clean area, the profile through M13, M18 and M26 displays a sagging mode tilting down to the west with a differential settlement of 2mm from one side to the other. Two sections through M14, M16, M15 and M15, M19, M24 are both flat but tilt down in an easterly direction with a differential settlement of 2mm edge to edge. The maximum settlement, shown in the last section, for any of the sections is 21mm. For the most conservative analysis undertaken for the wheat silo short centre line the design predicted the slab should be in a sagging mode with a maximum settlement of 180mm at the centre of the section and a differential settlement of 7.7mm from centre to edge using EI for the structure 8.86KNm²/m run, Young's Modulus for the Lias Clay 25MN/m².

Obviously, as already discussed, the predictions for total and differential settlement could have been closer had the appropriate values of overall stiffness of the structure and Young's Modulus for the Lias Clay been available. The analysis would not however have been able to predict the magnitude and direction of the resultant tilting as it is unable to model the adjacent structures.
The principal of the design appears to have worked well in this instance. The limitations of the programme were realised and the geotechnical expertise was available to make sound recommendations on the total settlement in the light of the analysis predictions as the shortcomings of the laboratory determinations for Young's Modulus were realised. The design used the safest value for the total stiffness of the structure which produced values of differential settlement marginally greater than those specified. However it was realised that the stiffness was greater than the value used and limited analysis was undertaken which showed even a small increase in total stiffness would bring the values of differential settlement to within acceptable limits.

The areas of concern to the design engineer when using this simple form of analysis must be;

1) whether the pile layout is uniform enough to be modelled by this method. The section considered must be representative of a common section through the slab. The piles need not be evenly spaced as the springs can be positioned in the model at the pile spacings.

2) whether any surrounding structures will seriously influence the behaviour of the structure to be modelled.

In this instance the surrounding structures are lightly loaded and appear to have caused the small amount of tilt. This may mask any effect which might have been detected due to the actual pile spacing and the inclusion of the additional piles. Had the surrounding structures been as heavily loaded and or the pile spacing been highly irregular, a much more comprehensive, and certainly more expensive, 3D analysis would have been required.
The areas of concern to the design engineer when using any form of analysis must be;

1) whether the stiffness assigned to the structure is realistic.

2) whether the modulus of the soil is realistic.

In both instances, as discussed above, the values were conservative, but more importantly were known to be conservative. The analysis was known to be predicting the worst possible case as it was realised that none of the parameters were overvalued. The major area of concern in the process of events was that it required the pile load tests to verify the suitability of the limestone after completion of the piling contract. In the normal run of events, if the site investigation left doubts about the bearing capacity of the stratum, a series of experimental piles should have been installed to verify this before the design construction stage. Had the pile load tests failed this would have resulted in major design changes and probably a major delay in the completion of the contract.

Finally the model predicted a sagging mode of deflection, it is noted in Fig.2.10/10 that the slab shows a slight sagging mode at the North End of the wheat silo. The differential settlement in this part of the slab is small at present. It will be of interest to note in the future if this influence will be maintained as settlement increases. The implications of this are that the positioning of the tensile reinforcement in the slab is dependent upon the mode deflection predicted. Thus this may be a serious flaw in the model if it is predicting a sagging mode when the structure displays a hogging mode. It would appear at present that the sagging mode is reducing (Figs.2.10/8 and 2.10/9) but it will only be known when the long term behaviour of the structure is monitored.
At present all the back analyzed parameters have been made on limited data which indicate primary consolidation is not complete. It is only at the completion of primary consolidation that more accurate predictions of the suggested parameters can be made. It is thus essential for the completion of the back analysis that continued monitoring of the structure is maintained such that the full implications of the design model can be realised.
Figure 2.10/1 Plan of the Mill Complex Showing the Settlement at the BRE Stations up to January 1985
Figure 2.10/2 Settlement Profile Across the Length of the Slab for Stations M5, M13, M14, M15 and M22
Figure 2.10/3 Settlement Profile Across the Length of the Slab for Stations M27, M26, M25, M24 and M23
Figure 2.10/4  Settlement Profile Across the Length of the Slab for

Stations M18, M17, M16, M19 and M21
Figure 2.10/5 Settlement Profile Along the Length of the Slab for January 1985
Figure 2.10/6 Settlement Profile Across the Width of the Slab for Stations M26, M18 and M13
Figure 2.10/7 Settlement Profile Across the Width of the Slab for Stations M25, M16 and M14
Wheat Silo Slab Width (m)

Settlement (mm)

December 1982

December 1982 after filling

January 1983

May 1984 (500 days)

January 1985 (800 days)

Figure 2.10/8 Settlement Profile Across the Width of the Slab for Stations M24, M19 and M15
Figure 2.10/9  Settlement Profile Across the Width of the Slab for
Stations M23, M21 and M22
Figure 2.10/10 Settlement Profile Along the Width of the Slab for January 1985
Figure 2.10/11 Settlement (---) and Load (-----) against Time for Station M20
Figure 2.10/12 Settlement (---) and Load (-----) against Time for Station M21
Figure 2.10/13 Settlement (---) and Load (-----) against Time for Station M22
2.11 Conclusions

In the seven years that the mill has been operational the complex has performed well to date. There have been only two instances when the mill personnel have reported any sign of structural distress, both of which could have been avoided with better joint detailing.

The first was the breakdown of a grout fill over a construction joint. The grout crumbled under a "lino" type floor covering revealing a gap. A more flexible grout might have avoided this.

The second was due to the application of sensitive finishes across a construction joint. This has resulted in a very small movement causing fine cracking through a high quality finished floor and upwards through the plaster covering the construction joint in the wall.

The heavily loaded Wheat Silo structure shows no visible signs of distress. It must, however, be realised that all the indications are that the primary consolidation is not yet complete. Thus evaluation of how the design model predicted the actual performance cannot be made accurately until this settlement has occurred. Evaluation of the design model in this report has been made on the forecasts of actual performance based on the apparent measured settlement trends to date. With this in mind the conclusions drawn to date are as follows.

1) The site investigation showed the need for good quality drilling to obtain representative, "undisturbed", good quality core from the very stiff to hard Lias Clay to be used in the laboratory testing. In the second investigation the use of rotary coring using P and S sized double tube swivel type corebarrels, Mylar liners and thick bentonite mud flush proved very successful as total core recoveries of 100% were normal. Insistence that
rotary coring commenced above rockhead also insured good quality core from the limestone and that the limestone thickness at that location was known accurately.

2) The collaboration between the structural engineer and the geotechnical consultant ensured that the parameters for the both structure and ground were looked at in context and in conjunction with one another such that sensible decisions could be made in the light of the overall design.

3) The requirement for the construction piles to prove the design pile toe stress is not desirable. Had the site investigation been planned then the limestone and the Lias Clay could have been sampled to provide the parameters for a pile design. If the designer had still required a toe stress in excess of that indicated by the site investigation, then a series of experimental piles could have been driven and tested to failure making an appropriate allowance for skin friction. From this the required design toe stress could have been verified or an estimation of the safe toe stress could have been made in order to redesign.

4) Heave will be a problem when closely space piles are driven through a soft cohesive material into a rock bed. Partial reboring was not enough to avoid heave. It was considered, that in such ground conditions, it would have been prudent to organise a programme of pre-boring to 0.5m above rockhead, driving, monitoring and redriving. Having undertaken this programme it is essential to re-monitor to ensure the piles are seated into the limestone. Test piles installed to prove that pre-boring is not necessary to limit pile heave should be of the same diameter and at
the closest pile spacing of a representative cluster of working piles.

5) Cole's method of predicting pile heave (Cole 1972) was in good agreement with the pile heave measured for the pile in Fig. 2.2/5. His method for predicting pile heave was in general agreement with the settlements measured during the redriving of the closely spaced piles.

6) If the overall stiffness of the Wheat Silo is back analyzed from the slab differential settlement measured to date a value of $40 \times 10^6$ kNm$^2$/m run is obtained. This will need to be re-analyzed when the load-settlement plot indicates that primary consolidation is substantially complete.

7) From the time-settlement plot, and using the simple assumptions in section 2.9.4. for primary consolidation, a primary consolidation in the order of 50mm is obtained. The tilting evident in the slab would indicate that settlement at the North end of the slab might reach 60mm.

8) For a simple two layer elastic model to predict 50mm settlement, if the ratio of Young's Modulus $E_1$ (0-30m) and $E_2$ (30-300m) is correct, the values of $E_1$ and $E_2$ to use in the model are about 90 and 360MN/m$^2$ respectively. The value for $E_1$ of 90MN/m$^2$ is thought to be representative for the Lias Clay. At a depth below the sphere of influence of the foundation the settlements are much reduced. To reflect this in the model the value of $E_2$ is not chosen to represent the true modulus of the Lias Clay at that depth but to produce more realistic settlement values within the lower layer.
9) Based on the predicted settlement of 50mm the in situ modulus of the soil lies between 80 and 120MN/m² dependent upon the actual stress distribution and hence the settlement distribution with depth. Again it is essential that the prediction is reanalysed when a true reflection of the primary consolidation is available in the future. This prediction is based upon elasticity. The work by Jardine et al (1986) would result in a higher value of modulus calculated from the back analysis carried out using the non-linearity and small strain behaviour.

10) Based on the geotechnical engineer’s experience, and realisation of the limitations of his laboratory determined moduli for the Lias Clay, his prediction of 70mm settlement when analyzing the model’s output data is a realistic one. Thus in realising the parameter limitations, and the effect changes in these parameters have on the model, the model has predicted the behaviour reasonably well. The trends in Figs.2.10/2 to 2.10/5 would suggest that the tilt along the slab from north to south is not increasing at present. If this is shown to be the case in the future there is no need for concern. Again it will only be proved when the primary consolidation is complete.

11) Some concern must be shown at the structures apparent across slab hogging mode evident in Figs.2.10/6 to 2.10/10. This is of only very small proportions and would appear to be decreasing. This may be due to the influences of the surrounding structures settling down; the slab may in the future assume the across slab sagging mode predicted in the model. Here again continued monitoring is needed to check that the hogging mode does not increase. A change in the mode from sagging to
hogging has structural implications for the design engineer to take into account as it means the tensile forces are actually in the top face of the slab whereas the model predicts them to be in the bottom face of the slab.
3 The Alders Building Basildon

3.1 Geology of the Locality

From Sheet 258/9 'Southend and Foulness' and Sherlock (Third Ed. 1962) the geology of significance is made up of those strata of the Eocene period.

Section 2 on sheet 258/9 runs through Basildon and indicates the succession as London Clay of great depth (125-135m) overlying Thanet beds (26-55m).

Thanet Beds

The lowest division of the Eocene, it appears in a narrow outcrop along the margin of the chalk, extending as far west as East Clandon, Surrey. Traced by borings, it is found to terminate towards the north-east along a line drawn through Weybridge, Sunbury, Ealing and Hendon. In Essex it extends underground northward to a point beyond Braintree, but outside this line the Woolwich and Reading Beds overlap the Thanet Beds and rest directly on chalk. The thickness of the formation varies from 0 to 55m. The Thanet Beds consist mainly of fine-grained, pale yellow or grey sand, passing downward into silt, with, at the base, a layer of green loam with green coated flints. The green colour is due to the mineral glauconite.

London Clay

The greater part of the London Clay is a stiff, dark or bluish-grey clay which weathers at outcrop to brown. Characteristic of the London Clay are the septaria, or concretions of argillaceous limestone, occurring as layers of nodules and, in some cases, containing numerous fossils. They are known as cementstones, as they were at one time in great demand for making cement. The lowest part of the formation is a
sandy bed with black flint-pebbles and occasional layers of sandstone and is known as the Basement Bed. It is probable that the oxidation of iron pyrites, present in the bluish clay, resulting in the formation of sulphuric acid which attacks the calcareous shells and nodules and forms crystals of gypsum, selenite, is responsible for the absence of London Clay fossils in most localities.
3.2 Site Investigation

3.2.1 The Investigation

The site investigation, Report No. B436/SJB/vw, was carried out by Ground Explorations Ltd to determine the ground conditions for the foundation design of the buildings and roads associated with the Basildon SE Town Centre Development.

The development consisted of;

1. two buildings of five storeys over a basement,
2. a ten storey office block partly over a basement,
3. a two storey and a five storey building without basements,
4. associated ramps down to basements and a helical ramp up to higher level car parking.

Eleven boreholes were drilled to determine the succession of strata and to observe the ground water conditions. A piezometer was inserted in one borehole and perforated stand pipes in five others for subsequent observations of the ground water levels.

Samples for examination and testing in the laboratory were taken from the boreholes. In ten of the boreholes, the tests were to establish the soil parameters for foundation and basement wall design. In the other borehole, the samples were for determination of electrical resistivity for design of an earth for a proposed electrical generator.
The location of the boreholes with respect to the proposed location of the above mentioned structures is shown in Fig. 3.2/1.

The boreholes were nominally 150mm diameter and were drilled with clay cutter type drilling equipment between 16th October and 4th November 1980.

The piezometer was installed at a depth of 10m in Borehole 3 and standpipes 8m long were placed in Boreholes 5A, 6A, 7A, 8A and 9A. The standpipes were not perforated along the top 1.5m, the upper part being embedded in bentonite grout and the top protected with a stop-cock cover set in concrete.

Whilst the site was planned in terms of the proposed structures the method or methods by which the foundations were to be designed were not outlined by the consultant to the site investigation company. This is evident from the following extracts from the site investigation report where there appeared to be little guidance as to the information required by the design team for any one of the structures.

3.2.2 Findings and Recommendations of the Ground Investigations

From logging of the boreholes (maximum depth of 30m) the following points were reported in the site investigation report:

(A) Apart from superficial deposits of made ground and blue or blue and brown organic clay with gravel at the surface, the whole of the stratum encountered was the London Clay.

(B) The London Clay showed the usual succession of weathering from the surface downwards.
(i) light brown clay, with little or no fissuring (in boreholes 7, 8 and 9 only)
(ii) blue and brown mottled clay or brown clay with blue fissuring
(iii) stiff brown fissured clay
(iv) unweathered, stiff to hard, blue highly fissured clay.

The weathering of the London Clay decreases continuously with depth, with no abrupt changes in characteristics, from the light brown, highly weathered clay at the surface in boreholes 7, 8 and 9. The depth to the unweathered blue clay was between 8 and 10m which is not unusual.

(C) The blue clay was noticeably more silty in most of the boreholes at depths between 16 and 20m below ground surface.

(D) A regression analysis on the data shown in Fig. 3.2 resulted in the line plot shown on the figure. From this it indicates that average cohesion (undrained shear strength on 38mm diameter specimens) increases with depth, showing the clay to be generally firm at a depth of less than 2m, stiff from 2 to 8m and very stiff below.

(E) The individual points are scattered about the mean value, as expected with over-consolidated, fissured clays. The scatter is greater for the stiffer clays which are more highly fissured. At a depth of 4m, the ratio of mean cohesion (undrained shear strength) given by the line to the minimum measured cohesion is 2.1. At greater depths, this ratio does not exceed 1.7. This would indicate that if average cohesion (undrained shear strength) should be used to estimate ultimate bearing capacities, a factor of
safety of 2.1 or more should ensure that none of the clay is overstressed.

(F) From the interpretation of the results of the consolidation tests a plot of coefficient of volume compressibility $m_v$ verses depth, Fig.3.2/3, was produced. A line based on a regression analysis, represents average compressibility with depth. This indicated no significant variation with depth to 10m below ground surface. At greater depths the clay becomes less compressible.

(G) To a depth of 10m the compressibility is classed as moderate and at 20m and below is small.

(H) The scatter of the individual points about the line is similar to that for the cohesion (undrained shear strength) against depth relationship. The coefficient of consolidation is small, indicating a slow rate of consolidation settlement.

(I) For London Clay, the elastic modulus has been determined from the assumed relationship elastic modulus $E_u = 400C_u$. From the modulus, a compressibility applicable to elastic movements was obtained. For a Poisson's Ratio of 0.5 the relationship is:

$$m_v = 0.75/E_u$$

This compressibility derived from the cohesion, indicated by the regression analysis, is plotted against depth in Figure 3.2/4.

A safer assessment of item (E) would be to apply the factor of safety on the line through the minimum strength $C_u$ with depth. If the practice suggested in (E) above is followed, with respect to the values for $C_u$, it may be that some areas of the soil under the
foundation have a factor of safety of 1 or less on bearing capacity failure and not a minimum factor of safety of at least 2. The implications of assuming the relationship $E_u=400C_u$ are discussed later in section 3.5 (Design Analysis).

The compressibility is expressed in two forms in Figs.3.2/3 and 3.2/4 in the forms $m_g$ and $m_a$. The suggested values are quite different and there are a number of reasons for this. The test used to obtain Fig.3.2/3 was carried out under $K_0$ conditions whereas those in Fig.3.2/4 were obtained from the undrained strength test and an empirical formula. The very nature of the test would suggest that Fig.3.2/3 overestimates the stiffness. The compressibility in Fig.3.2/4 was calculated from the relationship $E_u=400C_u$ and not $220C_u$ as is used in practice in industry. This would result in larger values of $m_a$ by a factor of about 2 which make them more comparable with the values of $m_g$ from Fig.3.2/3. Table 3.2/1 shows the stiffness values $1/m_g$ and $1/m_a$ suggested from the two laboratory tests and compares the values of stiffness with the stiffness suggested by Butler (1974) from the relationship $E_u=220C_u$ and the stiffness suggested after the dimensionless plot of $E_u/C_u$ v strain after Jardine et al (1986). The value of suggested stiffness is taken from the strength tests from the undrained triaxial.

3.2.3 Design Guidelines Outlined in the Report

Although there was no specification to undertake the work the site investigation company suggested design limits for the foundations for spread footings, piles, basements and retaining walls. Specific reference was made to the Alders basement. Comments are made after the appropriate extracts from the report where appropriate. The extracts from the investigation report are in Italics.
General Considerations

Made ground is present in all the boreholes, varying in thickness from 0.3 to 0.7m. In Boreholes 1, 2, 3, 4, and 9, this is followed by a sandy clay, sometimes with gravel and organic matter. This is possibly alluvium and is between 0.3 and 0.8m thick where it occurs. The variation in thickness of these superficial deposits ranges from 0.3m (Borehole 8) to 1.5m (Borehole 1).

The clay below had a high shrinkage potential and roots were observed in samples to a depth of 1.5m below ground surface in Boreholes 3, 5, and 6. The possibility of foundation movement would be likely at founding depths less than 2m because of slow changes in moisture contents of the clay after construction.

The minimum depth for the founding of any of the major structures on site should be taken as 2m below present ground level.

The above are standard practice for working in a material such as the London Clay. More specific design limitations were then outlined in the form of particular foundation types again extracts from the investigation report are in italics.

Spread Footings

The net safe bearing capacity of spread footings was estimated from the cohesion (undrained shear strength) given by the line in Fig. 3.2/2, allowing a factor of safety of about 3 on the estimated bearing capacity. Table 3.2/2 outlines the results of this exercise carried out for footings at different depths and of different sizes. The estimated long term settlements for some of these bases are also indicated. Settlements of the same order would also be expected during construction. For the larger bases the estimated settlements are considerable.
It is thus likely that the criterion for the design would need to be on acceptable settlement rather than bearing capacity.

**Piles**

Table 3.2/3 gives the estimated ultimate values of pile bearing characteristics for bored cast-in-situ piles. The pile bearing characteristics are based on a correlation between results of pile loading tests and the soil characteristics based on the distribution of cohesion given by the line on Fig.3.2/2. The figures given for skin friction are average values which depend greatly on the construction of the pile. For cast-in-situ bored piles, constructed to high standards, the skin friction could be 50% more than the values given.

In considering the table the bearing capacities should be used for preliminary design purposes only. The following points should also be taken into account.

(A) If the bearing capacities are confirmed by test loading, then a factor of safety of about 2 used on the ultimate load should be sufficient for the stability of uniform diameter piles.

(B) In the absence of test loading, when additional allowance should be made for variation in the ground conditions and for the method and type of pile construction, then the estimated ultimate load should be divided by a factor of at least 2.5 to give the working load of uniform diameter piles.

(C) For piles in groups, the bearing capacity obtained for a single pile should be multiplied by 0.8 giving a reduced value allowing for the group effect.
(D) For belled piles with under-reamed bases, no skin friction should be allowed to a height above the base at least equal to its diameter.

(E) For the belled piles, the factor of safety on the end load should be increased to at least 3, chiefly to limit the immediate settlement which occurs with large diameter piles of this type.

(F) It is important with belled piles to ensure that all the spoil and disturbed clay is removed from the base of the pile before concreting. The blue clay at depths below 15m is fissured, but from the samples obtained, the amount of fissuring is consistent with that found elsewhere in the London area.

It is of interest that the general reduction factor of 0.8 is given in (C) for the reduced recommended bearing capacity of the piles when they are in a group. If test piles have been used then the work by Douglas and Butterfield (1984) suggests that the load deflection plot can be converted to a dimensionless pile stiffness plot and the group reduction factor can then be calculated directly by using the general plots produced by the program PGROUP. Other methods of making an allowance for groups were also available at the time as well as those currently proposed (Poulos and Davies (1980), Poulos (1989), Cooke et al (1980), Cooke (1986)). The latter makes specific reference to the conservative nature of pile design in general and outlines ways in which the number of piles can be significantly reduced. If the design is looked at in the context of using the bearing capacity of the piles to carry the required working load, making the factor of safety on the piles unity, the real factor of safety would be provided by the bearing capacity of the raft. Another method he suggests is to
stipulate an acceptable settlement and increase the number of piles only until this value is attained.

In a more general context, relating to the observations made during the ground investigation the following points of note were made:

1) No water was noted during the drilling of any of the boreholes. It would thus appear reasonable to assume that similar conditions would prevail during the construction of any pile boreholes. Small seepage from the clay is not likely to affect the construction of the pile.

2) It was noted, during visual sample examination, that some of the clay appeared to be of a rather more silty nature. This was found to be the case particularly between 15 and 20m. It is known from past experience of the London Clay that inclusions of a more silty nature do occur and thus they must be expected in any pile borehole. Where this silty clay occurs there may be difficulties in forming an under-ream, however, no difficulties arose whilst drilling the trial boreholes.

Considerations of the order of magnitude of settlement expected were then highlighted.

It is possible to estimate the immediate settlement which will occur as the load is applied with sufficient accuracy for this to be a useful guide to the amount of settlement to be expected. After the initial settlement, long term consolidation settlement of these piles should be small, particularly if they are found in the less compressible clays at a depth of 20m or more.

Table 3.2/4 shows possible pile sizes required for different working loads calculated in the same way as the examples. The settlement
figures could vary by about \( +50\% \) of the figures given. The pile caps were taken at a depth at which construction is likely. If they should be founded at a higher level, the working loads would not be significantly increased, nor would the estimated settlements significantly alter. The long term settlements, as previously mentioned, would be small as the piles are all founded at a depth of 20m or more.

**Basements**

During the excavation for a basement, the soil below the excavation is relieved of some of its load, expands elastically and results in heave. It can be seen from Fig.3.2/4 that should basements be sited at a depth of 5m or more below present ground level the clays of greater compressibility are avoided. For a large excavated area, say 50m wide and 100m long, the elastic heave in the central part of the excavation is of the order of 20mm for the removal of overburden equivalent to 100kN/m\(^2\). Around the edge of the excavation the heave will be smaller and smaller still at the corners. The heave may in general be taken as proportional to the overburden pressure removed and will not vary greatly within that area if the area is greater than a 15m square.

Long term heave is likely if the soil at basement level remains unloaded. Although there are not sufficient observations of measured movement available to give a reliable correlation between heave and soil parameters, a tentative estimate of the fractions of the total long term heave is indicated below in Table 3.2/5.

Taking the above and general observations of the site conditions into account, the following points were raised:

A) Should the basement slab be loaded to overburden pressure, the heave during the unloaded period would be recovered, following which there should be little further movement.
If piled foundations were not to be used, there would therefore be an advantage in spreading the column loads over the basement slab.

B) It has been suggested, (Settlement of Structures 1975), that the "use of double-skinned basement floors allows the clay beneath freedom to heave. Where rigid structures on piles have been installed to prevent heave effects, a measure of success seems apparent".

C) It must be assumed that there is a water table, (Table 3.2/6), which may be as high as 1m below ground surface as indicated in borehole 5. It is considered that further confirmation be obtained by two piezometers installed at 9m in the area of the service basement. Further as the water level in the Alders basement was not determined it must be assumed to be at 1m below ground surface, or at least two piezometers should be installed to determine the water table in this location.

Retaining Walls

The outside walls of the basements will need to act as earth retaining structures. No difficulties in excavation are foreseen. Apart from superficial spalling the firm and stiff clays should stand unsupported for a short time, with the superficial soils cut back at an angle of 45°.

The active pressures were estimated for three conditions, allowing interpolation for other water levels.

1) Completely dry.
2) Water table at 2.5m below surface.
3) Water table at surface.
In all cases where the calculated active pressure is less than the hydrostatic ground water pressure, the latter applies, and was used for the made ground and blue clay with gravel.

**Alders Basement**

Specific observations were made with respect to the Alders Basement in the context of the overall ground investigation.

Only one borehole was drilled in this area, and no provision was made for observations of the ground water level. In the absence of proof to the contrary, water levels as found in the boreholes which they were measured over a period of time must be assumed. As already pointed out (previous section Basements) it was considered advisable to establish the water level at a number of points within the basement area.

The proposed basement was to have the floor level at a depth of about 5m over the whole of an area roughly 105m long and 48m wide. The proposed column loads ranged from 5000 to 9000 kN. It was estimated that if the column loads were uniformly distributed over the area of the basement slab, the average loading would be about 96 kN/m² excluding the slab itself.

Taking the above into account the following points were made:

1. If the average bulk density (unit weight) of the soil excavated is 18 kN/m³ then the relief of pressure at the basement level would be about 90kN/m². During the excavation, the estimated elastic heave is about 20mm, and as the load was replaced on the basement slab, a similar settlement would be expected under elastic compression.
b) If water was present to allow swelling of the clay by the consolidation process, a slow swelling would occur. The total amount of swelling would depend on the length of time during which the clay was unloaded. With a construction period of about 6 months, this may be of the order of 10 to 20mm, and a similar recompression would again be expected on reloading.

c) After these initial movements, there should theoretically be very little consolidation movement, as the pressure on the clay immediately below the slab is very similar to the present overburden pressure at that level.

d) However, with the high column loads, spaced widely, it is not likely that the slab could be designed to be sufficiently rigid to distribute the column loads to approach the condition of uniform loading below the slab. The distribution of pressure likely was not estimated, but the differential settlements between different parts of the floor slab and particularly between heavily loaded internal column and lightly loaded external columns must be anticipated.

e) The use of piled foundations would eliminate most of the above uncertainties and give simpler design.

Items a), d) and e) are of most interest when looked at together. The use of piled foundations or a thick slab would result in concentrations of load under the column locations creating areas of positive and negative net bearing pressure and would have implications on the design of the foundation even if the design bearing pressure for the foundation matched the net relief pressure of 90kN/m².
Table 3.2/1 Suggested values of Modulus for the London Clay

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$S_u$ (kN/m²)</th>
<th>$1/m_u$</th>
<th>$1/m_c$</th>
<th>Butler (1974) 220CU</th>
<th>Jardine (1986) 400CU</th>
<th>$E_u0.01$</th>
<th>$E_u0.1$</th>
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</thead>
<tbody>
<tr>
<td>5</td>
<td>130</td>
<td>10</td>
<td>62</td>
<td>28</td>
<td>52</td>
<td>131</td>
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<td>180</td>
<td>22</td>
<td>133</td>
<td>40</td>
<td>72</td>
<td>182</td>
<td>99</td>
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Table 3.2/2 Estimated Safe Bearing Capacity and Settlements of Spread Footings

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Width (m)</th>
<th>Strip Footing $q_s$ (kN/m²)</th>
<th>Square Base $q_s$ (kN/m²)</th>
<th>Total load square base (kN)</th>
<th>Estimated settlements square base (mm)</th>
</tr>
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<tbody>
<tr>
<td>2m below</td>
<td>1</td>
<td>180</td>
<td>215</td>
<td>215</td>
<td>8</td>
</tr>
<tr>
<td>surface</td>
<td>2</td>
<td>170</td>
<td>205</td>
<td>820</td>
<td>22</td>
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<tr>
<td>3</td>
<td>3</td>
<td>165</td>
<td>200</td>
<td>1800</td>
<td>22</td>
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<td>4</td>
<td>160</td>
<td>195</td>
<td>3120</td>
<td>36</td>
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<tr>
<td>5</td>
<td>5</td>
<td>160</td>
<td>190</td>
<td>4750</td>
<td>36</td>
</tr>
<tr>
<td>1m below basement</td>
<td>2</td>
<td>285</td>
<td>340</td>
<td>340</td>
<td>13</td>
</tr>
<tr>
<td>at 5m</td>
<td>3</td>
<td>270</td>
<td>325</td>
<td>1300</td>
<td>36</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>265</td>
<td>320</td>
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<td>5</td>
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<td>5040</td>
<td>55</td>
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<td>basement</td>
<td>5</td>
<td>270</td>
<td>310</td>
<td>7750</td>
<td>55</td>
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<tr>
<td>1m below basement</td>
<td>2</td>
<td>315</td>
<td>380</td>
<td>1520</td>
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<tr>
<td>at 7m</td>
<td>4</td>
<td>310</td>
<td>370</td>
<td>3330</td>
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</tr>
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<td>5</td>
<td>305</td>
<td>365</td>
<td>5840</td>
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</table>

Table after Ground Explorations Ltd.
### Table 3.2/3 Estimated Ultimate Values of Pile Bearing Characteristics

<table>
<thead>
<tr>
<th>Depth</th>
<th>Adhesion/Skin Friction</th>
<th>End Bearing Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Present ground level to 2m Neglect</td>
<td>kN/m²</td>
<td>Straight Belled</td>
</tr>
<tr>
<td>Depth - less than 5m</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>5 - 10m</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>10 - 15m</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>15 - 25m</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>Below 25m</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>10m</td>
<td>1500</td>
<td>1500</td>
</tr>
<tr>
<td>15m</td>
<td>1850</td>
<td>1550</td>
</tr>
<tr>
<td>20m</td>
<td>2050</td>
<td>1700</td>
</tr>
<tr>
<td>25m</td>
<td>2200</td>
<td>1850</td>
</tr>
<tr>
<td>30m or below</td>
<td>2350</td>
<td>1950</td>
</tr>
</tbody>
</table>

Table after Ground Explorations Ltd.

### Table 3.2/4 Possible Design of Piles (Pile Caps at about 6m Depth)

<table>
<thead>
<tr>
<th>Working Load (kN)</th>
<th>Shaft Diameter (m)</th>
<th>End Diameter (m)</th>
<th>Depth of Base (m)</th>
<th>Estimated Immediate Settlement Under Working Load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>600</td>
<td>600</td>
<td>23</td>
<td>under 2</td>
</tr>
<tr>
<td>2000</td>
<td>900</td>
<td>900</td>
<td>26</td>
<td>under 2</td>
</tr>
<tr>
<td>4000</td>
<td>900</td>
<td>2700</td>
<td>20</td>
<td>15</td>
</tr>
<tr>
<td>6000</td>
<td>900</td>
<td>3000</td>
<td>30</td>
<td>15</td>
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<td>8000</td>
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</tr>
<tr>
<td>10000</td>
<td>1500</td>
<td>4500</td>
<td>20</td>
<td>31</td>
</tr>
<tr>
<td>10000</td>
<td>1500</td>
<td>3900</td>
<td>30</td>
<td>21</td>
</tr>
<tr>
<td>12000</td>
<td>1500</td>
<td>4500</td>
<td>30</td>
<td>26</td>
</tr>
</tbody>
</table>

Table after Ground Explorations Ltd.
### Table 3.2/5 Estimated Long Term Heaves

<table>
<thead>
<tr>
<th>Time (months)</th>
<th>% of total heave</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>5-10</td>
</tr>
<tr>
<td>6</td>
<td>12-18</td>
</tr>
<tr>
<td>12</td>
<td>20-25</td>
</tr>
<tr>
<td>18</td>
<td>25-30</td>
</tr>
<tr>
<td>24</td>
<td>30-35</td>
</tr>
</tbody>
</table>

Table after Ground Explorations Ltd.

### Table 3.2/6 Water Levels in Piezometers and Standpipes

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>Depth of water (m) below ground level on:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.11.80</td>
</tr>
<tr>
<td>3</td>
<td>*</td>
</tr>
<tr>
<td>5</td>
<td>*</td>
</tr>
<tr>
<td>6</td>
<td>*</td>
</tr>
<tr>
<td>7</td>
<td>*</td>
</tr>
<tr>
<td>8</td>
<td>1.5</td>
</tr>
<tr>
<td>9</td>
<td>1.8</td>
</tr>
</tbody>
</table>

* No reading Taken
+ Obstructed by Parked Car
$ Obstructed by Materials in Builder's Yard

Table after Ground Explorations Ltd.
Figure 3.2/1 Plan of Phase II of the Basildon South East Town Development

NB. The position of TP.2 is not indicated on the available drawings.
Figure 3.2/2 Undrained Strength Tests on 38mm Diameter Samples
Figure 3.2/3 Plot of $m_v$ against Depth. $m_v$ derived from Consolidation Tests on 75mm Diameter Samples
Figure 3.2/6 Elastic Compressibility - Depth Relationship
3.3. Description of the Shopping Complex

The Alders building forms part of Phase II of the Basildon South East Town Development instigated by the Basildon Development Corporation. The development, which includes two large stores, two office blocks, a large number of individual shop units, and multi-storey car parking facilities, forms part of one of the largest covered shopping areas in Europe. A plan indicating the general layout of the development is shown in Fig.3.3/1. Two basements are contained within the development, one beneath the large department store (A) and the other below the shop units (B). These basement areas are indicated in Fig.3.3/1 by the broken outline.

Alders Building

The large department store building is rectangular in plan, 105.6m long by 48.0m wide, comprising of a basement ground floor and two upper floors. The basement occupies part of the overall area, being 75.6m long by 48.0 wide, as shown in Fig.3.3/2. The main structure of the building is of reinforced concrete construction, with brick cladding and a lightweight metal decking roof system supported on structural steelwork. Columns are arranged on a 10.8 x 9.6m grid.

Construction of Alders

The principle behind the construction process is outlines in Figs.3.3/3 to 3.3/8.

The only major constraint on construction was that the basement walls were designed as a propped cantilever which imposed the condition that no backfilling could be permitted until the basement and ground floor were completed. This constraint also meant that the excavation was pumped until this backfilling was complete. This, it was thought, would help minimise any build-up of water pressure under the basement.
before the ground floor was constructed and further construction dead loads had been added.

The construction works were undertaken utilising a tower crane positioned centrally on the basement slab. This necessitated infilling of floor slabs once the building had been substantially completed.

The floor slabs were cast on a scaffolding system and thus the construction loads were substantially taken down to the basement slab until such time as the scaffold was stripped out. The loads were then taken by the columns and hence substantially down to the piles on which the basement columns were cast. This had the affect of redistributing the load from the basement floor slab to the piles.
Figure 3.3/1  Site Plan Showing the Location of the Basements
Figure 3.3/2 Location Plan, Building Plan and Section of the Alders Building
Drawings after White Young & Partners

Figure 3.3/3 Pile Installation

Figure 3.3/4 Excavation for Substructure
Figure 3.3/5 Construct Substructure (Including Ground Floor Slab)

Figure 3.3/6 Backfill to the Structure
Figure 3.3/7 Construct the Superstructure (Short Term Loading)

Figure 3.3/8 Finished Structure (Long Term Loading)
3.4. General Considerations

An assessment of the anticipated loading showed that for the two basements the maximum gross foundation pressure would be as follows:

Department store basement 70kN/m²
Unit shops basement 98kN/m²

Assuming a unit weight of 18kN/m³ for the excavated clay, the weight of soil removed would be equivalent to a stress removal of:

Department store basement (5m deep) 90kN/m²
Unit shops basement (6m deep) 108kN/m²

This leaves a predicted net foundation pressure of:

Department store basement -20kN/m²
Unit shops basement -10kN/m²

Thus in both cases the anticipated foundation loads were less than the weight of soil removed. With these considerations in mind the structural design consultants considered several options for the design of the basements.

In the first instance a conventional raft solution was considered by the consultant. With the total building loads approximately the same order as the weight of overburden removed it is clearly attractive at first sight. However, the large column spacings dictated by the design brief meant that a raft of considerable thickness would have been required to distribute the loads evenly to the ground. A cost comparison exercise showed that a thinner slab supported on piles would be cheaper.
A further consideration was that the basements only extend over part of the buildings, and the possibility therefore arises of substantial differential settlements between those areas of the buildings outside the basement areas and those within them. Consideration of the high degree of interconnection between the various buildings within the development because of the expensive level of finishes, some very sensitive to movement, led to the following conclusions.

The differential settlements which could occur with pad and raft foundations were not acceptable, and therefore piled foundations were adopted for all the buildings in the development, including the basements.

Having chosen a piled basement solution, the consultant considered interaction of the piles, the basement slab and the soil. He identified the principal loads acting on the foundation system as follows:

(i) Building loads.

(ii) Uplift due to water pressure on the underside of the slab.

(iii) Uplift due to clay heave, resulting from the net relief of overburden pressure.

In order to eliminate the effect of heave, some thought was given to the possibility of forming a void beneath the basement slabs, thus allowing the heave movements to take place without applying uplift forces to the underside of the slabs. A number of ways of achieving this were investigated, including the use of temporary, or permanent formwork, and a patent void former. All were rejected, either because of cost, or difficulty in ensuring a satisfactory waterproofing detail and in the case of the patent void former, reservations about the construction life of the product.
Piled basements with no void were chosen. Initially the designer approached the design by following the conventional route to carry all the structural load on the piles, with one large diameter pile located at each column position. The basement slabs were designed to carry the full theoretical uplift. However, previous research into buildings with piled raft foundations indicated there was a significant degree of load-sharing between the piles and the raft. (Cooke et al 1981, Cooke et al 1984, Cooke 1986, Hooper 1979, 1980, Poulos 1989).

The designer's structural engineers realised that substantial savings could be made if this behaviour was taken into account in the design of the basements. To this end a specialist geotechnical consultant was approached for further guidance.

The report from the geotechnical consultant highlighted the following points:

(a) With a piled raft in the London Clay it is usually assumed that all the load is carried by the piles. If the raft is to be placed at some depth below the original ground surface such that a significant amount of excavation is required, it is sometimes further assumed that the raft should be designed to resist a heave pressure equal to the maximum relief in loading. In a situation where the net increase in loading on the ground is zero, as is the case for both Alders and Unit Shops, the above two assumptions can lead to a piled raft foundation being designed to carry over twice the applied loading. This is clearly most uneconomical.

(b) The problem of load sharing between raft and piles is extremely complex and cannot be reliably analyzed at the present time. It should also be borne in mind that it is
known that the relative proportions of load carried by the raft and piles will vary with time and therefore any design must take the most adverse combinations of load sharing into account.

(c) Summarising the available data on load sharing for piled rafts in the London Clay and on the basis of these data, it should be possible to make safe, but economic proposals for the foundations for Alders and Unit Shops.

Two reports were cited which contained information with particular reference to the behaviour of piled raft foundations in the London clay. The first containing four case records was published by Hooper (1979) and included the following:

(a) Hyde Park Cavalry Barracks Tower.

(b) Victoria Street Redevelopment.

(c) National Westminster Bank Tower.

The second report for the case record Stonebridge Park, has been described by Cooke et al (1981).

The data are summarised in Table 3.4/1, omitting the Victoria Street Redevelopment. The specialist geotechnical consultant gave the following reasons for omitting this record.

The foundations of the Victoria Street Redevelopment project have been instrumented and the results reported by Hodgson and Bryan (1975). The consultant was associated with this project and pointed out that the observations were very difficult to interpret. The distribution of pressure under the raft appeared to vary greatly and seemed to change with time in a somewhat inconsistent manner. Of the third
level basement, the instrumentation only covered a part of the total foundation area and clearly there will be interaction between all parts of the foundation.

On this basis the consultant considered that not a great deal of reliance should be given to the case record. It is however interesting to note that the measured contact pressures varied from 20 to 100kN/m² which may be compared with the gross loading (approximately equal to the maximum relief of loading) of about 200kN/m².

It was considered unfortunate that definite conclusions could not be drawn from this project, as it is the only case where the net increase in load is very small, as is the situation with both the Unit Shops and Alders.

Consideration of Table 2.4/1, for the four cases, showed that the load taken by the piles varied from 55% to 75% of the total load. However, care must be taken in using these data since all had significant net positive foundation pressures applied to the ground, whereas for Alders the net foundation pressure was slightly negative.

A point of further interest, raised by the geotechnical consultant is that at Stonebridge Park, where there was very little unloading and the base of the raft was placed in the weathered and softened London Clay, the raft still took a significant (within the 25% to 45%) proportion of the total load.

In considering heave the Consultant made the following comments in considering negative net increases in total load.

*For both Alders and Unit shops the theoretical maximum possible uplift pressure cannot exceed the relief in loading due to excavation, as a downward pressure of this magnitude applied by the slab to the clay*
would entirely prevent any heave occurring. In practice the real uplift pressures will be less than the theoretical maximum.

(a) because the excavation naturally takes place before the slab is cast and therefore some heave of the ground takes place before the slab is constructed and less pressure will be needed to counteract the remainder of the heave.

(b) the piles, having already been installed, will, to a certain extent, act as reinforcement in the soil and will carry some of the heave pressure.

Calculations have shown that an economical slab can be designed if an uplift pressure of 70% of the theoretical maximum possible relief pressure is taken into account. This figure results in the design uplift pressures of 75.6 and 63.0kN/m² for Unit Shops and Alders respectively, as shown in Table 3.4/1. These pressures represent 77% and 90% of the applied loading.

The Consultant compared the proposal that the slabs should be designed to carry uplift pressures of 77% and 90% of the total loading with the general recommendation by Tomlinson (1977). "In any piled basement where bored piles are installed wholly in compressible clay, the basement slab should be designed to withstand an uplift pressure equal to one-half of the dead and sustained imposed load of the superstructure." and explained as follows:

Because the net increases in loading on the ground are negative for the two structures, there is the possibility that a slight swelling of the clay may occur with time, leading to somewhat higher uplift pressures than would be expected for cases where net positive pressures are applied to the ground, which is the more usual situation which Tomlinson no doubt had in mind when making his general recommendation.
In order to accommodate any additional loads transferred to the piles by skin friction from the clay, tensile reinforcement in the piles has been designed to carry 85% of the theoretical maximum possible relief pressure less the minimum weight of the buildings.

The Consultant's comments on the compressive loads follows.

As far as the design compressive loads for the piling are concerned, the most critical situation is likely to occur at the end of the construction period, before any significant swelling of the clay has occurred. The uplift pressures on the slab are likely to be a minimum at this time. It should also be noted that the column spacing is large and, with the piles in general being placed immediately under the columns, a relatively flexible slab is obtained.

Detailed computer analysis of the piled slab interaction based on the average distribution of total load between piles and slab indicated that for Alders basement the piles would carry 51% of the total load with the corresponding figure for Unit Shops being 60%. To provide an additional safety factor, these percentages have been increased by 16%, 10% being a contingency allowance and 6% for long term settlement effects following Hooper (1979). This adjusts the above figures to 67% and 76% for Alders and Unit Shops respectively.

It was pointed out that in comparing these figures with the observed data in Table 3.4/1, that although the percentage of load taken by the piles lies within a fairly narrow range (55% to 75%), for the four sets of data observed the loading at Basildon is different. Noting that the net increase in loading under the slabs for the Basildon structures is slightly negative while large positive increases were observed for the four case records, it must be expected that for structures at Basildon a greater proportion of the load will be taken by the piles.
The Consultant's comments on the likely heave were as follows:

The present state of knowledge makes it difficult to predict reliably the rates of heave, but intermediate values of heave pressure expressed as a percentage of the maximum relief pressure may well be 25% and 50% at 6 months and 12 months respectively.

The Consultant's concluding remarks were as follows:

Based on published case records and, bearing in mind that the most critical periods for the maximum pile loadings and maximum uplift pressures are likely to occur at quite different times, the following design recommendations are made and are believed to be safe and economical.

1) Slab uplift pressure = 70% of reduction in loading due to excavation.

2) Pile capacity in compression = a minimum of 76% of total applied loading for Unit shops and a minimum of 67% of total applied loading for Alders.

3) Pile capacity in tension = 85% of theoretical maximum possible relief pressure, less the minimum weight of the buildings.

The designers, aware of the fact that the piled basements in the Basildon Project differed from the previously reported cases, recognised two main design conditions:

(i) Short term: with piled raft system designed to resist the maximum building loads, allowing for load sharing between the piles and raft, but
ignoring uplift due to water pressure and heave effects.

(ii) Long term: with piled raft system subjected to maximum uplift forces with minimum building loads.

They further qualified their position thus.

Clearly, many other intermediate load cases could occur and indeed were more likely to occur but these two conditions were regarded as representing the two feasible extremes. Yet by taking load sharing into account a more economical solution could be achieved than using more traditional methods.

Loading of the structure during the various stages of construction was also considered. A possible critical condition was recognised if substantial uplift, due to water pressure, was allowed to develop before substantial building weight had been applied to counteract it. Design of the basement walls as propped cantilevers imposed the condition that no backfilling could be permitted until the basement and ground floor were completed. Pumping the excavation free of water would then minimise any build up of water pressure under the basement before the ground floor was constructed and sufficient dead weight had been added. The situation above did not take into account the way in which the loads would be acting at this phase of the construction. The design of the building meant that both dead and live loads would be transmitted to the columns, through to the pile heads which were situated under the column locations. Thus the piles would need to move for the slab to carry load transmitted down the columns before load sharing between the piles and the raft took place. The situation that exists during the construction phase is very different. The floor slabs were constructed on a scaffold, falsework, system. This system takes the construction dead loads from the building and the falsework, through the falsework and not through the columns. The
load taken through the falsework results in a uniformly distributed load (udl) on the basement floor. Very little load is taken through the columns to the pile heads until there is sufficient movement of the basement for load to be taken by the piles and load sharing to commence.

3.4.1. Alders Loading

The clients specification for the building meant that large column spacings were a necessity. As a result the following tables give the dimensions given by the client and the resultant column dead and live loads (Table 3.4/3). This information is followed by the properties of the London Clay to be used in the design and a statement about the water table, the latter due to its implications on the design.

Column Loads

The column loads are to be found in Table 3.4/3. Reference should be made to Fig. 3.3/2 for the column locations.

Properties of the London Clay

\[ E_u = 400 \times C_u \]  
See Fig. 3.2/2.

\[ \text{m} \] to a depth of 10m classed moderate compression.

20m and below classed small compression.

Liquid limit, plastic limit and plasticity indices classification CV.

Water Table

Observations of the ground water levels indicated a high water table during the winter months Table 3.2/5. Seasonal variations were not possible from the data. Where no direct readings were taken the water table must be assumed to be close to the surface.
Table 3.4/1 Summary of the Available Load Sharing Data

<table>
<thead>
<tr>
<th>Case</th>
<th>Gross Record Foundation Pressure (kN/m²)</th>
<th>Relief of Loading due to excavation (kN/m²)</th>
<th>Net Foundation Pressure (kN/m²)</th>
<th>Uplift Load on Piles (kN/m²)</th>
<th>Load on Slab (%)</th>
<th>Values for use in design</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>370</td>
<td>170</td>
<td>200</td>
<td>148</td>
<td>60</td>
<td>75.6 76 MIN 77</td>
</tr>
<tr>
<td>B</td>
<td>260</td>
<td>115</td>
<td>145</td>
<td>91</td>
<td>65</td>
<td>63.0 67 MIN 90</td>
</tr>
<tr>
<td>C</td>
<td>625</td>
<td>285</td>
<td>340</td>
<td>156</td>
<td>75</td>
<td>75 45</td>
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<td>0</td>
<td>187</td>
<td>84</td>
<td>55+</td>
<td>35+</td>
</tr>
</tbody>
</table>

* At an early stage of the construction
+ At the time of occupation

A Hyde Park Cavalry Barracks
B Dashwood House
C National Westminster Bank
D Stonebridge Park

Table 3.4/2 Design Parameters as a Result of the Clients Specification

- Plan Dimensions: 48.0m x 75.6m
- Column centres: Along the length 10.8m, along the width 9.6m

- Foundation pressure
  - Maximum gross: 70kN/m²
  - Stress relief (removed soil to depth of 5m): 90kN/m²
  - Therefore predicted net: -20kN/m²
<table>
<thead>
<tr>
<th>Column Type</th>
<th>Dead Load (kN)</th>
<th>Live Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long Wall Column</td>
<td>3570</td>
<td>1235</td>
</tr>
<tr>
<td>Short Wall Column Adjacent to Liftwell</td>
<td>3972</td>
<td>1387</td>
</tr>
<tr>
<td>Internal Column Adjacent to Stairwell</td>
<td>4538</td>
<td>1948</td>
</tr>
<tr>
<td>Internal Column</td>
<td>5625</td>
<td>2465</td>
</tr>
<tr>
<td>Short Wall Column</td>
<td>4356</td>
<td>1495</td>
</tr>
<tr>
<td>Corner Wall Stairwell</td>
<td>5460</td>
<td>2293</td>
</tr>
</tbody>
</table>
3.5 Design Analysis

3.5.1. The General Design Method

The design analysis was undertaken for the complex in two phases.

1) At tender stage, when the feasibility of the design was undertaken.

2) At contract stage, when the piling contract had been let and the pile design could then be modelled in the analysis with the finalised design loads.

The majority of the analysis was undertaken in phase 1 where variations to parameters were studied to verify the principle behind the design. A report was submitted to the Client, Basildon Development Corporation, by White Young Consulting Engineers as a "Detailed Investigation of Parameters Affecting Percentage Load Sharing Between Piles and Raft within the Basements".

The consultant introduced the report as follows.

The loads applied generally to the foundations were such that in 50% of the column positions, pad foundations could seriously be considered. However, in this situation there were two problems associated with them.

(1) Differential settlement

(2) Irregular layout of the buildings.

The size and spacing of column loads within the office building were such that piles were without doubt the correct solution. The same comment must also hold for the column loads within the unit shops.
development. All the buildings are interconnected by two Shopping Malls which were not only covered by sensitive glazed roofs but also had an expensive level of finishing, e.g. marble floor coverings. Further, the size of the buildings interconnected by the malls vary from two to ten storeys. Thus the resulting variation in pad sizes would have been a problem not only within the buildings themselves, but also between adjoining buildings.

The basements presented their own design complexity. Firstly they do not extend uniformly under any one building. Secondly, in the case of Unit Shops, the basement is a very irregular shape. However, the weight of the building erected on the basements is less than the weight of the soil excavated to build them. Thus the soil underwent a net loss of loading at the underside of basement slab level and would be expected to heave. There are numerous case histories to support this theory, for example Shell Centre London, Tomlinson et al (1977).

The adopted design solution was to use piled foundations throughout the development. To ensure no problems of differential settlement, either positive or negative inter slab movement, piles were proposed within the basements. The basements had to be designed to overcome possible heave pressures. Therefore the stiffness of the basement raft was unlisted and a shared pile/raft design solution was adopted at tender stage.

Besides basic 'hand checks', the principal calculations for the design element made extensive use of the computer. The report investigated the effect of varying the soil moduli, size of analysis grid and pile model have on the percentage load sharing between pile and raft.

The consultant carried the analysis out using the programs, commercially available through SIA Computer Services, 'RAFT' in conjunction with 'LEAP' following the guidelines "Introduction to Grillage Analogy" produced by the same company.
The basement slab was modelled as a grillage. The soil was simulated as a series of discrete springs supporting the grillage node points situated on both slab and pile locations.

The grillage was analyzed by LEAP using an initial estimate of the spring stiffness to obtain a set of spring loads. These loads were then used in the RAFT analysis to obtain a revised set of spring stiffness. These revised spring stiffness were then used for re-analysis by LEAP. The analysis continued with this iterative procedure until relatively small changes in spring constants occurred.

The three major variables in the analysis were considered as;

a) Young's Modulus for the soil.

b) the method of modelling the piles as springs.

c) the size of the analysis grid layout.

The site investigation report had suggested a value of undrained elastic modulus ($E_u$) equal to 400$C_u$. The consultant also suggested that this is also the value suggested by Butler (1974) for London Clay and used values ranging from 200 to 600$C_u$ to investigate the sensitivity of the LEAP/RAFT model to the soil stiffness. Butler (1974) reported values ranging from 140$C_u$ for laboratory tests after Skempton and Henkel (1957) to values of 600 to 830$C_u$ for constant-rate-of-strain testing on large plates after Marsland and values of 650 to 830$C_u$ for plate bearing tests after Burland, Butler and Duncan (1966). The latter are very sensitive to the time between excavation and testing (Marsland 1973) and had been found to be halved if tested 10 hours after excavation. Butler’s suggested value, based on the above, was made as follows.
The impossibility of performing a field test in truly undrained and undisturbed conditions and the difficulty of obtaining representative undisturbed samples for laboratory testing leaves the results of both methods open to question. Certainly from the behaviour of London Clay in the mass a value of $E_u/C_u$ between the two extremes seems more likely. 

Thus the value of $E_u = 400C_u$ is only inferred as being within a likely range. Butler (1974) also examined the range suggested by back-analysis of structures by Finite Element Analysis after Hooper (1973) of 310 to 480C_u and after Cole and Burland (1972) of 310C_u and the range suggested by analysis of good quality U4 samples undertaken by Ove Arup and Partners (1971) of 222C_u and by the Building Research Station of 234C_u. Work since carried out on small strain measurement of carefully prepared excellent quality laboratory samples (Burland and Symes 1982, and Clayton and Khatrush 1986) suggest that the laboratory measured values are low. A non-linear approach to a back analyzed value of stiffness from field data suggests that a calculation undertaken assuming elasticity on the same field data will also produce a much lower result (Jardine et al 1986). Hence, the value taken by the consultant as being that suggested by Butler may be realistic when taking the above into account rather than it being the mean of a range Butler suggested may be more appropriate as suggested in the quote above taken from his paper (1974).

The program RAFT only accepts direct loadings, that is the piles are required to be modelled as end-bearing. To allow for the piles being a combination of friction bearing and end-bearing the consultant considered two alternatives necessary to generate a reasonable computer model of these piles in terms of equivalent end-bearing piles.

(a) The load in the piles shed at a depth equal to two-thirds the length of piles and the base area artificially
increased to limit stresses at the base of the pile and hence keep settlements to a realistic level.

(b) The load in the pile shed at a depth of two-thirds \(L\) and the actual base area of the pile is used.

An idealised pile raft foundation, shown in Fig.3.5/1, was used by the consultant to investigate the variations in input data shown in Table 3.5/1 below.

A further computer run was made using HECB pile group analysis program 'PGROUP', Banerjee and Driscoll (1977) to check the results of the RAFT/LEAP analysis with respect to the overall load sharing characteristics of the model raft. The distribution of loads between the piles was not expected to correlate between the two programs since this is influenced by raft stiffness. However, the consultant considered the comparison provided a valuable check.

The major differences between RAFT LEAP and P GROUP are:

a) PGROUP analyses the piles as friction piles.

b) PGROUP uses a "rigid" raft for analysis whereas Raft/Leap uses a flexible raft.

As an approximation in the first instance the consultant assumed a raft thickness of 600mm and based the I values on "An Introduction to Grillage Analogy". The soil stiffness input was in two zones, one for raft springs and the other for pile springs. A relatively high \(E_u\) value was used for 1.0m directly below the loaded area as recommended in the RAFT manual. \(E\) values for the two zones are shown in Figs.3.5/2 and 3.5/3. The Pile input data in Table 3.5/2 are used to produce the column loads in Table 3.5/3. The figures for the column loads are
3.5.2. Results from the Piled Raft Model

The results are shown in Table 3.5/4. The Table is taken from the design documentation for the contract and is not instantly obvious. To assist in the understanding the following text endeavours to explain the case for Run A. $E_u$ is 400C_u and the pile area was increased in the analysis. From Fig.3.5/1 it can be seen that there are four types of pile in the idealised pile raft foundation. These are:

Pile type 1) No.s 1,5,17 and 21. These each carried 8.3% of the total load in the analysis,

Pile type 2) No.s 9 and 13. These each carried 7.9% of the total load in the analysis,

Pile type 3) No.s 3 and 19. These each carried 7.6% of the total load in the analysis,

Pile type 4) No. 11. This pile carried 7.4% of the total load in the analysis.

Hence the total load carried by the piles for Run A =

$$4 \times 8.3 + 2 \times 7.9 + 2 \times 7.6 + 1 \times 7.4 = 71.9\%$$

Hence the total load carried by the slab for Run A = 28.1%

3.5.3. Effect of Varying the Elastic Modulus of the Soil

The results from runs A, B and C (Table 3.5/4) show that large changes in the Elastic Modulus result in relatively small changes in the overall load sharing between the piles and the raft. Since the
consultant considered the recommended that a value of $E_u = 400C_u$ was that suggested by Butler for London Clay (1974), this value was adopted in the final analysis of the basements. It is fortunate that the value of elastic modulus proved insensitive considering the range of values Butler had in fact quoted. The value of $E_u = 400C_u$ has been used in the estimation of immediate settlement which required the compressible layer to be sub-divided into zones of constant "average" $E_u$ in the papers that Butler reviewed. After this analysis Butler points out that;

_It should be noted however that in conventional application lower values of $E_u$ are often used and no allowance is made for increasing $E_u$ with depth._

Fortunately it can be seen that the result for $E_u=200C_u$ in Table 3.5/4 provides estimates of load sharing similar to those for $400C_u$. The more common value used in practice is $E_u=220C_u$.

### 3.5.4. Effect of How the Piles are Modelled

The results from runs A, D and PGROUP (Table 3.5/4) are compared bearing in mind that;

a) the load sharing between the piles and the raft is primarily a function of the pile size and spacing, and

b) the distribution of load between the individual piles is a function of raft stiffness.

The results show that for a flexible raft (runs A and D) the load is fairly evenly distributed between the piles whereas for the rigid slab (PGROUP) the perimeter piles carry considerably more load than the
centre piles. The consultant stated that was as expected from (b) above (Hain and Lee (1978). The PGROUP analysis considers the actual length, diameter and spacing of the piles and therefore was considered to give a more accurate assessment of the load sharing between piles and raft. Analyzing the results of runs A and D with those of PGROUP, the consultant considered that modelling of the piles with an effective length of 2/3L and using the actual base area, run D gave a more accurate assessment of the load sharing than was given by any artificially increased base area.

Average values of immediate (elastic) settlement of the raft for runs A, D and PGROUP were 12.5, 17.6 and 25.3mm respectively. Settlement of the raft pile system obtained from the RAFT LEAP and PGROUP analyses would be expected to be similar if the equivalent end-bearing piles in the RAFT program were a reasonable computer model of the actual friction piles. The settlement obtained from run D is most comparable to that obtained from the PGROUP analysis, although a substantial difference in actual values was noted. This difference is best explained by considering the way that the soil is modelled in the two programs.

The RAFT program models the soil as having an elastic modulus increasing linearly with depth, which gives a close approximation to the real soil.

The PGROUP program models the soil as having an elastic modulus remaining constant with depth.

Although a value of $E_u = 400C_u$ has been used for both programs the actual value used in PGROUP was taken as the average value over the length of the pile, therefore, the $E_u$ value for the soil below the pile is considerably less than that used in RAFT.
The settlement obtained from the PGROUP analysis would therefore be expected to be greater than that obtained from RAFT because the soil at lower depths in the former is less stiff than in the latter. It was thus concluded by the consultant that a reasonable computer model of a friction pile in terms of an equivalent end-bearing one was best obtained by inputting an effective length equal to 2/3 the actual length with the base area of the pile equal to the actual area.

The consultant then used this computer model in the final analysis of the basements.

Care must be taken in assessing the methods of analysis. Reddaway and Elson (1982), when analyzing a piled bridge abutment at Newhaven, found that no one method of analysis was successful for all the pile load cases considered. They concluded that PGROUP was most successful, but a simple static approach was also quite successful in estimating the pile loads. Thus whilst the designer considered that the true pile dimensions being included in the PGROUP analysis was advantageous, his conclusion that this resulted in a more accurate assessment of the load sharing between the pile and the raft is not sound. Added to this a rigid foundation will load the outer and corner pile more than the inner piles (Cooke et al 1980, Cooke 1986). However a non-linear stress-strain approach would suggest that linear elastic theory will tend to over predict group settlement ratios and to exaggerate the non-uniformity of loads within the rigidly capped groups (Jardine et al 1986). They suggest that in assessing the interaction it is necessary to consider the initial response of the soil shearing with the full accuracy afforded by the new laboratory techniques described by Burland and Symes (1982) and Clayton and Khatrush (1986). Jardine et al (1986) concluded that, at working loads, small strain non-linearity and consideration of local failure have important implications in considering soil-structure interaction. Yet, despite this, others (Chow 1989, Poulos 1989) suggest that elasticity is an acceptable basis for working loads. PGROUP, as has
been pointed out in the text above, considers the raft to be rigid and with a pile to diameter spacing ratio of about 10, and the raft being relatively thin, the raft is not rigid. The consultants conclusion that modelling of the pile with an effective length of 2/3L and using the actual base area gave a better assessment of load sharing is questionable. Cheung et al (1988) stated that the plate spring approach predicted the load distribution for a flexible pile cap fairly well but the results were similar to those obtained by treating the piles as independent rigid supports. Cheung (1988) however concluded that empirical rules, whilst adequate for a rigid pile cap, need extreme care when being applied to a flexible pile cap.

Providing a simple but sound method to indicate the likely pile loads can be difficult. Poulos (1989) indicates that elasticity, whilst simple, provides a useful basis for the prediction of pile behaviour provided that the appropriate elastic parameters are selected for the soil. Elastic solutions are still considered acceptable in the majority of cases (Chow 1989, Poulos 1989) at normal working loads (40 to 50% of ultimate) despite the work at Imperial College (Chandler and Martin 1982, Jardine et al 1986, Jardine 1985) which shows that small strain non-linearities exist.

The results of any analysis must be looked at in sound engineering terms. It matters little which method gives the better load sharing characteristics, from which the pile and slab are to be designed if the design worst case loading conditions are not recognised. The loading conditions suggested by the consultant were sufficiently different from those associated with a conventional design to consider the output from any one method to be better than another. The results must be looked at, in engineering terms, to assess if they are of the correct magnitude. The particular problem with the design of the Alders basement was that the loads were placed onto the columns. These columns are sited over the piles. For the slab to take the construction loads the piles must be mobilised and the slab will then act as a restraining membrane. Equally if full water pressure and
heave pressure are present then the slab will dome between the column positions before the piles restrain further movement.

The difficulties in making allowance for the load sharing are still not covered well in the literature. Many of the methods are for the group effect of free standing piles but with the pile cap rigid and making no contact with the soil surface (Douglas and Butterfield 1984). Unless detailed to do otherwise the pile cap will transmit some load directly onto the loaded strata so that for a given settlement the support is greater than that provided by the piles alone. The cap increases support, or reduces settlement, by about 8% under working load conditions (Cooke et al 1980). Interaction factors for pile groups for soils should be used carefully. Those theoretically based on a soil having uniform stiffness should not be used for designs in London Clay or other soils which exhibit an increase in stiffness with depth (Cooke et al 1980).

Cooke (1986) suggests several ways in which the piled raft may be designed to take into account the load sharing capabilities of the pile and the raft which in principle agree with those outlined by the geotechnical consultant in 1982. Cooke’s two principal suggestions for an economic design are;

1) to design the piles to carry the working load. The factor of safety on the piles alone would then be unity while the true factor of safety would be provided by the available bearing capacity of the raft,

2) to stipulate an acceptable settlement and increase the number of piles only until that settlement value is attained.

It has further been suggested that where piles are included to reduce the settlement no further reduction is achieved once a pile spacing of
four diameters is achieved (Cooke 1986) although model tests suggest that this may be as high as six to eight pile diameters.

Without inserting intermediate piles, the location of the piles is determined by the column locations. The designer has attempted to reduce the thickness of the slab to implement his saving through load sharing without any study of the implications the pile size may have on further savings. In the past when piled raft foundations were designed for the piles to take the full load and the slab to take full uplift pressures the true factors of safety were well in advance of those built into design because the loads are shared. With the design taking into account load sharing the designer's in-built extra margin of error is reduced and it is now necessary that he considers his structure carefully and considers that he has designed for the worst load cases and not those probable from case histories. Further to this the implications of his material savings must be looked at in the context of the method of construction. It may be that the design will mean a more costly and or time consuming construction technique which may negate any material cost saving. If the design implies any special construction considerations away from normal practice these must be spelt out to the contractor.

The Alders design was still complicated as the weight of the structure is less than the weight of soil removed. There were no case records to indicate possible short or long term behaviour. The difficulties were realised and a great deal of discussion between the designers and the geotechnical consultant was entered into to ensure that the worst possible load cases were designed for.

3.5.5. Effect of Long Term Consolidation Settlement

Theoretical analysis and field measurements have shown the effect of consolidation settlements is to increase the load taken by the piles and reduce that taken by the raft. Hooper (1974) obtained, through
calculation and field measurement, a value of 6% of the total building weight as an increase in the pile loads. To allow for the effects of consolidation settlement the pile loads obtained from the final analysis of the basement were increased accordingly by the consultant.

The consultant assumed that the piles carry 65% of the applied load
the pile load was increased by

\[(0.65 + 0.06 - 1) \times 100 = 9.2\% \text{ say } 10\%.
0.65\]

3.5.6. **Heave Pressures**

The actual heave pressures to which the basement slabs were assessed to be subject were considered a function of the following:

1) a proportion of elastic to plastic heave of the soil.

2) the stress history of the soil.

3) the skin friction generated on piles as clay tries to heave upwards.

All these points are difficult to assess precisely. However, the designer used guidance from past case histories and the soil report led to him using the following design conclusions which he illustrated in Figs. 3.3/3 to 3.3/8.

1) **Maximum possible upward pressure is equal to the depth of soil removed x soil bulk density (18kN/m³). Minimum possible upward pressure is equal to the water pressure.**

2) **At the time of excavation, elastic heave is eliminated.**
3) Long term heave on the slabs reduced by skin friction on the piles.

4) A proportion of the original consolidation is irreversible due to the stress history of the soil.

This led to the following design values.

1) Upward heave pressure to which the piles may be subject, minus the minimum weight of the building, taken as 85% of the maximum possible heave.

2) Upward heave pressure to which the slab may be subject, minus the minimum weight of the basement slab, alone, taken as 70% of the maximum heave pressure (NB this ignores any load sharing between column and slab which would tend to reduce the out of balance further).

3) For ultimate reinforced concrete design analysis the values computed above (ie working loads) were multiplied by ultimate load factors which gave ultimate design pressures in excess of the maximum possible pressures. For the pile design this value was 1.33.

At the time of tender the consultant used "shared load" analysis was for the basements which led to the following conclusions being drawn at that stage.

Unit Shops Basement

of Column Load P

0.70P onto the pile

0.30P onto the slab
Alders Basement

of Column Load P

0.90P onto corner wall piles
0.80P onto mid-span wall piles
0.70P onto mid-basement piles

The consultant considered that a more precise breakdown on Alders was justified due to its more regular shape.

3.5.7. Design Calculation

Having been awarded the contract the consultant decided that, in the light of the tender analysis, two main design conditions were to be analyzed. These were:

i) Short term: maximum building loads, allowing for load sharing between the piles and the raft, but ignoring uplift due to water pressure and heave effects.

ii) Long term: maximum uplift forces with minimum building loads (ie no live loads).

For the short term case the soil-structure interaction analysis was undertaken using the grillage shown in Fig.3.5/4. As the instrumentation was limited to the quarter of the building indicated in Figs 3.5/5 and 3.5/6 all further design predictions will be limited to this quadrant.

In one part of the analysis the basement slab was idealised as a grillage of beams and the soil was modelled as an arrangement of discrete springs at the grillage node points. Initially the spring
stiffness were assigned from experience. In the second part of the analysis the Mindlin Solution (Mindlin 1936) was applied to calculate the settlement of an array of point loads on and within an elastic solid. The loads were obtained from the first part of the analysis and the soil was modelled as a homogeneous isotropic elastic half-space, where Young’s Modulus increased linearly with depth ($E_u = 400C_u$ and Poisson’s Ratio 0.5). The piles were simulated by applying the load to the "soil" at two thirds the pile depth below the basement slab. From the calculated vertical displacements and applied loads, new spring stiffness were obtained for use in the first part of the analysis. Iterations were made between the two analysis until the spring stiffness converged.

Fig.3.5/5 shows predicted settlements for the short term case of up to 20mm with a maximum differential settlement of 9mm and indicates that in the centre of the slab 50% of the building loads should be taken by the piles.

For the long term case the uplift pressures were assessed as follows. In practice some of the uplift pressure is dissipated as immediate elastic heave before the slab is cast and it was estimated that 15% could be eliminated in this way. The piles were therefore designed for 85% of the overburden pressure less the minimum weight of the building; that is the basic dead weight discounting partitions, live loading etc, further reduced by a factor 0.9. Based on these assumptions, a maximum tension per pile of 4100kN was calculated. The piles themselves would act as reinforcement in the soil so the skin friction would tend to reduce the heave pressure on the slab. It was thus estimated that this could count for a further 15% of the heave pressure, and the slab was therefore designed for 70% of the overburden pressure.

The maximum pile loads, in both tension and compression obtained as a result of the analyses are shown in Fig.3.5/6.
The design of the substructure thus consisted of 700mm thick reinforced concrete basement slab, which was thickened in the area of the piles to 900mm. Internally directly under each column location a single bored cast-insitu pile 1050mm diameter, 11m long and underreamed to 3150mm was placed. The walls to the basement were constructed integrally with basement slab and the ground floor slab to form a box structure. The walls were generally 600mm thick and column positions were supported on similar piles, as were the partition walls to the stairwells and elevator shaft. These piles again 11m long but were 900 and 750mm diameter with smaller under-reams.
Table 3.5/1 Variation in Model Input Data

<table>
<thead>
<tr>
<th>RUN</th>
<th>UNDRAINED ELASTIC MODULUS</th>
<th>PILE MODEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$E_u = 400C_u$</td>
<td>Base Area Increased</td>
</tr>
<tr>
<td>B</td>
<td>$E_u = 200C_u$</td>
<td>&quot;</td>
</tr>
<tr>
<td>C</td>
<td>$E_u = 600C_u$</td>
<td>&quot;</td>
</tr>
<tr>
<td>D</td>
<td>$E_u = 400C_u$</td>
<td>Actual Base Area</td>
</tr>
</tbody>
</table>

Table 3.5/2 Pile Input Data

<table>
<thead>
<tr>
<th>Pile No.s</th>
<th>Shaft Diameter (m)</th>
<th>Base Diameter (m)</th>
<th>Length (m)</th>
<th>Working Load (kN)</th>
<th>Actual Base Area (m²)</th>
<th>Increased Base Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,5, 17, 21</td>
<td>1.2</td>
<td>1.2</td>
<td>24</td>
<td>5323</td>
<td>1.13</td>
<td>8.2</td>
</tr>
<tr>
<td>3, 9, 13, 19</td>
<td>1.35</td>
<td>1.35</td>
<td>24</td>
<td>6173</td>
<td>1.43</td>
<td>9.5</td>
</tr>
<tr>
<td>11</td>
<td>1.35</td>
<td>2.7</td>
<td>24</td>
<td>7405</td>
<td>5.73</td>
<td>11.4</td>
</tr>
</tbody>
</table>

Stress Limit for Increased Base Area = 650kN/m².
### Table 3.5/3 Column Loads

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>Pile Load (kN)</th>
<th>Column Load Pile Load/0.7 (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 5, 17, 21</td>
<td>5300</td>
<td>7500kN (say)</td>
</tr>
<tr>
<td>3, 9, 13, 19</td>
<td>6150</td>
<td>8750kN (say)</td>
</tr>
<tr>
<td>11</td>
<td>7400</td>
<td>10500kN (say)</td>
</tr>
<tr>
<td><strong>Total Applied Load</strong></td>
<td><strong>75500kN</strong></td>
<td></td>
</tr>
</tbody>
</table>

### Table 3.5/4 Results for the Load Sharing Analysis

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>Run A</th>
<th>Run B</th>
<th>Run C</th>
<th>Run D</th>
<th>PGROUP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Eu</td>
<td>400Cu</td>
<td>200Cu</td>
<td>600Cu</td>
<td>400Cu</td>
</tr>
<tr>
<td>Pile Area</td>
<td>inc</td>
<td>inc</td>
<td>inc</td>
<td>actual</td>
<td></td>
</tr>
<tr>
<td>Z load taken by individual piles</td>
<td>8.3</td>
<td>8.0</td>
<td>8.6</td>
<td>7.8</td>
<td>8.8</td>
</tr>
<tr>
<td>1, 5, 17, 21</td>
<td>9.13</td>
<td>7.9</td>
<td>7.5</td>
<td>8.4</td>
<td>7.0</td>
</tr>
<tr>
<td>3, 19</td>
<td>7.6</td>
<td>7.2</td>
<td>8.1</td>
<td>6.6</td>
<td>6.6</td>
</tr>
<tr>
<td>11</td>
<td>7.4</td>
<td>7.9</td>
<td>7.8</td>
<td>7.2</td>
<td>3.0</td>
</tr>
<tr>
<td><strong>Overall Load</strong></td>
<td><strong>Z Piles</strong></td>
<td>71.9</td>
<td>69.3</td>
<td>75.3</td>
<td>65.8</td>
</tr>
<tr>
<td><strong>Sharing</strong></td>
<td><strong>Z Raft</strong></td>
<td>28.1</td>
<td>30.7</td>
<td>24.7</td>
<td>34.2</td>
</tr>
</tbody>
</table>

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Figure 3.5/1 Computer Model to Investigate Variations in Input Data
Figure 3.5/2 Initial E values Zone 1 Raft Springs

IE = 400 x cohesion

Extract from Soil Report after Ground Engineering Ltd.
Spring Stiffness values after White Young & Partners
Undrained Shear Strength (kN/m²)

Layer 1
Layer 2
Layer 3
Layer 4
Layer 5
Layer 6

IE = 400 x cohesion

Extract from Soil Report after Ground Engineering Ltd.

Pile Spring Stiffness values after White Young & Partners

Figure 3.5/3 Initial E values Zone 2 Pile Springs
Figure 3.5/4  Design Grillage Alders Basement
Figure 3.5/5 Predicted Settlement and Predicted Pile Load () as a Percentage of Column Load

Figure 3.5/6 Pile Design Loads (negative is tension)
3.6 Scope of the Instrumentation

A detailed proposal of the installation of monitoring cells was submitted by the consultant to the client, Basildon Development Corporation.

Objectives

The objectives of the installation of the load monitoring equipment were summarised by the consultant in a document to the Basildon Development Corporation as follows:

(1) A check on heave pressures in the short term condition. For any reason, should an extended delay in the construction programme have occurred, then the cells would have indicated whether kentledge would have been required to limit the tensile stresses in the piles until such time as the building weights increased.

(2) Provide valuable design data on the behaviour of piled raft foundations. It was considered that not only would this be of great interest to the industry, but should gain valuable additional recognition and publicity for the development.

(3) The additional complications of design and detail for the basement slabs in this method of construction were considered more than off-set by the financial savings in the piling. An increase in the number of documented cases should lead to its greater use.
Documented Case Histories

It was pointed out to the client that the development was either the fifth or sixth major project to utilise the design method, Hyde Park Cavalry Barracks, Victoria Street Development, Dashwood House and the National Westminster Tower being the best documented. At the time of the proposal to install instrumentation it was pointed out that the monitoring cells proposed for the development were identical to those installed at the new Conference Centre, Chancery Lane, by the Building Research Establishment.

Technology

The state of the art was summarised in the consultants proposal document to the client as follows:

One of the great disadvantages of pressure/load monitoring carried out to date had been the unreliability of the monitoring equipment itself (Hooper 1979, Furley and Curtis 1981). The Building Research Establishment had at this time just developed much improved cells for use under slabs and within piles (Price and Wardle 1983).

The slab cells are approximately 600mm diameter. They were buried within the slab blinding, with the top face coming directly into contact with the uninterrupted/disturbed slab surface above. The connecting cables were proposed to be laid within a sealed conduit, also laid in the binding, passing up through the slab in one agreed location. These cables would then be linked to a datalogger located in a suitable position within the basement.

A pile cell takes the form of a set of individual cells linked to each bar of a special EN24T steel reinforcement cage located in the top of the pile. These cages could be positioned by the Piling Contractor. Should so called 'collapse' of a cell occur, a traditional misgiving
of pile cells, then the improved design would result in a maximum settlement of 3mm. The tensile capacity is greater than adjacent pile reinforcement, assuming the use of high yield reinforcement. Leads would again be sealed in similar conduit to that of the slab cells.

**Contractual Implications**

All items were proposed to be billed in the contract documents under the foundation contract and piling contract where appropriate. The consultant had taken the opportunity to monitor the installation works on the Chancery Lane site and had witnessed no undue problems. The architectural implications on the works would be the requirement of a small space within the basement area with restricted access.

**Slab Cells**

The slab cells were as described by Wardle and Price 1983. They are approximately 600mm diameter and consist of 3 sensing units of the vibrating wire type. Water is allowed access behind the measuring plate, around the sensing elements, so that the cells are insensitive to water pressure and thus measure effective stresses. These cells were developed at the Building Research Station. The positioning of the slab cells is shown in Fig.3.6/1.

**Pile Cells**

The pile cells were as described by Wardle and Price 1983. They consisted of 12 sensing units which were constructed in the top of the pile such that they formed an integral part of the reinforcement cage. The sensing units were of the vibrating wire type. The cells were developed at the Building Research Station. The position of the pile load cells is shown in Fig.3.6/1.
Water Pressure Measurement

A system of pipe work connected the slab cells to each other. This facility was utilised by taking the pipe work to a Bourdon gauge at the readout station. This enabled an evaluation of the average water pressure under the monitored quadrant of the basement slab to be made.

Settlement Stations

Settlement stations were built into the floor slabs as the construction progressed. These were of the BRE type (Chenny 1974). Instead of using the bolt, as would be the case if the studs were horizontal in a wall, it was proposed that a stainless steel ball-bearing would be located in the top of the socket. This was to avoid ensuring that the threaded socket was clean so that the bolt could locate accurately, as it would be impractical to keep the floor area clean. The location of these stations is shown in Fig.3.6/2.

As the walls and columns were constructed, studs were included at a preset height. As the surveying was to take place in a basement where wind would not be a problem, a small rule was hung from the stud. Thus the studs were set at a height that they could all be read from a common collimation for the instrument. The location of the wall and column stations is shown in Fig.3.6/2.

Two deep datums were installed away from the influence of the shopping complex building to maintain a cross check on the initial readings. A permanent facility was also provided to transfer the level from ground level to the basement via the stairwell.
Precise Levelling Procedures

The precise levelling was undertaken by the BRE. The levelling was undertaken between two deep datums. The procedure was to level from the first datum, around the basement via a permanently positioned aid in the stairwell, to the second datum. Levelling was undertaken from one deep datum to the other to check any differential settlement between the two. Having entered the basement the level positions had been selected so that it was possible to undertake the exercise with the minimum possible number of changes. The column and wall stations had been installed at the same height above ground level. The stations were studs and not the BRE settlement sockets. These allowed the use of a rule hung from the stud. As there was no trouble with wind or illumination this allowed the levelling to be undertaken without the problems of verticality associated with a level staff. The levelling undertaken on the floor slabs used ball bearings set into the top of the BRE sockets. The use of the ball bearings instead of the settlement bolts ensured that the ball could be seated accurately in the recess. The bolt could not be seated easily as this would require thorough cleaning out of all material from the socket to ensure that there was a proper fit. The floor slab locations were quickly covered with building materials and were unreadable.

Slab Cells and Pile Cells

Both the slab cells and the pile cells were installed by the BRE and were read by the site staff of White Young Consulting Engineers. The readings were taken once a week and at regular intervals analyzed by the Building Research Establishment's staff. At the same time the reading on the Bourdon gauge was taken to measure the average water pressure under the quarter of the slab monitored by the slab cells.
Figure 3.6/1 Position of the Pile Load Cells • and the Slab Load Cells X

F = Floor Station
W = Wall Station
HP = Transfer Point from Ground to Basement Level

Figure 3.6/2 Position of the Precise Levelling Stations
3.7. **Analysis of the Data**

Interpretation of the Basildon data is limited as only a small proportion of the data has been analyzed and published by the Building Research Establishment. This includes the pile and raft cell data, which whilst owned by the Building Research Establishment, the consultant, the Basildon Development Corporation and the University of Surrey, has yet not been released in analyzed form to the other parties. The limited data available to date will be commented on in the following text.

3.7.1 **Analysis of the Raft Cell Data**

The position and last known readings for the raft cells are shown in Fig.3.7/1. The time histories of the raft cells are shown in Figs.3.7/2 to 3.7/16 inclusive.

It is quite readily obvious from looking at the above figures that the raft cells are not responding as they were designed too as it is not possible for negative effective stress to exist. The obvious solution is that the slab cells have malfunctioned but there are a number of pointers against this:

a) slab cells of a similar design have always worked well,

b) slab cells of exactly the same design have worked well situated on London Clay,

c) the cells, having had the slab poured over them were all capable of detecting the weight of a man.

Examination of the site diary gives the first insight for the cells reading negative effective stress due to the amount of water present during the construction phase of the contract.
a) Even before the basement walls were complete the
evacuation was being continually pumped and tension was
evident in all the raft cells.

b) In early February 1983 the basement walls were complete.

c) In early March the backfilling was complete.

d) Despite backfilling not being complete leaks were reported
in late February.

e) In early March the basement was reported as very wet with
little progress being made to dry out and repair damp
patches.

f) Snagging reported right through 1983, early December
reported 50% leaks treated but not all repairs successful.

g) March 1984, reports that water continues to leak through.

h) Again full details of readings of the Bourdon Gauge are
not available, but the one value reported by Rickard et al
(1985) indicates a head of 4.8m of water above the
underside of the slab.

It would appear, as has been intimated elsewhere, that there was a
substantial amount of water continually available at this location.
It was reported by the site staff that the walls to the excavation
were dry and yet the excavation required quite substantial pumping at
times. The water can only have been coming from below whether the
source was man made in the form of an unmarked water main or a natural
occurrence in the London Clay.
It has been suggested that the raft cell gauges themselves may be a party to the presence of water under the slab. The cells were interconnected with piping which led to the Bourdon Gauge so that the average water pressure under the quadrant could be monitored. It might then be conceived that if water were present in one location it would, more rapidly than normal, gain access to the other gauges. This may well have been the case in the monitored quadrant, however, the site diary shows the leak problem to be all over the basement and not just limited to that quadrant with instrumentation present.

Having perceived that there is a substantial amount of water present the fact that the effective stress cells are reading negative is still not explained. However with regard to the Fig.3.7/17. It has been suggested in conjunction with the Building Research Establishment that the bitumastic paint may be sticking to the top of the cell to the slab and that the cement bed is adhering to the bottom of the cell to the clay. This implies that water pressure is causing heave on the underside of the slab and for the clay to be holding the bottom of the cell down the clay must be in tension. This may well be feasible in the short term but as the pressure in the clay dissipates this would only leave the weak inter-particle forces in the clay to maintain the tension. The inter-particle forces cannot maintain tension of this magnitude and the pore pressure dissipation would not allow the cells to show negative effective stress for the periods indicated in Figs.3.7/2 to 3.7/16. Further thought has led the writer to believe that the cells may be weighing themselves if they are adhered to the underside of the slab by the bitumastic paint and there is sufficient water pressure for the slabs to dome so that the bottom of the cell breaks contact with the clay. The following calculations were made with reference to the site drawings and Price & Wardle (1983) although it must be said that no base plate dimensions were available for the cells. From Fig.3.7/17 it can be seen that the cell consisted of a base plate with three pillars and a precast concrete top unit. The concrete top unit is separated from the pillars and the base plate by
a 3mm soft membrane. The gauges were zeroed when positioned before the ground slab was cast and hence the weight of the precast concrete top unit was zeroed out.

If the cell were weighing itself, i.e. hanging from the underside of the slab the minimum it would be weighing would be the base plate plus the weight of the top cap unit which had been zeroed out. The only unknown is the thickness of the base plate but an extremely conservative value of 12mm has been taken.

The area of a 600mm diameter plate = 0.283m²
Taking a density for steel of 8Mg/m³ the weight of a 600mm diameter 12mm thick plate = 27.14kg =0.266kN

The weight of the concrete top unit taking the diameter as 600mm and the depth as 250mm less 12mm for the base plate less 3mm for the soft membrane and allowing a density of concrete of 2.4tonne/m³
Area of base plate (250-12-3)x2.4x9.81kN =0.283(250-12-3)x2.4x9.81kN =1.564kN

Thus the minimum total negative weight the sensing units would weigh if the cell were stuck to the underside of the slab would be 1.564+0.266 = 1.830kN

This represents an apparent negative effective stress of 1.830
0.283

=6.47kN/m²

This figure would be increased by a further apparent 0.5kN/m² negative effective stress for every 20mm of cement bed (Fig.3.7/17) that was adhered to the underside of the bottom base plate. Reviewing
Figs. 3.7/2 to 3.7/16 this value would account for all the cells that indicate negative effective stress apart from RC5 (-14.7 kN/m²), Figs. 3.7/1 and 3.7/6, and RC6 (-24.2 kN/m²), Figs. 3.7/1 and 3.7/7 for which only zero drift can be offered as an explanation.

The relationship between the above explanation and the values under the slab must now be looked at in the context of the boundary conditions along the edges of each grid between the pile locations and the amount of heave present under each grid. Any heave that has taken place since construction will be at a maximum at the centre of the slab and will tail off towards zero at the edge of the slab. Doming of the slab due to water pressure will be dependent upon the restraint, thus the least domed slab would be the corner slab. Then all the edge slabs would have one edge restrained due to connectivity with the basement wall and the most domed slabs would be those centrally located where they are only partially restrained in the corner positions by the pile and column locations.

Thus the following might be expected

a) The corner grid restrained by the basement wall on two sides would have least doming due to water pressure and least heave. Add to this the further restraint of the stairwell wall and the piles under the wall would further reduce the doming and transmit load to the location and a positive effective stress would be indicated. Indeed apart from the initial 50 days when just the weight of the slab was present and the excavation was being continually pumped this is the only slab cell (RC1 Figs. 3.6/1 and 3.7/1) to show a build up and a maintained measured pressure. The initial values of negative effective stress for the first 50 days are of the order of 5 to 10 kN/m² (Fig. 3.7/2) which would suggest that even at this early stage the water pressure was such that the slab had domed and the cell was
weighing itself. Prior to scaffolding and construction of the ground floor.

b) Those grids where one edge is restrained by the basement wall would have more doming than the corner situation and would have more heave (Figs. 3.6/1 and 3.7/1). Dependent upon the ratio of these two effects the slab cells might be weighing themselves but could all be showing a pressure. Indeed slab cells RC2 (Fig. 3.7/3), RC3 (Fig. 3.7/4), RC4 (Fig. 3.7/5) & RC12 (Fig. 3.7/13) are all now showing a small value of effective stress. All these gauges again show a value of -5 to -10kN/m² suggesting they are weighing themselves prior to the scaffold being installed for the ground floor slab.

c) Those central grids (Figs. 3.6/1 and 3.7/1) where there is no edge restraint and hence maximum doming but where heave would be a maximum would as in b) above, depending upon the ratio of heave to doming, be likely to show self weighing. Indeed all the internal raft cells (RC7 Fig. 3.7/8, RC8 Fig. 3.7/9, RC10 Fig. 3.7/11, RC11 Fig. 3.7/12, RC13 Fig. 3.7/14, RC14 Fig. 3.7/15 and RC15 Fig. 3.7/16) show very small positive or negative effective stress values indicating low pressure contact, partial separation or suspension of the cell from the slab indicating separation from base contact with the clay. Initially all the slab cells show -5 to -10kN/m² until the scaffold is erected. The two exceptions to this are the edge restrained RC5 (Fig. 3.7/6) and the internal RC6 (Fig. 3.7/7) which both show large negative effective stress values.

All the raft cells show a self weight tendency until the scaffold is erected and from this time they all show effective stresses as the construction dead loads increase transmitted through the scaffolding to the basement floor slab. It must be remembered that the design condition was for the loads to be
transmitted through the columns to the piles and that at this time the likely situation was; the majority of the construction dead loads to the slab and a minimal amount the columns, apparent maximum water pressure and probable maximum heave. It is not surprising that the internal piles (Fig.3.7/21) show tension during this period and that the raft cells show contact pressures. With time the raft cells appear to be showing a shedding of the contact stress and the piles show less tension.

The majority of the raft cells would indicate a major shedding of effective stress at about day 300. This does not coincide with the stripping out of the basement scaffold which the site diary indicates took place at about day 390. However it could coincide with the removal of the formwork to use elsewhere, with a redistribution taking place despite repacking on the spreaders or purlins. However the site diary does not indicate that this procedure took place or indeed that if it did it took place at about day 300.

3.7.2. Analysis of the Pile Cell Data

The position of the four monitored piles and the last known readings of the pile cells is shown in Figs.3.6/1 and 3.7/1. The time histories of the pile cells are shown in Figs.3.7/18 to 3.7/21 with Figs.3.7/20 and 3.7/21 having the construction history for that locality superimposed.

With regard to Fig.3.5/6 showing the pile design loads Fig.3.6/1 showing the location of the piles and Figs.3.7/20 and 3.7/21 showing the time histories of the measured load the following is noted.

a) Figs.3.7/20 and 3.7/21 readily show that there is little or no construction load pick-up directly associated with any specific construction process in the locality of the pile. As was stated in the analysis of the raft cells this is primarily due
to the method of construction which took the loads through the basement raft slab, and not as the design allowed for primarily through the columns to the piles.

b) All the piles are showing a response towards the long term design condition even though still in the construction phase. This must be associated with the presence of the water. It is again most unfortunate that a time record of the water pressure in not yet available.

c) Whilst pile cells B, C and D have all remained well within the design tolerances it is evident that, on a number of occasions PCA has exceeded the design maximum tensile force. Indeed it is only as a result of monitoring this pile cell that the magnitude of the tensile force was realised and action taken to pump out the locality and to apply kentledge.

3.7.3. Discussion on the Raft and Pile Cell Data

For PCB, C & D, despite the fact that it is thought that the long term condition was present but with only partial construction loads and not full dead load, the design is still shown to be very conservative. All the loads are a quarter or less of the predicted design loads. For PCA however the maximum design tensile load has been exceeded. This would indicate that the model is not predicting the distribution into this pile correctly and allowance must be made. It would appear that the compressive load is such that there should be sufficient steel in the pile for the compressive load requirement to more than adequately cope with the additional tensile force shown in PCA. However in the absence of these specific calculations it was still prudent to pump the excavation and add additional load to relieve the excess tensile load.
The necessity of the complete slab levelling and Bourdon gauge data analysis is essential for a better assessment of the events. This data would enable a more confident interpretation of the raft cell readings when taken in conjunction with the column levelling data, the pile load cells and the construction diary.
Figure 3.7/1  Pile Cell and Slab Cell Readings in 1984
Figure 3.7/2 Raft Pressure for Position RC1
Figure 3.7/3 Raft Pressure for Position RC22
Figure 3.7/5 Raft Pressure for Position RC4
Figure 3.7/6 Raft Pressure for Position RC5
Figure 3.7/7 Raft Pressure for Position RC6
Figure 3.7/8 Raft Pressure for Position RC7
Figure 3.7/9 Raft Pressure for Position RC8
Figure 3.7/10  Raft Pressure for Position RC9
Figure 3.7/11 Raft Pressure for Position RC10
Figure 3.7/12 Raft Pressure for Position RC11
Figure 3.7/13 Raft Pressure for Position RC12
Figure 3.7/14 Raft Pressure for Position RC13
Figure 3.7/15 Raft Pressure for Position RC14
Figure 3.7/16 Raft Pressure for Position RC15
Figure 3.7/17 Section Through a Raft Cell
Figure 3.7/18 Pile Load for the Corner Pile PA
Figure 3.7/19 Pile Load for the Edge of Basement Pile PB
Figure 3.7/21 Pile Loads for Internal Piles PC and PD
3.8 Discussion

Site Investigation

A site investigation typical of that expected when dealing with such a universally known material as the London Clay was carried out. This case record again highlights the dangers of such an assumption. The usual "appropriate" number of boreholes were dotted about the site in the locality of the major structures with six boreholes having ground water measurements taken. Whether the piezometers were established long enough to equilibrate with the ground water conditions and whether they were read enough must be questioned. These and other shortcomings in the site investigation are commonly as a result of the economic restraint and the appearance of giving value for money to the client in the presentation of the document. Only one borehole was located in the Alders basement and this was not monitored for ground water. The report itself highlights that for a successful design the water table in this region should be determined by at least two piezometers but in its absence suggested that ground water should be assumed at the winter reading of 1m below ground level. In hindsight the suggestion of taking the water level to be 1m below ground level was prudent in view of the water problem encountered during construction. However, ground water at the locality of a proposed deep basement should have been monitored in the original site investigation, and not recommended as an afterthought.

The requirements a site investigation must achieve need to be appreciated before the investigation. In an ideal world all the parameters would be deemed necessary such that any design eventuality could be met subsequent to the investigation without the need for further field or laboratory works. In reality the bounds of behaviour of such a material as the London Clay are well known and in the majority of cases the above requirement and cost are excessive. If however a building of some prestige and novelty in its design concept
is to be put forward then the site investigation should also reflect this approach and should command at least as much forethought. In this way a design may be based on the prevailing conditions and not those thought to be the worst case.

**Method of Design**

The principles behind the design and the design loading cases were well prepared prior to any calculation, especially with respect to how load sharing would take place. The advantages of the LEAP/RAFT system to the design engineer were that as interaction took place a feeling for how the loads were distributing was gained as the spring stiffnesses converged. The method also allowed the designer to apply the load through a column location directly onto a spring positioned at depth where a pile would act. To be able to achieve this three dimensional feel for the structure by finite element analysis would have been far more costly and complicated, beyond the resources for a structure of this size, and would have resulted in answers requiring equal engineering judgement as to their soundness. The use of the correct soil parameters has been emphasized throughout the literature (Jardine et al 1986, Cooke 1986, Cheung et al 1986, Poulos 1989) and yet a soil stiffness in excess of that generally recommended was used. It is fortunate that the design method was relatively insensitive to changes in soil stiffness and that the calculation undertaken with a soil stiffness value of $E_u=200C_u$ yielded similar results to that undertaken for $E_u=400C_u$ (Table 3.5/4).

**Construction of Alders Basement**

The method of design imposed certain construction requirements. The design of the basement walls as propped cantilevers meant that the area should be continually pumped and that no backfilling could take place until the basement and ground floor were completed forming a stable box construction.
With such a large basement it is difficult to comply specifically with the above requirements of no backfilling until the ground floor is fully complete; thus the design must also consider the construction methods available. To have such a large area open during construction provides access and safety problems. The design of the basement should specifically allow backfill of the walls in sections along the length of the basement as the floor slab is constructed. As the ground floor slab is completed across the width of the basement it must be good practice to follow the construction up by backfilling the basement wall. This procedure is safer than leaving a large excavation open until the basement box structure is completed and closed before backfilling. On a busy construction site access is continually required to the main building works for construction to progress. This must be safer and easier than providing access over the open excavation. To close the excavation as each floor bay was completed would require the design to allow for backfilling against a box section and not the complete box structure of the entire basement as implied by the design.

In view of the wealth of data available for the London Clay and the available literature and case records mentioned for pile raft interaction the short and long term design principles were reasonable. It would appear however that the long term water conditions were met close to the end of construction and hence not all the design building loads were present to counteract the maximum assumed heave and water pressure. The tensile loads designed for the piles must reflect this situation as a worst case and not assume that this state of equilibrium will be met well after construction and internal fitting. To this end the design must take account of construction methods and the worst possible situation, no matter how short term, be considered. In certain circumstances where the design is particularly novel the contract documents should specify the construction procedure to avoid temporary unstable states not allowed for in the design. A specific case in point is due to the construction method of the basement. In
the method of construction the basement floor slab took a great proportion of the load until the floors' support scaffold was stripped out. This had the effect of applying the building dead loads as a uniformly distributed load to the floor slabs and not as column loads transmitted directly to the piles as the design assumed.

Problems met as a result of design/construction incompatibility

As a result of the incompatibility of the design and the method used to construct there were further problems. Full appreciation of the requirements that the design had on the construction method appear not to have been fully realised by either the contractor's and or the consultant's site staff. The design implications on construction methods must be conveyed to all site staff by the design engineer. A needless failure of the structure could be caused by the site staff following normal construction procedure in ignorance of requirements of a new design approach.

This case record has highlighted how quickly water pressure can become a problem. It was fortunate that a good case was made to the client to allow monitoring of the structure. The main backbone of the argument was that monitoring would indicate any unforeseen problems arising and allow measures to be taken before a failure in the structure occurred. This situation did arise, and when large tension forces were indicated in the corner pile kentledge was applied and pumping was restarted. The tensile force was close to the design maximum tensile load for the pile, a long term design condition and not a predicted construction load. The reason for this can only be hypothesised but it would indicate that even though all parties designed conservatively, the unexpected occurred. Whether as a result of the construction method or the ground conditions the designer must realise that it is feasible to have maximum uplift and very little dead load to counteract this. The loads monitored for both the pile and slab cells have implications for the design. The short term pile
loads were predicted to be compressive. The heave in the London Clay should not have taken place, and as the excavation was pumped the increase in construction loads should have produced compressive loads in the piles. The construction procedure meant that, due to the scaffold system, the slab took more load than was designed for but this should still have resulted in compressive pile loads. This was not the case and the pile load cells showed tensile loads. During construction all pile histories showed tensile loads and at the end of construction two piles were still in tension.

With the scaffold system taking the load away from the piles and onto the slabs, the raft cells should be showing higher than predicted effective stresses. Again this is not the case, and the situation is worse because the raft cells indicate "negative" effective stress. With no compressive load indicated on either the piles or the slabs the inference is that water pressure is lifting the basement. Negative effective stress cannot be maintained over the time spans indicated by the raft cells. Thus, either the cells have malfunctioned or the readings are a function of the design of the cell and the ground conditions under the slab. The gauges have a proven pedigree and have not been reported to be subject to zero drift. Thus, the suggestion is that the readings were a function of the design and the situation. It has been suggested in the analysis section 3.7 that the gauges are weighing themselves and that for this to occur the slabs must be floating. This could not be explained by ground heave as the cells are designed such that it is immaterial whether the force is applied from above or below. The pile cells measure a total stress in the piles and thus the tensile force measured could be as a result of either heave or water pressure or indeed a combination of the two. It would appear that the levelling only commenced after this upward pressure was already effective and thus only small settlements have been recorded as the construction loads increased. This would seem to lend weight to this hypothesis,
as the heave pressures should still be increasing so soon after the excavation.

A full series of level readings is as yet unpublished by the BRE and it might well be that an initial heave was detected. The piles acting in their capacity as temporary works might keep this heave displacement to a minimum at the column locations. The slab levelling would then show if there were significant doming of the slab between the piles to enable the raft load cells to be away from the ground. If the uplift had already taken place when the initial set of level readings were taken then any increase in slab load would have no effect on the slab cell until such time as contact were made with the ground but the slab levelling would indicate a settlement. The combination of the slab level readings, the pile load cell readings, the Bourdon gauge readings and in the majority of cases the lack of slab reading would all indicate a severe water problem far earlier than was designed for and thus the full dead load of the building was not present to counteract the effect.

Instrumentation

The pile load cells have a proven record and behaved well in both tension and compression. The design to such rigorous tolerances was a necessary requirement for the corner pile. The prudence shown in not making the instrumentation the weak link of the tensile reinforcement has been justified.

There is a problem with the slab cell readings. If, as has been suggested, the gauges are weighing themselves suspended from the underside of the slab then the unforeseen has indeed occurred. In this instance an effective stress gauge is not indicating the stress situation under the slab. The combination of a total stress gauge, effective stress gauge and Bourdon gauge might have confirmed the situation if all three gauges could be believed together. The gauges
have a good record of trouble free operation working in London Clay and it is unlikely that each individual gauge would develop the same fault resulting in negative stress readings. It is not thought that the water problem is as a result of the inter-connectivity of the gauges to the bourdon gauge. However, if the water problem were local to one part of the building this inter-connectivity would certainly have accelerated its effect. Water was always a problem and the site diary shows it to have been causing trouble in the region of the instrumentation even before backfilling of the basement walls took place.

Surveying

BRE settlement sockets set into a vertical surface are well proven. The use of them set into the floor has not been reported previously. Provided that the faces of the sockets can be protected from the construction activity, the method removes doubt that the socket has been cleaned out thoroughly and that the bolt can be screwed all the way home. The same ball bearing diameter must then be used on all occasions. Equal care must be taken with the faces of the ball bearing so that they seat consistently into the BRE socket mounted in the floor. The use of studs in the walls and columns from which to hang a rule is sound provided the stud does not rust or in any other way oxidise. A permanent facility was provided to transfer the level down from ground level to the basement and a good closure between the two deep datums was maintained. Closure was said to be better than 1mm on all occasions and that levelling between the two datums suggested less than 0.2mm differential settlement had occurred between them. Little data has been published by the Building Research Establishment to confirm these as yet verbal statements about the accuracy of the levelling. All the indications from the levelling data published to date are that there is a small amount of settlement of the slab when construction was in an advanced stage. This does not agree fully with the indications the pile cells are giving or indeed
fully agree with the scenario for the slab cells. However this all
presumes the base readings for the levelling were taken prior to the
presumed heave taking place. If the base readings were taken after
the heave then the wall and column level histories are compatible in
that they measure a settlement expected with increasing building
loads. A full set of level data might help with the situation, but
equally it might cloud the situation further. Some time is required
to set up the level stations and, in the normal situation in London
Clay, it is more important to provide a permanent means of level
transfer from ground to basement to give good base readings than to
obtain the first readings, as heave is normally minimal at this stage.

The levelling data released by the Building Research Station to date
are limited to that reported by Rickard et al (1985). The data,
released at the end of construction but prior to fitting out, show
only very small settlements at the wall and column locations. This is
in agreement with the overall pile cell readings which show tension in
piles A, C and D. The pile cells would indicate that small heaves or
settlements should be recorded by the levelling depending if the
initial level readings were taken before or during the proposed
conditions of water gaining access under the slabs.

Water

It is well known that water equilibrates very slowly in the London
Clay. It would appear that monitoring of piezometers during the site
investigation did not show otherwise for the Basildon site. Hence the
presence of water was designed for as a long term condition. The lack
of specific information in the basement area was not considered
detrimental providing the design was undertaken with the water table
1m below ground level. Sandy lenses can produce local water problems,
but these can normally be dealt with by pumping as they are generally
of a finite size. The rest of the site did not indicate the presence
of such lenses so if they did exist they are unlikely to have produced
the proportions of water encountered. Unless this stratum were at basement level its presence was not suggested in any site records, indeed the excavation was described by site staff to be dry and clean.

During construction the water problem was evident. The extracts from the site diary would indicate water seepage through cracks in the basement to be a serious long term, difficult to cure, problem. Water seepage through cracks in the concrete was reported weekly on the snag list for over a year with little success for the contractor repairing them. Pumping took place throughout the construction, apart from one instance early in the construction of the basement, and thus no detrimental long term effects to the building should be expected. The one instance occurred during a holiday period when the corner pile cell indicated a very high tensile force, pumping was started, kentledge applied and the tensile load on the pile was reduced. Once the full construction load was in place and the working shop loads applied the loading conditions were as per the long term design condition.

The British Geological Survey had just undertaken a survey of the area covered by the memoir. The authors (Lake and Sheppard-Thorn 1985) could not find a precedent for the conditions encountered at Basildon. Indeed their borehole data for the whole area and the borehole data held by Basildon Council revealed no such conditions.
3.9. Conclusions

1) The piezometers monitored during the site investigation showed a water level within 2m of the ground level in four of the six boreholes. A high water table, as reported in the site investigation, is evident during the winter months from these boreholes. In London Clay water in the quantities encountered during construction is not expected and as a result was probably not looked for in the site investigation.

2) The site investigation must be planned thoroughly at the desk study phase by the consultant. In times where money appears to be reluctantly spent on the site investigation the consultant must ensure that he is giving the client full value for money. Consideration of the likely foundation designs for the proposed structures will lead to the consultant giving specific direction to the site investigation company to produce parameters necessary to complete a specific project. It is not the desired situation for the site investigation report to recommend further investigation that would be desirable to undertake the likely design approach as for the Alders Basement. The consultant should have positioned his boreholes so that one fell within the bounds of each basement on the site. In the case of the Alders basement no provision was made to monitor the ground water conditions in the site investigation. This information has implications for both the long and short design considerations.

3) The site investigation did not highlight the water problem. It is not a feature of the London Clay to have a water problem in the timescale encountered and was thus not looked for in the site investigation. It is debatable that if a water problem, such as that encountered, were looked for in the London Clay at the site investigation stage that it would be found. As the
excavation was to be open until the basement walls were complete, it was fortunate that provision had been made to keep the excavation clear of water. The site staff reported that the excavation sides were dry and yet pumping was virtually continuous indicating a water source below basement foundation level.

4) The designer must check that normal construction practice does not have implications for his design. If the design requires a specific form of construction then the tender and contract drawings must include a specific item for this. The Alders building was clearly designed so that the loads came down the columns onto the piles. The method for construction brought all the load down through the scaffold system as a uniformly distributed load onto the basement floor slab. The slab was designed to take the short term condition, 

maximum building loads, allowing for load sharing between the piles and the raft, but ignoring uplift due to water pressure and heave effects.

The method of construction put virtually the maximum building loads plus the formwork and falsework loads onto the slab. It is perhaps fortunate that the water pressure was present or the slab may have suffered deflections greater that those designed for.

5) As a result of the above effects piles B, C and D have all operated well within their design limits (Figs. 3.5/6, 3.7/19, 3.7/20 and 3.7/21).

6) The design method has not predicted the loading extremes on the corner pile PCA (Fig.3.5/6). The pile was designed for 500kN tension (Fig.3.5/6) and yet it can be seen that the measured load was in excess of this on a number of occasions before substantial building loads were present, (Figs.3.7/18 and
The design failure of a concrete pile in tension is unacceptable cracking of the concrete and thus it is unlikely that yielding of the steel would have occurred on the occasions the load has exceeded 500kN. Although the design load in tension is 500kN "failure" may not have occurred due to factors of safety applied to the dead and live design loads. The design calculations were not available to the author and so no definite conclusion can be drawn about pile failure due to excessive crack widths. It can be concluded, however, that the design condition of maximum water pressure and minimum building loads has resulted in a design load requirement for the corner pile of a greater tension load that was predicted by the load conditions assumed by the consultant.

7) The raft cells have not functioned as they were designed to. It is believed that they are stuck to the underside of the slab and have broken contact with the clay due to the slab doming between the column locations. To prove this a full analysis of all the levelling, raft cell, pile cell and Bourdon gauge data would need to be undertaken. To be conclusive the first set of floor levels should have been taken before the postulated doming took place. The column locations should then show very little movement whilst the floor slab positions should show heave. If this it true it would explain the very small settlements reported in Fig.3.7/22 (Rickard et al 1985).
4.0. General Conclusions

1) Site investigations must be planned carefully before site work commences. Having been commissioned to design and or construct a structure, the size of the foundations and the ground loadings will be known. With some knowledge of the founding stratum, from desk studies, the consultant must have an idea of the likely foundation design method. With all of the above known the field work must be specified to provide those parameters necessary to complete the design. In both of the case records presented some parameters necessary to the design had not been investigated or monitored. The site investigations were completed to specification yet the foundation designs could not be completed without assumptions being made or without further investigation being undertaken. In the case of the Flour Mill at Corby a second investigation was undertaken to provide the strength and thickness of the limestone and to provide a value of stiffness for the underlying Lias Clay. In the case of the Alders building at Basildon assumptions were made about the ground water in the basement area where no monitoring had taken place. A strength-stiffness relationship was assumed for the London Clay to provide a modulus value for the settlement analysis. It was common practice at that time for the designer to obtain a value of modulus in this way. Commercial testing houses now undertake the new laboratory testing techniques with local strain measurement. As a result a direct measurement of stiffness is possible and preferable. In the case of Basildon the use of a directly measured value for stiffness would have avoided the misinterpretation of Butler(1974) where the relationship $E_u=400C_u$ was used in the design when the relationship $E_u=220C_u$ was the more common value used at the time.

2) Good quality undisturbed core is required from boreholes. In the case of Corby good quality core provided by rotary drilling
with bentonite mud flush under close site supervision was necessary to sample the limestone and the underlying Lias Clay. New laboratory testing techniques involving local strain measurement and mid plane pore water measurement are now available in the Commercial laboratory. It is important that samples tested by these techniques are undisturbed. Any loosening of fissures would result in low values of stiffness from the small strains the samples are subjected to initially.

3) The new laboratory techniques used on good quality core are necessary to produce values of modulus for the soil at small strains. Representative values of modulus are essential in the settlement analysis if good predictions of settlement are to be achieved. The value of stiffness attributed to the founding stratum will determine the magnitude of the settlement predicted. This is the case with all methods of settlement analysis from the simple to the complicated although some methods are more sensitive than others to changes in the stiffness of the founding stratum.

4) Analysis predicting the differential settlement across a foundation slab is sensitive to the value of stiffness of the founding stratum and to the value of stiffness of the structure. It is usual for the designer to use the stiffness of the slab only, but, potentially most completed structures are very stiff bodies. This could increase the overall value of stiffness for the structure by several orders of magnitude. A method of predicting a representative value of stiffness needs to be found. The alternatives at present are,

a) to use the value of stiffness for the slab only, knowing that this will over predict the amount of bending of the slab, and to make an engineering judgement as to whether the true
bending settlement will fall within the bounds of the specified differential settlement limits.

b) to increase the thickness of the foundation slab in the design so that the slab stiffness alone will restrict the differential settlement to lie within the specified bounds.

Clearly some form of judgement is required in a) which could lead to errors. In b) the method could lead to an uneconomic solution with a very thick slab unmanageable quantities of reinforcement steel and additional, unnecessary, load on the founding stratum.

A safe method of prediction for overall stiffness is necessary to avoid failure of the kind described by Burland and Davidson (1976).

5) The method of construction must be considered when assessing the short term loading conditions for a design. The method of construction at Basildon, for the Alders building, involved the use of falsework to deck out the ground floor slab and subsequent floors above ground level. The use of falsework directed the building loads away from the columns and transmitted the load as a uniformly distributed load on the basement floor slab. The short term design considered the structure as complete, but without fixtures and fittings. The short term design loads are transmitted through the columns to the pile heads, an equally valid but different load condition for both the slab and the piles, which in this case did not produce the worst short term load conditions.
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Appendix A

In the general case $E_u$ does not equal $1/m_u$.

Hooke's Law states that

$$
\epsilon_y = \frac{1}{E_y} (\sigma'_y - v'\sigma'_x - v'\sigma'_z) \quad \text{-------1}
$$

$$
\epsilon_x = \frac{1}{E_x} (\sigma'_x - v'\sigma'_y - v'\sigma'_z) \quad \text{-------2}
$$

$$
\epsilon_z = \frac{1}{E_z} (\sigma'_z - v'\sigma'_x - v'\sigma'_y) \quad \text{-------3}
$$

Under $K_0$ conditions

$$
\epsilon_x = \epsilon_z = 0 \quad \text{and} \quad \sigma'_x = \sigma'_z
$$

Equation 1 can now be written as

$$
\epsilon_y = \frac{1}{E_y} (\sigma'_y - 2v'\sigma'_x) \quad \text{-------4}
$$

Since

$$
\epsilon_x = \epsilon_z
$$

equation 2 reduces to

$$
\sigma'_x = v'(\sigma'_y + \sigma'_z)
$$

but $\sigma'_x = \sigma'_z$

$\therefore \sigma'_x = v'(\sigma'_y + \sigma'_x)$
Substituting equation 5 into equation 4

\[
\varepsilon_y = \frac{1}{E'} \left( \sigma_y - 2\nu \left( \frac{1}{1-\nu} \right) \sigma_y \right) \\
\varepsilon_y = \frac{\sigma_y}{E'} \left( \frac{1-\nu}{1-2\nu} \right) \\
\therefore \varepsilon_y = \frac{\sigma_y}{E'} \left( \frac{1+\nu(1-2\nu)}{1-\nu} \right)
\]

The coefficient of volume compressibility is defined as

\[
m_v = \frac{\Delta H}{H_0} \frac{1}{\Delta \rho'}
\]

\[
\therefore m_v = \frac{(1+\nu)(1-2\nu)}{(1-\nu)E'} \frac{1}{\Delta \rho'} \quad \text{---6}
\]

\(E', \nu', E_u \text{ and } \nu_u\)
can be expressed in terms of the Shear Modulus, \(G\)

\[
\left( \frac{1-\nu'}{E'} \right) = \left( \frac{1-\nu_u}{E_u} \right) = G \quad \text{---7}
\]
substitute equation 7 into equation 6

\[ m_v = \frac{(1-2v') (1+v_u)}{(1-v') E_u} \]

For undrained conditions

\[ \frac{\Delta V}{V} = 0, \text{ and therefore } v_u = 0.5 \]

\[ m_v = \frac{1.5(1-2v')}{(1-v')} \frac{1}{E_u} \]

Thus \[ m_v = \frac{1}{E_u} \text{ only if } v' = 0.25 \]

The variation in the relationship can be seen in the table below

<table>
<thead>
<tr>
<th>( v' )</th>
<th>( m_v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10</td>
<td>( \frac{1.333}{E_u} )</td>
</tr>
<tr>
<td>0.15</td>
<td>( \frac{1.235}{E_u} )</td>
</tr>
<tr>
<td>0.20</td>
<td>( \frac{1.125}{E_u} )</td>
</tr>
<tr>
<td>0.25</td>
<td>( \frac{1.000}{E_u} )</td>
</tr>
<tr>
<td>0.30</td>
<td>( \frac{0.857}{E_u} )</td>
</tr>
</tbody>
</table>