16. Deep basement construction at College Road, Harrow

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The Legal & General Assurance Society Limited development in Harrow was completed during 1991. A five storey deep basement was constructed beneath the structure using a top down form of construction. Building loads were carried on large diameter bored piles founded in stiff to hard sandy clays of the London Clay and Woolwich & Reading Beds present beneath the site. The perimeter retaining wall was formed using a contiguous bored pile wall construction. Substructure construction was carried out using methods designed to minimise ground movements and their effect on adjacent properties, roads and services. Retaining wall movements were predicted as part of the design process and these have been compared with measurements taken during the excavation. Measurements confirm that wall movements were within the design limits.

INTRODUCTION

The Project

1. The Legal & General development in College Road Harrow was completed during 1991. The building comprises a seven storey reinforced concrete framed structure about 35m by 40m in plan area providing approximately 6500m^2 office space. The site is in an extremely busy and congested part of Harrow and it was therefore necessary to provide on site car parking facilities. Because of the restricted site a five storey deep basement beneath the structure was proposed. To avoid loss of car parking space the 17m deep basement comprises reinforced concrete slabs on ten half levels which form a spiral down to the deepest level. Figure 1 shows a typical section through the basement.

Site Constraints

2. The particular circumstances of the site and required construction programme dictated the choice of foundation and method. The principal constraints were as follows:

- The proximity of adjacent structures (Figure 2), particularly St.Anns Shopping Centre required a construction method that would minimise ground movements and prevent undue distortion or damage to existing buildings.
- The limited site area required the maximum use of the space available and a method of construction that would minimise the size of the perimeter retaining structure.
- Final planning consent necessitated a very tight construction programme requiring the superstructure to be built at the same time as the basement.

3. These constraints led to a top down method of construction being selected. The perimeter basement retaining wall was proposed to be formed using a contiguous bored pile wall propped by the permanent basement floor slabs. Foundation loads were proposed to be carried on large diameter bored piles.

GROUND CONDITIONS

History

4. The Legal & General site comprised a vacant plot of land close to Harrow town centre. The existence of old foundations over the northern part of the site indicated previous development but no details are known.

Site Location

5. The site comprises a level area approximately 35m by 40m in plan area at about 66m OD (see Figure 2). The site is bounded by College Road to the south, by the St.Anns Shopping Centre to the east, and by the approach ramp to the St.Anns Centre car park on the west and north sides.

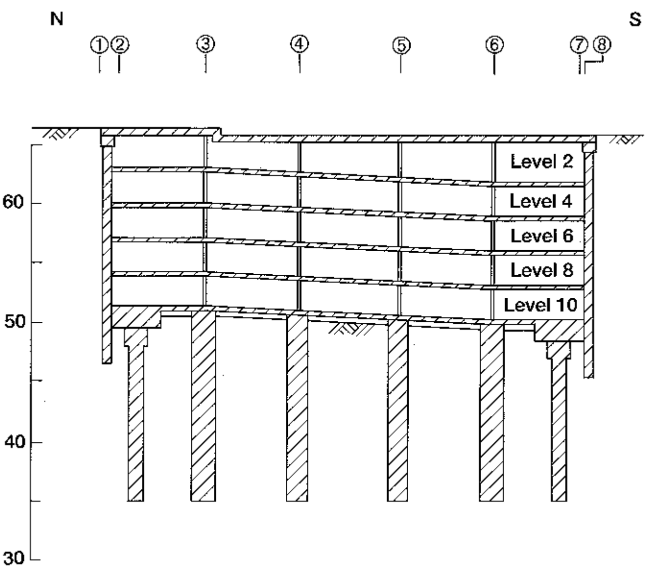

Fig. 1. Typical basement section.

Site Investigation

6. During 1989, six boreholes were sunk to depths between 19m and 43m to enable insitu tests to be carried out and to recover samples for laboratory testing. Two piezometers were installed to monitor piezometric pressures. Ground conditions encountered are typical for Harrow, comprising a thin layer of fill over London Clay over Woolwich and Reading Beds at a level of about 36m OD (see Figure 3). The underlying Thanet sand and chalk were not proved but are thought to be at levels of about 20m OD and 15m OD respectively.

Fill

7. The made ground was found to be predominantly brick and concrete hardcore with gravel and a partial clay matrix. Although only indicated to a maximum depth of 2.2m below ground level in the boreholes, disturbed ground and fill was encountered to depths up to 7m close to the existing access ramp to St.Anns Centre.

London Clay

8. The London Clay was found to comprise an upper brown weathered zone to about 8m depth over unweathered fissured clay. The weathered clay was described as firm mottled brown and grey generally fissured clay becoming stiff brown fissured clay. Scattered selenite crystals and pockets and partings of fine sand were also noted.

9. The unweathered London Clay comprised very stiff grey fissured clay with scattered thin partings of grey fine sand over hard grey silty to very silty clay with partings of light grey fine sand. The lower clay is generally only slightly fissured or unfissured. Between about 39m OD and 36m OD the London Clay was found to be hard grey very silty sandy clay interbedded with clayey sand. These beds are referred to as the Basement Beds often noted for their potential instability. However, although some minor water seepages were noted from this zone no instability was encountered.

10. In situ Standard Penetration tests were carried out in one borehole. Laboratory testing included routine quick undrained triaxial tests and soil classification tests to determine profiles of undrained shear strength, moisture contents and index tests. Summaries of these tests are given in Figure 3.

11. In addition specialised tests were carried out to determine effective stress parameters and soil Young's modulus for design of the retaining walls.

Woolwich & Reading Beds

12. The Woolwich and Reading Beds were encountered at about 36m OD and were proved to a level of 23m OD. They generally comprise hard

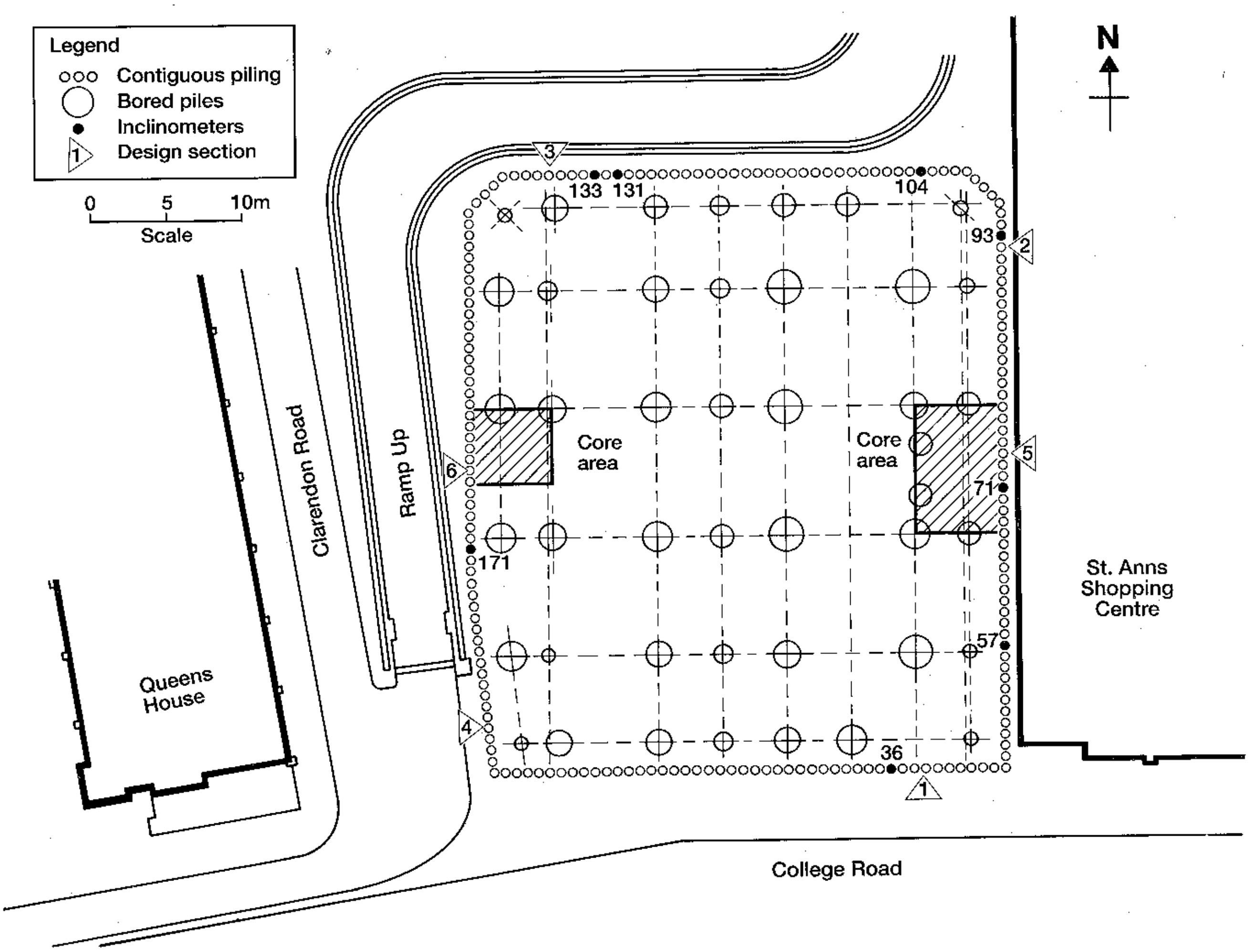

Fig. 2. Site layout plan.

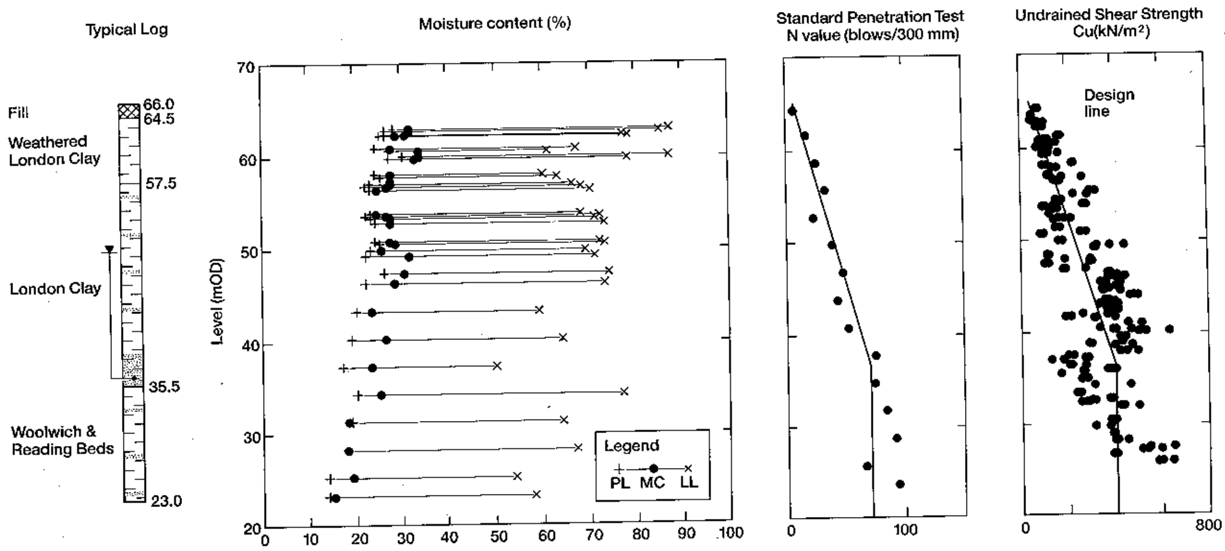

Fig. 3. Typical log and laboratory test results.

multicoloured clay with occasional partings of fine sand. Some sandy and very sandy clays were also noted.

13. Laboratory testing included quick undrained triaxial and soil classification tests.

Groundwater

14. Minor seepages of groundwater were noted close to the base of the London Clay. Two standpipe piezometers were installed to allow long term monitoring. Readings taken in September 1989 suggested a piezometric head at about 50m OD. However it is not certain that these readings reflect the true piezometric level even allowing for underdrainage caused by many years of pumping from the underlying aquifer.

FOUNDATIONS

Top Down Construction

15. As a result of the site constraints a top down form of construction was proposed allowing the perimeter wall to be propped before excavation by the permanent basement floor slabs. To provide support to the floor slabs it was necessary first to install steel column sections founded on large diameter bored piles with deep cut offs up to 18m below street level. Support to the sides of the bore through the depth of the proposed basement was provided by Armco casings brought to ground level.

16. Pile layout is shown in Figure 2.

Design

17. Proposed pile loads varied between 2300kN and 12100kN. To ensure similar pile toe levels for all piles it was decided to construct piles varying in diameter between 900mm and 2100mm as shown in Table 1.

18. Preliminary pile design was based on the traditional empirical approach described by Skempton (1959). Pile shaft friction was limited to $100kN/m^2$ with an assumed end bearing pressure of $3600kN/m^2$. A factor of safety of 2 was adopted which was to be confirmed during a preliminary pile test.

Table 1. Pile sizes

Pile dia (mm)	Maximum load (kN)	Number of piles
900	4000	7
1200	6000	5
1500	8000	15
1800	10500	12
2100	12500	5

Pile Testing

19. A preliminary trial pile was installed by Keller Foundations on the 26th April 1990 together with four anchor piles. A 1200mm diameter pile was installed to a depth of 34m below ground level. The upper 18.5m of shaft was sleeved with a permanent galvanised liner tube coated with a 6mm layer of Colas Bitumen Compound SL to act as a slip layer to simulate the deep cut off. The small annulus between clay and liner was filled with dry sand.

20. Pile testing was carried out between the 8th and 13th May 1990 to a maximum load of 1500 Tonnes generally following the ICE Specification for Piling (1988). Two load cycles were carried out as shown in Figure 4 resulting in a maximum

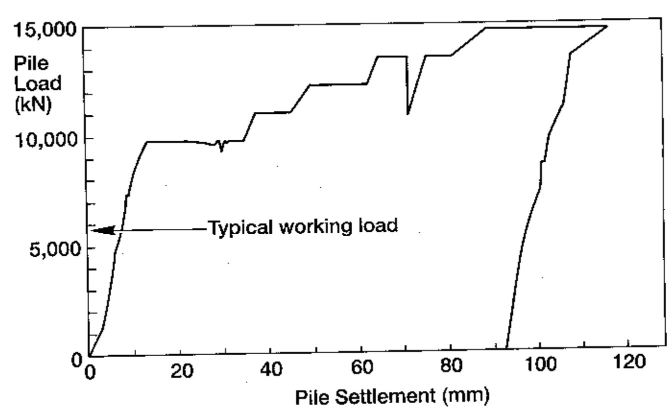

Fig. 4. Pile test results.

settlement of about 120mm at the maximum test load.

21. Back analysis of the test results indicated actual achieved shaft friction between 130kN/m^2 and 170kN/m^2 depending on the assumed reduction in shaft friction obtained over the upper 18.5m. Using the undrained shear strength design line shown in Figure 3 indicated alpha values for shaft friction between 0.37 and 0.48. Use of the lower value enabled calculated shaft capacity to be increased by about 30% when compared to the preliminary design resulting in savings to the contract in excess of the cost of the testing.

Pile Settlements

22. From the preliminary test pile results (Figure 4) settlement at working load can be estimated to be about 6mm allowing for compression in the shaft over the upper 18.5m. Back analysis suggests an operating Young's Modulus equal to about 300 Cu. On this basis individual pile settlements between 5mm and 10mm were anticipated for the range of pile diameters proposed although group effects were expected to result in settlements of about 20mm.

Basement Heave

23. Prediction of substructure movements also required the effect of the basement excavation to be taken into account. Excavations between 17m and 18m in depth were estimated to reduce the total stress at basement level by about 350kN/m^2. Computed heaves between 30mm and 60mm were anticipated at the basement formation level for the immediate and long term fully drained situations. This was expected to cause heave of the piled foundation between 10mm and 20mm resulting in almost zero net movement of the building in the long term.

24. The effect of the ground heave on the piles was checked to ensure sufficient control of cracking. Maximum pile shaft tensile strains for both the undrained and fully drained conditions for an unloaded pile were in all cases less than the maximum allowable strain of 1750 x 10^{-6} suggested by BS8007 as being sufficient to limit cracks to less than 0.2mm.

RETAINING WALLS

Sequence of Excavation & Construction

25. Top down construction was adopted as the most effective means of minimising retaining wall displacements and thus damaging movements in the soil mass outside of the site area. The choice however also allowed the thickness of the basement wall to be reduced, but most importantly enabled the superstructure to be built at the same time as the basement. This last aspect enable considerable savings to be made to the construction programme.

26. Ground conditions were ideal for using the top down method of construction which is particularly favoured to cohesive soils. Ground conditions also allowed the use of contiguous bored piles to form the perimeter wall without the need for expensive grouting or other measures to control the inflow of groundwater.

27. The principal steps adopted for the construction sequence are illustrated in Figure 5 and described below:

Carry out local enabling works including removal of old foundations and other obstructions.

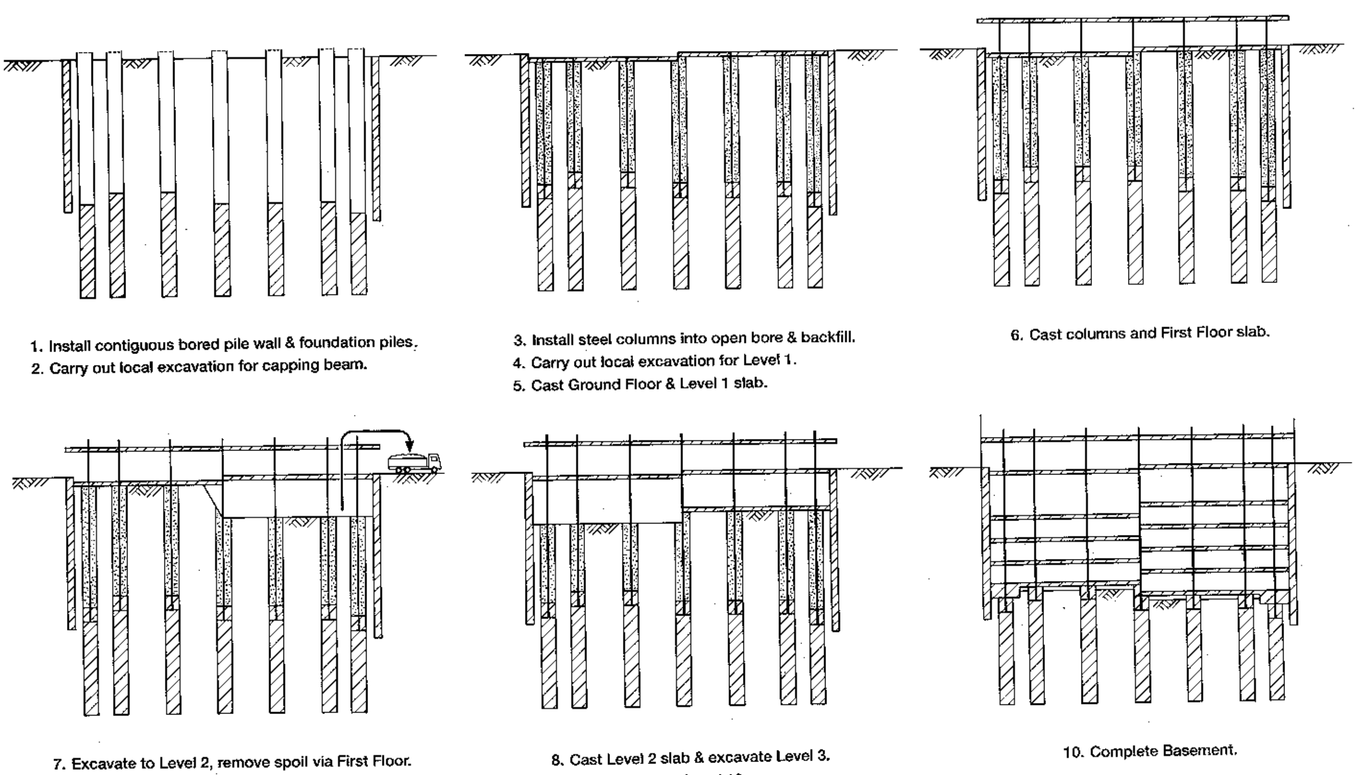

Fig. 5. Basement excavation and construction sequence.

- Install perimeter contiguous bored pile retaining wall using 600mm nominal auger bored piles constructed at 0.75m spacing.
- Construct large diameter bored foundation piles with a deep cut off with an open bore lined with Armco casing through the depth of the basement.
- Install permanent steel universal column sections into open bore after trimming and scabbling the pile head.
- Construct ground floor slab on top of the steel stanchions. Continue traditional construction upwards to complete the superstructure.
- Commence basement excavation to the first basement slab level (Level 2) over half the basement area removing spoil via core shafts.
- While carrying out steel fixing and casting of the first basement slab complete excavations to the second basement slab level (Level 3) over the remaining half of the basement.
- Following acceptable curing of the basement slab, excavations below can begin to the third half level (Level 4).
- Continue the excavation, slab construction and curing cycle level by level until completion of the basement (Level 10).
- Complete basement core shafts and internal perimeter cavity wall construction.

Design

28. Considerable difficulties exist in designing multi propped retaining walls in heavily over consolidated cohesive soils. No relevant code of practice is available but design was generally carried out in accordance with CIRIA Report 104 particularly with regard to choice of soil design parameters.

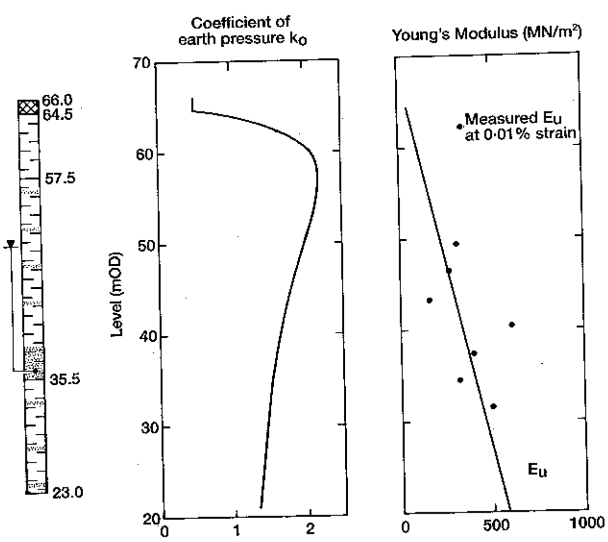

Fig. 6. Assumed soil stiffness and k_o profile.

29. The statically indeterminant nature of a multi propped wall and the high degree of soil-structure interaction expected precluded the use of traditional design approaches. Design was therefore based on the results of analyses carried out using the Oasys Limited computer program FREW. This computer program enables sophisticated analysis of the soil-structure interaction to be carried out in steps corresponding to the actual construction sequence proposed. FREW is more advanced than other available computer programs in using a soil continuum flexibility method based on true finite elements rather than the unrealistic subgrade reaction models. A full description of the program and examples of its use are given by Pappin et al (1986).

30. The soil stiffness and Ko profile adopted in the analysis together with a typical design section are given in Figures 6 and 7.

Wall Displacements

31. Retaining wall analyses were carried out for four typical sections representing the lengths of wall illustrated on the pile layout plan (Figure 2) and the two core areas. For each analysis, computer output included wall movements, shear forces, bending moments, soil pressures and prop forces at each stage of construction. Considerable quantities of data are generated and the ability to display much of this in the form of graphical output is extremely valuable. Results for the deepest length of wall (section 1) are given in detail in Figure 8. These show the development of the wall movements stage by stage until completion

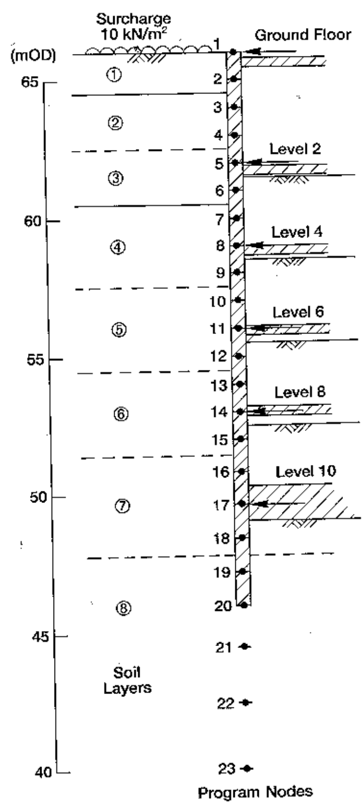

Fig. 7. Typical design section.

of the basement with maximum movements of about 30mm computed.

32. A summary of the computed wall movements is given in Table 2 for the four wall sections and two core areas considered. Although this table details computed wall displacements it was considered that these values were upperbound and were unlikely to be exceeded. Assumed soil properties were generally taken as moderately conservative resulting in lower strengths and stiffnesses and higher initial stresses than is probably the case for the soil insitu. Likewise assumed values for the wall member and slab props were also conservative and were expected to give larger computed movements. Predicted movements were thus expected to be smaller than those quoted.

CONSTRUCTION

Retaining Wall Piles

33. Installation of retaining wall piles commenced during late April 1990 using a crane mounted Soilmec auger boring rig and back up crane for handling casings and reinforcement cages. The retaining wall piles comprised 600mm nominal diameter piles installed at a spacing of 0.75m. Bored depth varied between 17.5m and 27.3m below ground level at about 65.5m OD.

34. Piles were generally installed using a short length of casing to support the bore through the upper fill deposits with the remainder of the bore being completed unlined. Casings used were generally about 3m although piles close to the access ramp to St.Anns centre required casings up to 8m in length because of the disturbed nature of the ground. No problems with groundwater were encountered.

35. Retaining wall piles were reinforced full length with an 8T20 cage increasing to 10T20 adjacent to the core areas and 8T32 for the deeper excavation. To cater for the anticipated high shear forces T16 hoops were used either at 300mm centres or at 200mm centres for the heavier cages.

36. The achieved construction programme resulted in a total of 193 piles being installed in a period of 8 weeks.

Bored Piling

37. Construction of foundation piles varying in diameter from 900mm to 2100mm was started after completion of about 75% of the perimeter retaining wall during early June 1990. Because of the very restricted nature of the site the order in which piling was carried out was carefully sequenced to avoid access problems. All piles were constructed following a sequence of construction illustrated in figure 9 and described below:

- After installing temporary casing to ensure stability of the fill a pilot hole was bored one or two metres short of the final depth.

- The pile bore down to cut off level was reamed out to either 1800mm or 2100mm using a special auger with attached lead tube section to ensure concentricity of the ream.

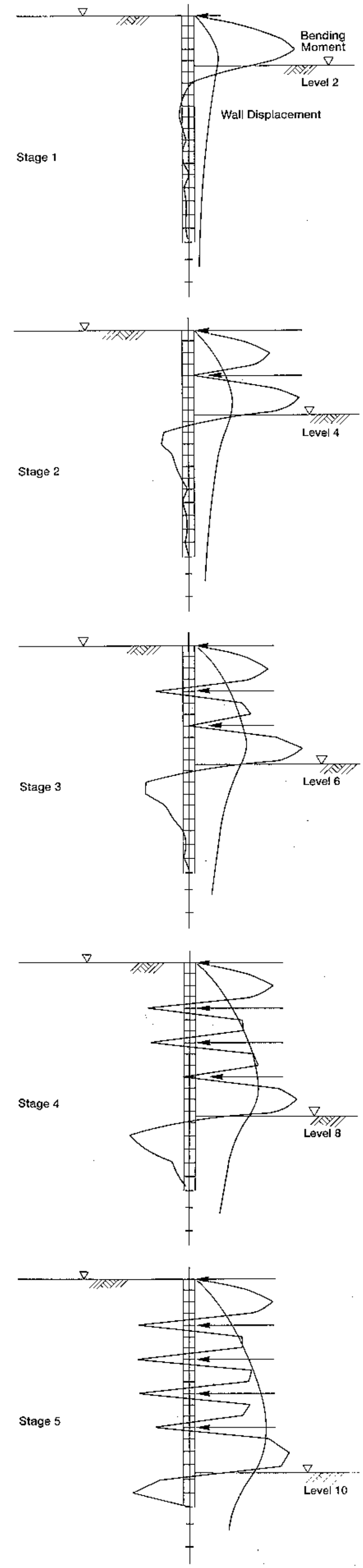

Fig. 8. Computed wall movements and bending moments.

Table 2. Computed wall displacements

Section 1		Movement of wall (mm)						
Stage	GF	2	4	6	8	10	TOE	MAX
1	2	10	9	7	6	5	4	10
2	2	13	15	12	9	8	6	15
3	2	14	19	19	14	11	9	20
4	2	14	20	23	22	15	12	24
5	2	15	21	25	27	24	16	27
6	2	15	21	26	28	28	31	31

Section 2		Movement of wall (mm)						
Stage	GF	2	4	6	8	10	TOE	MAX
1	5	5	4	3	3	3	2	5
2	7	10	8	7	7	6	5	10
3	7	13	14	11	11	9	7	14
4	7	13	18	17	17	12	10	18
5	7	14	19	22	22	20	14	22
6	8	14	19	23	23	23	24	24

Section 3		Movement of wall (mm)						
Stage	GF	2	4	6	8	10	TOE	MAX
1		2	6	6	5	4	3	6
2		2	9	11	9	7	6	11
3		2	9	14	15	10	8	15
4		2	10	15	19	18	13	20
5		2	10	16	20	21	21	22

Section 4		Movement of wall (mm)						
Stage	GF	2	4	6	8	10	TOE	MAX
1	1	10	8	7	5		5	10
2	1	13	15	12	9		7	15
3	1	14	19	19	13		10	20
4	1	14	20	24	23		15	25
5	1	15	21	25	26		27	27

Section 5		Movement of wall (mm)						
Stage	GF	2	4	6	8	10	TOE	MAX
1	1	5	5	4	4	3	2	6
2	1	9	11	9	7	6	5	11
3	1	11	16	16	12	9	7	17
4	1	11	18	22	20	13	10	22
5	1	12	20	24	26	23	14	26
6	1	12	20	25	28	26	15	28
7	1	18	27	32	34	30	30	34

Section 6		Movement of wall (mm)						
Stage	GF	2	4	6	8	10	TOE	MAX
1	3		13	10	8	6	5	13
2	3		19	18	13	10	7	20
3	4		22	25	22	15	10	25
4	4		24	28	30	26	15	30
5	4		24	28	31	29	15	31
6	10		31	36	38	33	33	38

- Pile bore was then cleaned out and sunk to the required depth and the spiral liner tube installed to cut off level.

- A full length reinforcement cage was placed in the bore which was then concreted to a level within +200mm or -500mm of the required cut off. Reinforcement was left about 3.5m above cut off level.

- The pile construction was completed by backfilling the annulus around the permanent liner with pea gravel and welding a safety grid over the top of the liner.

38. Bored piles were installed with the same crane mounted Soilmec pile boring machine as used for the retaining wall piles. A back up crane was used to handle the reinforcement cages and permanent liner tubes.

39. Foundation piles were installed to depths between 31m and 35m below ground level end bearing in the Woolwich and Reading clays.

40. Piles were constructed with a minimum of 0.5% reinforcement over the upper section of shaft reducing to about 0.25% for the remainder of the bore. Typical cages used are summarised in Table 3.

41. Higher percentages of reinforcement were required for the smaller diameter piles to cater for induced bending moments caused by allowable positional and verticality tolerances. For the deep pile cut off levels a potential load eccentricity of over 300mm was possible. In

Table 3. Pile reinforcement

Pile dia (mm)	Reinforcement cage
900	18 T 32
1200	14 T 32
1500	12 T 32
1800	16 T 32
2100	22 T 32

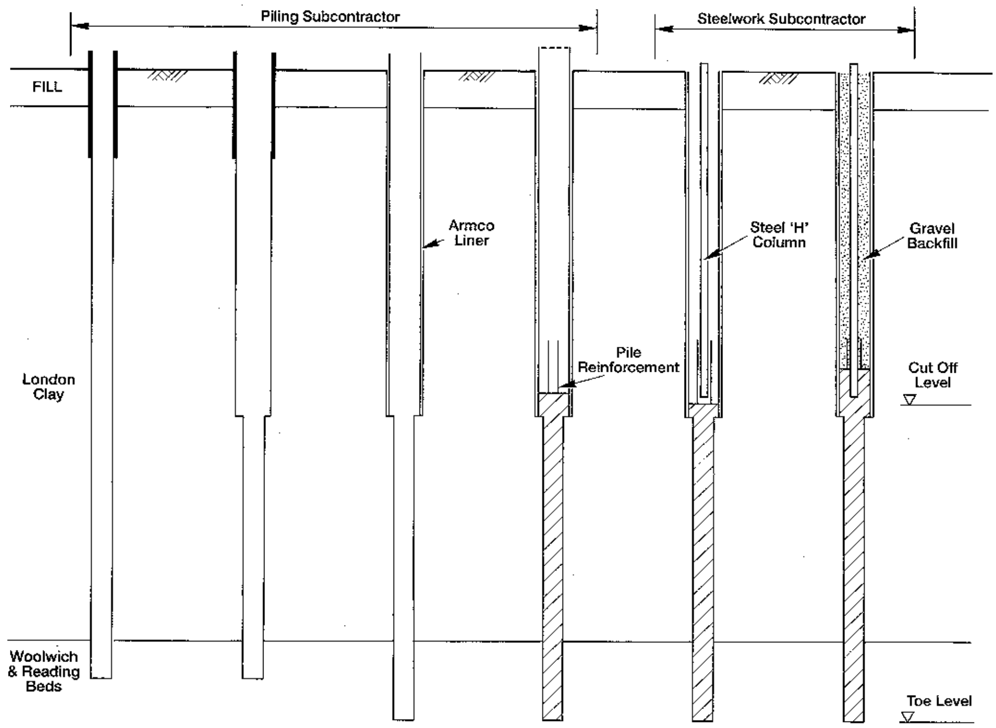

Fig. 9. Pile construction sequence.

addition to this check, pile reinforcement was also checked to ensure sufficient steel was available to cater for heave induced tensions.

42. A total of 42 permanent and 2 temporary foundation piles were constructed during a 10 week period (see Figure 2). To ensure that work was completed to programme a second piling rig was used for about two weeks. All piling works were completed on schedule enabling the follow up subcontractors to commence without delay.

Basement Construction

43. Basement construction started in August 1990 with the placing of permanent steel columns within the lined pile access shafts. Figure 9 shows details of this operation including the backfilling of the shafts with pea gravel and concrete plugs to provide the necessary stability to the columns.

44. The generalised sequence of excavation and construction was described above.

45. The first requirements were to construct the perimeter wall capping beam and ground floor slab which was to act as a strut to support the wall. Conventional top down construction could have started once this slab was complete. However, because of the restricted site the Main Contractor opted to begin construction of the building superstructure. Only after completion of the first floor slab was equipment required for basement excavation put in place. This enabled spoil to be lifted from the basement using air driven winches to first floor level, then via dumpers to hoppers feeding directly into waiting trucks at street level. This innovation by the Main Contractor removed muck shift operations away from the basement access point greatly assisting the logistics side of the construction.

46. As a result of waiting for completion of the second floor level, bulk excavations did not get underway until January 1991. However good progress was made with the cycle of excavation, steel fixing, concrete pour and curing taking about four weeks for each complete level. Completion of excavation was reached by June 1991 with final basement structural works finished by July 1991.

INCLINOMETER MONITORING

Description

47. During the preliminary design works it was recognised that control of retaining wall movements was essential to ensure minimal effect on adjacent buildings and services. As part of this control it was proposed to monitor retaining wall displacements using inclinometers. Measurements were required to provide reassurance to the owners and occupiers of adjacent structures and to protect the client against spurious claims for damages alleged to have resulted from the excavation works. In addition, Keller Foundations recognised the benefits in obtaining data for back analysis and research into actual field performance of retaining walls.

48. The inclinometer system used at the site comprised a Geotechnical Instruments Mk IV biaxial inclinometer torpedo which was used to log grooved access tube cast into retaining wall piles. The device is designed to measure inclination or slope and by taking consecutive readings in a tube it is possible to build up an accurate profile of the tube. By comparing successive profiles taken at different times it is possible to determine movement.

49. Inclinometer access tubing was grouted into steel ducts cast into piles during the piling works. A total of eight piles were

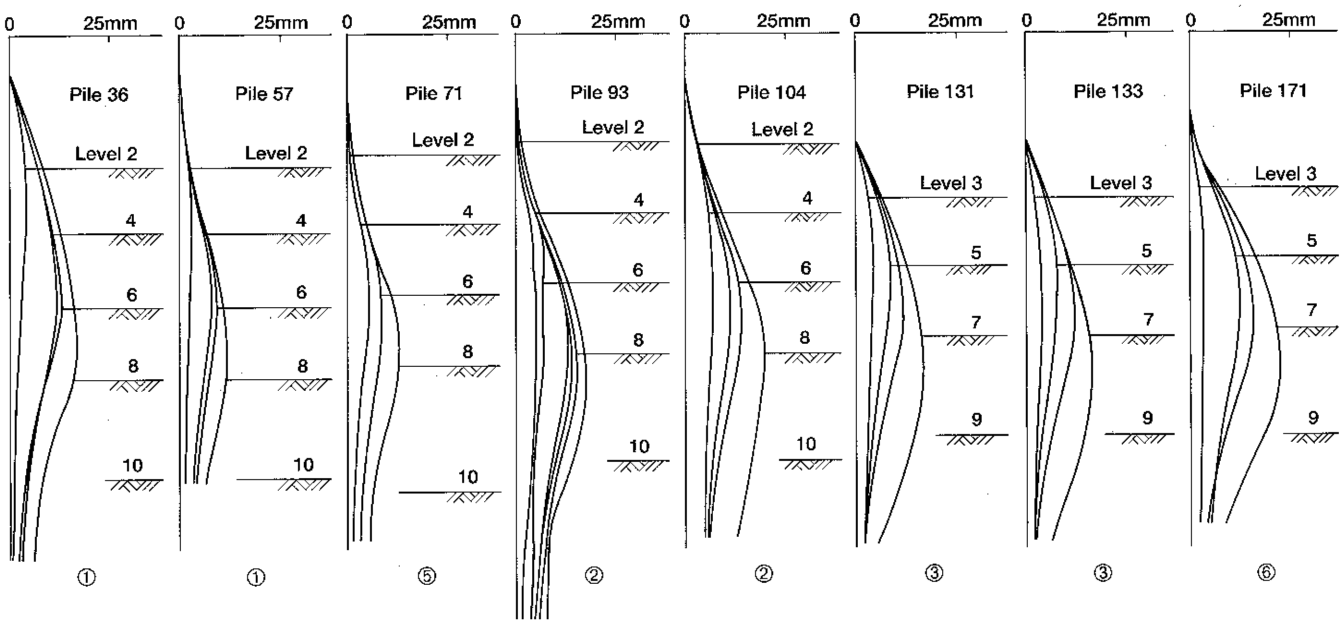

Fig. 10. Measured wall movements.

instrumented as shown on Figure 2 which included some redundancy in case of damage or loss to access tubes during construction. In the event all eight piles have been logged throughout the excavation period without loss.

Results of the Monitoring

50. Inclinometer profiles were taken at approximately one month intervals and were generally timed to coincide with the casting or curing of basement slabs. Because of logistics problems this was not always possible. Results are given for all eight installations in the form of displacements plotted against level and time and are presented in Figure 10. These figures also show the excavation level together with the location of the basement floor slabs.

51. Results have been processed assuming the upper level is fixed by capping beam and slab. This behaviour was expected based on the results of the analyses. Actual movements at beam level have been measured independently and it is understood that these measurements confirm almost no horizontal movement.

Comparison with Predictions

52. Results of the monitoring show very similar behaviour to that shown in Figure 8 obtained from the design analyses. However, the magnitude of movement is smaller than the computed values by about 30% to 40% which reflects the moderately conservative nature of the analyses. The results though are extremely encouraging and confirm that although simplified, the design method can be relied upon and realistic wall displacement predictions can be made.

ACKNOWLEDGEMENTS

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