GDP

Appendix 4

IStructE Papers

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ORDINARY MEETING

A paper to be presented and discussed at a meeting of the Institution of Structural Engineers, 11 Upper Belgrave Street, London SW1X 8BH, on Thursday 18 April 1985, at 6.00 pm.

The Granary site—

design and construction of a mechanised letter-sorting office

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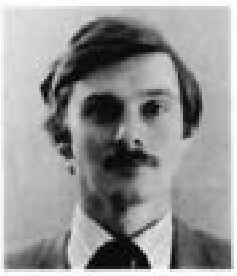
Keith White graduated through part-time study at Northampton Polytechnic, London. After National Service in the Royal Engineers, he spent 2 years as an Assistant Resident Engineer on a roads and bridgeworks contract. He then returned to the design office of Travers Morgan where he was continuously engaged as a Project Engineer for structural works until being taken into Partnership in 1973. As a Partner he has had particular responsibility for the firm's structural engineering work both in the UK and overseas.

He was first elected to the Institution's Council in 1973 and has been Chairman of both the Education & Examinations and Associate-Membership Committees. He has been a Vice-President since 1983.

Alan Myers joined the structural group of Travers Morgan & Partners on graduation from Imperial College. He has had extensive responsibility for the design and supervision of construction of many types of new building structure, upgrading and alteration schemes, and the rehabilitation of old buildings. He also has considerable experience in structural investigation work, appraisals, and certification. Other interests include the fire resistance of concrete structures, with membership of the appropriate Concrete Society committee, and computer aided drafting.

He was Project Engineer for the Granary site development from its inception early in 1980. He became an Associate of Travers Morgan in that year and a salaried Partner of the practice in 1984.



Andrew Dutton graduated from Trinity College, Cambridge, in 1973 and spent the next 6 years with Sir Bruce White, Wolfe Barry & Partners, initially involved with the design of ports for the Far East and later as principal designer for the Dartford Creek Barrier. From 1979 to 1981 he worked overseas in Mauritius as Deputy Engineer for a major water supply project. On his return to the UK he joined the structural group of Travers Morgan & Partners and has been concerned with the design of a wide range of projects in the UK and abroad. He has particular experience of buildings with difficult foundations and undertook much of the substructure design for the Granary site development.

Synopsis

This paper describes the design and construction of a district postal sorting office in North West London. The earlier use of the site and its location in relation to the Regent's Canal gave rise to the need for special consideration of the substructure. Particular reference is made to the problems arising from the removal of a former railway embankment and the soil movements likely to result. The way in which the design was developed in stages from feasibility to construction is described.

The development

The Post Office's new building complex in St. Pancras Way, London NW1, will form one of the last links in the programme for the mechanisation of letter sorting in the London postal region. Currently, the NW districts are run from three buildings, only one of which is in Post Office ownership and that is on a restricted site with little opportunity for expansion either within the site or on adjacent properties. The introduction of letter-sorting machinery would have required significant structural strengthening of that building, while the site itself would have been unable to cope with the associated increase in vehicle traffic.

In recognition of these facts the Post Office adopted a 'new build' strategy, seeking to create on one site mechanised letter-sorting facilities for the whole NW postal district. With their appointed architects they considered several alternative locations in the Camden area, together with various schemes for each, and eventually decided to proceed with the

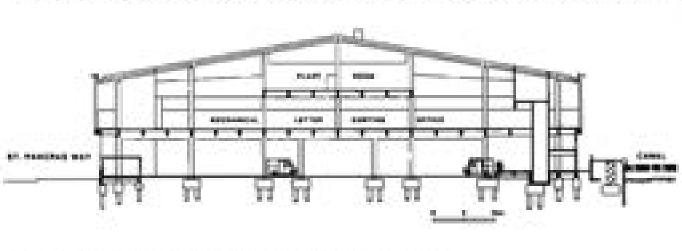


Fig 1. Cross-section through sorting office block

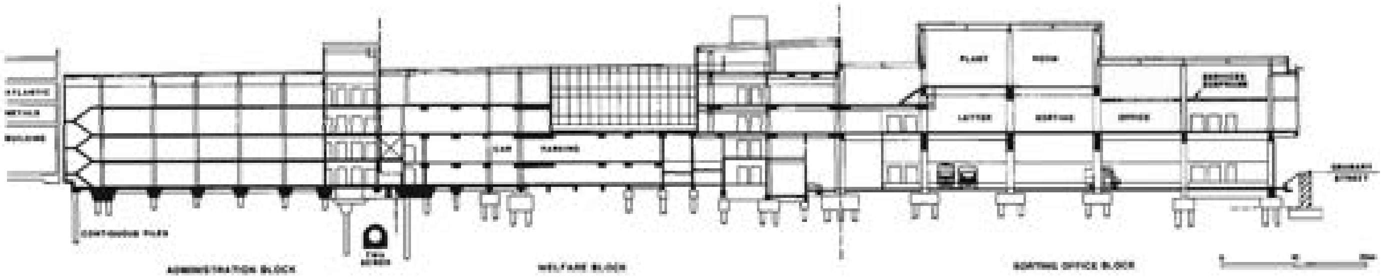


Fig 2. Longitudinal section through development

Granary site. Even that choice was less than ideal, since its limited area meant that the brief, which required all the components of the development to be located at ground level, could not be met. The necessary planning compromise was to stack some of the components so that, in the sorting office block, the mechanised letter-sorting office is at first floor above the yard for postal vehicle loading and unloading and the motor transport workshop (see Fig 1), while in the adjoining welfare block the facilities are located above a twin-level staff carpark. The development is completed by the administration block, which stands at the north end of the site (see Fig 2).

The site lies within a conservation area alongside the Regent's Canal and planning restrictions affected the height of the development.

History of the site

The Granary site is a 1·1 ha wedge-shaped piece of land between St. Pancras Way and the Regent's Canal (see Fig 3) and lies just to the north of St. Pancras Hospital. Historically, the ground sloped from east to west with a fall of about 3 m towards the River Fleet which is now culverted and lies to the west of St. Pancras Way.

In about 1817 part of the Regent's Canal construction passed along the eastern boundary where it was cut into the locally outcropping London clay. Two access bridges crossed the canal with roads running across the site. Following the advent of the railways, a brick barrel sewer was constructed beneath the canal, on the line of one of the access roads, to provide a foul drain to a goods yard on the east side of the canal.

Then, in 1864, the London and Midland Railway Company proposed the construction of the St. Pancras Ale & Corn Store, to be serviced by a branch line from the main St. Pancras railway lying to the east of the canal. The Ale & Corn Store, covering some 0·8 ha, later became known as the Granary, and was a structure with massive masonry walls and five floors supported by cast-iron columns. The railway access to the site had three tracks and crossed the canal on a two-span plate girder bridge. As the railway was elevated about 4 m above the canal, the whole of the north end of the site was raised to this level by backfilling between massive brick retaining walls up to 8 m high. Along the canal frontage a covered loading wharf was constructed by widening the canal up to the main external wall of the Granary; the wharf and roof were supported on cast-iron columns founded in the canal bed.

In 1907, as part of the trunk sewer scheme for London originally conceived by Bazalgette, the middle level sewer no. 2, now part of the Thames Water Authority's system, was constructed by tunnelling under the northern part of the site. The sewer is approximately 2 m in diameter and is brick lined throughout, with its crown about 4·5 m below the bed of the canal.

No further construction took place on the site, but in the mid 1960s a five-storey office building with a two-storey warehouse was constructed on adjoining land against the northern boundary. Known as the Atlantic Metals building, it is of reinforced concrete framed construction, with concrete block infill panels, and is founded at a relatively high level in the brown London clay.

In 1978, after several years of disuse and much debate about its future, the Granary burnt down in dramatic fashion. The ruins were considered dangerous and were demolished, measures being taken to stabilise the now free-standing canal wall and other perimeter and internal retaining walls. Some while later, the railway bridge across the canal was also demolished, thus leaving the site in the condition faced by the design team for the Post Office development.

Structural feasibility study

A feasibility study was among the first tasks undertaken, its purpose being to determine the most appropriate structural response to the proposed development. While prime cost was a significant criterion in choosing the form of the structure, considerable attention was also paid to the need to provide an expeditious resolution, in construction terms, to the problems arising from the nature of the site. The principal areas of the study are now discussed.

Superstructure options

As has been described earlier, the development is formed in three sections, the sorting office block, the welfare block, and the administration block (see Fig 4). Alternative structural solutions were considered for each section, both in steelwork and in reinforced concrete. The schemes for steelwork were discarded when they failed to show economy compared with reinforced concrete—that in part being a reflection of the complexity of some sections of the development, e.g. in the shape of the welfare block and in the shallow depth of the structural floor zone available in the administration block. The final selection was to use reinforced concrete throughout, except for the roofs which were in structural steelwork.

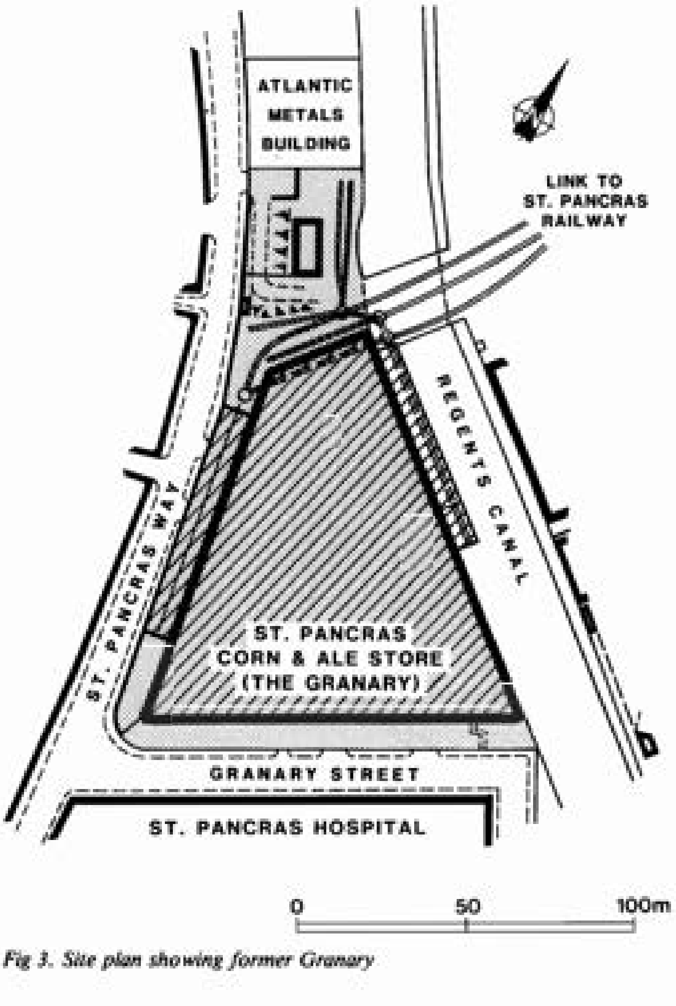


Fig 3. Site plan showing former Granary

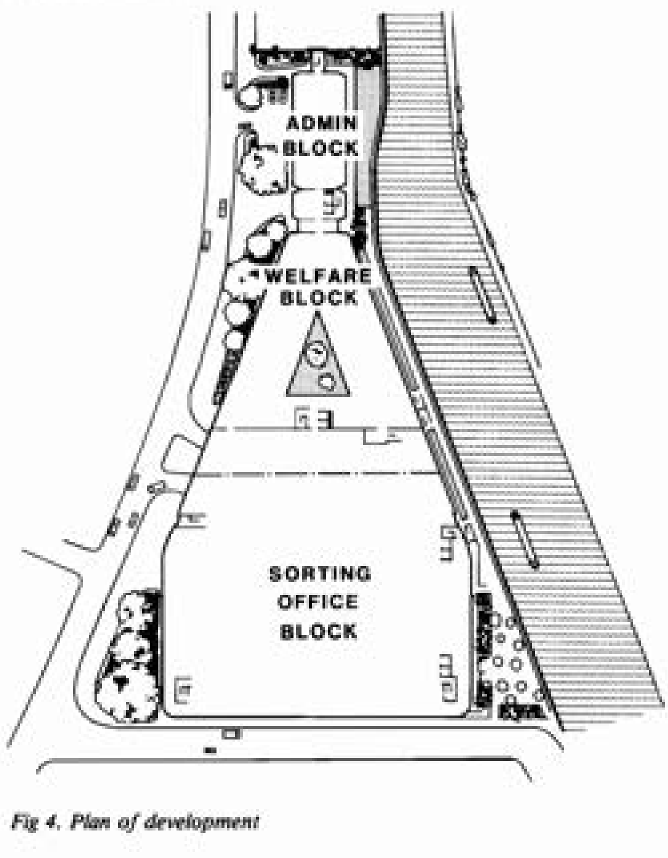


Fig 4. Plan of development

Sorting office block. In the sorting office block a column grid of 12 m × 12 m was adopted above first floor to accommodate the positioning of the mechanised letter-sorting equipment. Generally, that grid was carried down to ground floor but, to allow for vehicle movements, one row of columns was displaced. The resulting arrangement was less than ideal but to permit the optimum arrangement for vehicle movement would have made the first-floor construction excessively heavy.

The ground-floor slab in the vehicle movement areas was regarded as an external ground bearing pavement, and this resulted in the need to place 1·75 m of fill material to achieve the correct levels. Other areas at ground level, e.g. the receipt and dispatch platform, were all to be designed as fully suspended construction.

Welfare block. The welfare block, filling the triangular space between the administration block and the sorting office, was to be mainly four storeys high. The lower two storeys were to form a carpark, while the upper two were to provide welfare facilities and some offices. The brief required clear spans of 12 m in the carpark, but the height restrictions on the building meant that a structural zone for the floor construction of only 750 mm was available, resulting in a grillage of relatively shallow beams being provided.

Administration block. The administration block was also to be of four storeys but again, because of the planning restrictions, the structural zone was limited, this time to 350 mm. No drops were permissible beneath that zone and the structural options were therefore confined to some form of flat slab construction. The solution adopted was a traditional flush soffit slab spanning transversely onto longitudinal beam strips. The external column grid emerged, from architectural considerations, at 6 m, while, to allow for a central corridor, the internal column line was placed asymmetrically, giving 7·5 m and 6 m bays across the building.

So different are the three blocks in terms of mass, number of storeys, internal configuration, and grid arrangement, that they have been separated structurally by movement joints.

Substructure options

Usually it is possible to obtain some information about existing construction from records held by the original designers, the local authorities or the building owner. In this instance a few drawings were located in the Public Records Office and the London Borough of Camden's archives, but they related mainly to alterations to the superstructure and gave few details of the original construction. Similarly, the minutes of the Canal Company were most interesting to read but gave no details of the canal construction.

Clearly, for a building as massive as the Granary, there was every likelihood that there would be substantial foundations. As the building had covered the majority of the site up to the embankments at the north end, and seemingly had stood successfully for some 120 years, one of the first questions to be answered was whether it would be possible to reuse those foundations for the new development.

Characteristic of the period when developments such as the Granary were built was the practice of excavating the site to a common level and then covering the excavation with a substantial bed of concrete. From that excellent working platform the new building would be raised with the walls and columns being founded on some form of spread footings. However, the arrangement of internal columns for the Granary at 4·2 m square was a total mismatch with that for the new sorting office which produced much higher column loads on the 12 m square grid. Hence the best that could be looked for was to design pad foundations using the concrete raft, if it existed, as the equivalent of a generous layer of blinding concrete. However, calculations showed that pad footings would need to be very large and from costing exercises were shown to be uneconomical compared with bored pile foundations. Hence the latter were adopted, with the pile caps located above the existing foundation level.

At the north end of the site in the embankment area, the requirement was to level the site by removing some 8 m of fill material and the associated massive retaining walls. Further excavation would follow as necessary for the foundations to the administration block and part of the adjoining welfare block. It was recognised that heave would result in the London clays, and early calculations indicated that, in the worst case, this could be as much as 150 mm. Obviously, normal pad footings, which otherwise could have been used, would have been quite inappropriate in such conditions and some form of special foundation would be required. The decision was taken to use piling with the piles sleeved over their upper lengths, and to protect the pile caps, ground beams, and suspended ground-slabs, by the extensive use of Clayboard.

There remained the problems of how to deal with the boundaries of the site. Clearly, it was necessary to have regard to the damage that might occur to the Atlantic Metals building on the northern boundary as a result of ground heave, and it was decided that measures would be taken to isolate, as far as practicable, the ground beneath this building from that on the Granary site.

Although the highest level of protection would have been achieved by the use of a tied back diaphragm wall, this proposal was abandoned as it would have involved works outside the site boundary. Further options were considered and the choice eventually lay between a freestanding diaphragm wall and a contiguous piled wall; costing of alternative schemes led to the selection of the latter. In providing the contiguous piled wall with piles embedded some 25 m deep, it was intended that they would act not as a retaining wall but would provide pinning down action, holding down the upper heave zone to reduce the vertical movement on the area of the Atlantic Metals site.

Alongside the canal the problem was different in that the development of the site required the material against the canal wall to be excavated, effectively undermining it so that permanent support would be required from the new works. It was also recognised, ahead of the site investigation, that the construction of the wall was in some places less than robust, its watertightness somewhat doubtful, and that great care would be necessary during construction to avoid causing further distress. To overcome these problems the decision was taken to provide a line of support inside the site positioned as close to the canal wall as practicable.

Various options were considered, including a diaphragm wall, a contiguous piled wall, and a sheet piled wall. The diaphragm wall was discounted as, for the majority of the length of the canal, it would have had to be constructed through approximately 3 m of rubble fill. Guide walls to clear these obstructions would have had to be over 4 m deep and their construction would have endangered the stability of the canal wall. Sheet steel piling was rejected, as it was considered that the heavy driving likely to be necessary would in all probability further weaken the canal wall and in any case there were noise restrictions on the site inhibiting the use of normal pile driving equipment. The chosen solution was to employ contiguous bored piling inside the full length of the canal wall, even though there would be practical difficulties in clearing the brick rubble fill.

The site investigation

The site investigation was designed to achieve two principal objectives. The first was to determine the nature and characteristics of the underlying soils, while the second was to discover, as far as practicable, the details of the exisiting foundations and boundary construction. It was also intended to determine whether there was leakage through the canal wall and standing or moving water on the site.

Existing foundations and boundary wall construction

A trial pit investigation was carried out, comprising some 22 pits (see Fig 5). Most pits were machine dug but, because of the depth of fill materials, exploratory timbered shafts up to 8 m in depth were sunk at the north end of the site, particularly to determine the construction of the canal wall. Even though the locations of the pits and shafts were chosen to reveal all likely forms of boundary wall construction, it was not possible to explore every detail of the existing structures on such a large site, and in one area in particular this led to complications during the building works.

The trial pits revealed that the foundations to the Granary building had indeed been made by excavating to a common level, approximately 6 m below canal water level. A hydraulic lime concrete raft, some 750 mm thick, had then been placed over the whole 0·8 ha building area, above which bases to the cast-iron columns had been formed by positioning several layers of 225 mm-thick massive sandstone blocks on a 4·2 m grid. Infilling above the concrete raft and around the sandstone blocks had been carried out with approximately 1·2 m of clay on which a brick sett floor had been constructed. Standing water was encountered on the brown London clay at the underside of the concrete raft.

The trial pits also revealed the construction of the canal wall. Over the length of the Granary building the wall was 1·6 m thick, supported on mass concrete footings. It obtained lateral support from a second inner wall through a slab forming a gallery at an intermediate level. To the north of the railway bridge the wall showed evidence of several generations of construction with relatively shallow foundations and considerable repair work. The abutments to the railway bridge were formed in massive brick piers, in turn supported on sandstone blocks. No water penetration of the canal wall was observed in the trial holes adjacent to the southern section. However, those along the northern section revealed that water was leaking through the wall in several locations.

On the southern boundary another brick retaining wall some 5 m high supported the recently renamed Granary Street and made possible a lower cartway.

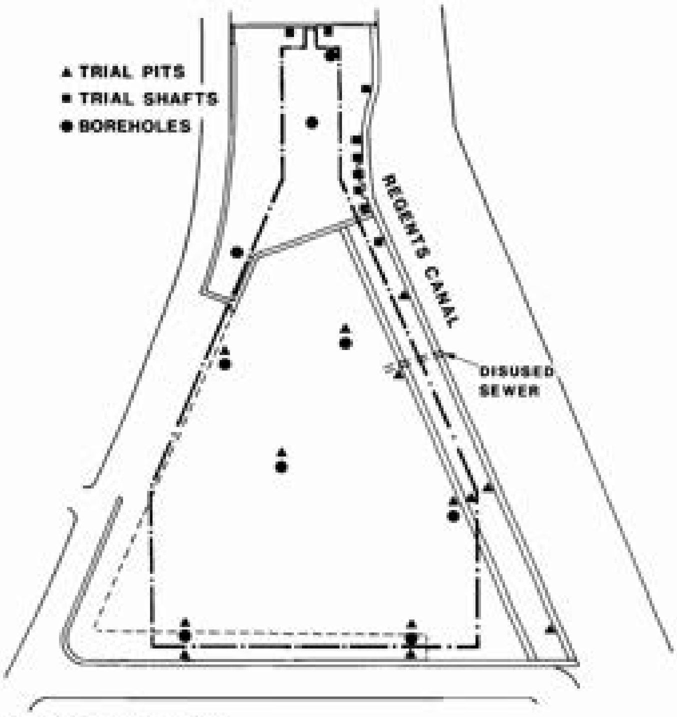


Fig 5. Site investigation

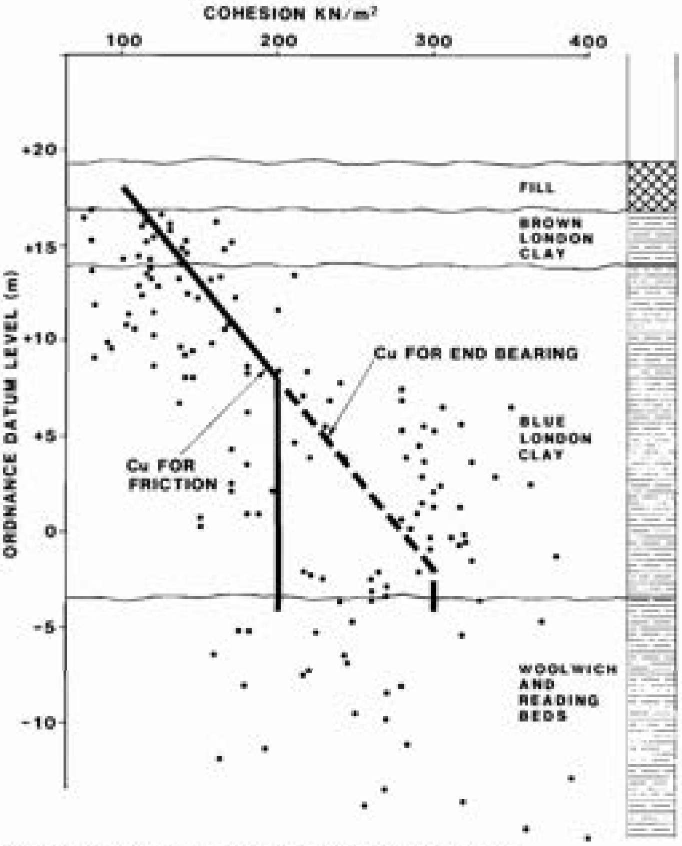


Fig 6. Cohesion values determined by site investigation

Calculations showed that many of the retaining walls had relied on the dead weight of the structure above to give them stability. This had been appreciated during demolition of the Granary as a large rubble berm had been left against the canal wall and temporary steel propping had been inserted to support the Granary Street and railway embankment retaining walls.

Borehole investigations

Nine boreholes were drilled (see Fig 5). Undisturbed soil samples were taken for laboratory testing from each borehole at intervals of 1·5 m depth. As anticipated from reference to the records of other nearby developments, brown London clay was found immediately beneath the oversite concrete of the former Granary building. That overlaid blue London clay, approximately 20 m thick, which in turn overlaid the Woolwich and Reading beds, these being clay in this area. Triaxial tests on the clays indicated cohesion values ranging from 50 to 400 kN/m^2 increasing in strength with depth but with a wide scatter (see Fig 6). The clay had few sand lenses and the bores were dry, except for one or two small incursions. Consolidation and expansion tests were carried out on selected borehole samples to determine the coefficients of compressibility and expansion of the clay from which an assessment could be made of the likely amount of heave.

Foundation design

Ground movements

During the feasibility study it had been recognised that removal of the railway embankment at the north end of the site would give rise to ground heave and that the design of piled foundations in that area would need to take account of such movement. At that time the principal concern was for vertical movement, but during the design stage it was further recognised that the removal of such a major surcharge load would also result in the ground recovering from its horizontal displacement. Despite an extensive literature search, very little information on the subject was located, probably attributable to the fact that ground heave is more normally encountered in deep basements and relates to the floor only. However, unlike basement construction where retaining walls are provided with effective restraint by the slab, in this project minimal restraint would be provided by the proposed piled foundations. Thus movement in the soils would continue until equilibrium conditions were reached, by which time structural failure of the piles would probably have occurred.

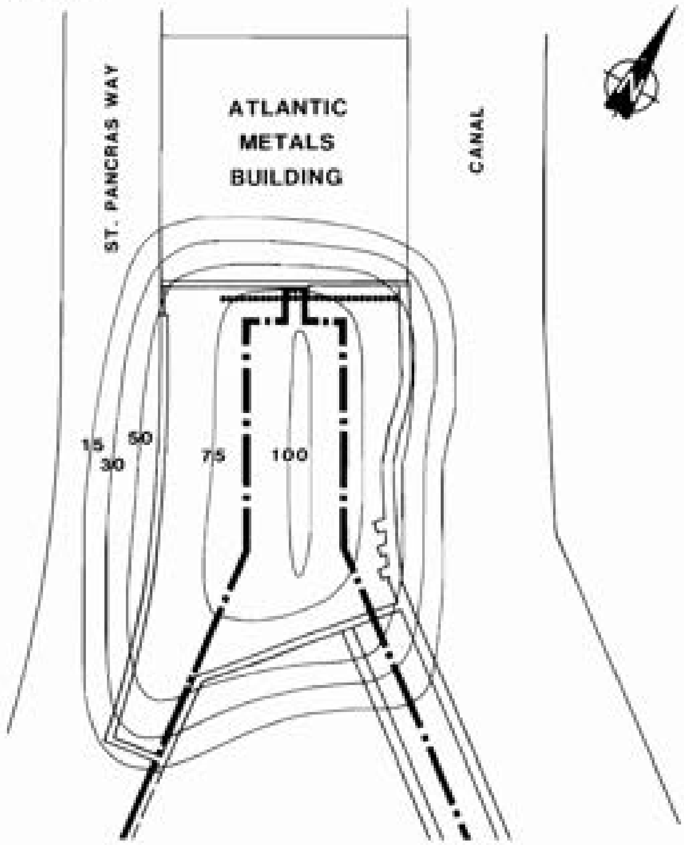


Fig 7. Anticipated unrestrained ground movement—vertical

Analyses were undertaken to determine the likely magnitude of both vertical and horizontal soil movements with regard to their time dependency. These movements were considered independently, using consolidation theory to assess the vertical heave, while an estimate of the horizontal movement was made by considering a simple stress path analysis.

Unrestricted heave was shown to be in the order of 100 mm at the centre of the area from which the embankment had been removed, reducing to about 50 mm at the boundaries of the site (see Fig 7). However, the restraining action of the piles for the new building would significantly reduce these figures by about half.

The horizontal component of the ground movement was estimated to be in the order of 20 mm at the perimeter of the administration block at surface level, reducing to virtually no movement at 20 m depth (see Fig 8). It was estimated that between one-third and one-half of this movement would occur within the first few months following removal of the surcharge, so clearly it would be advantageous to delay the construction of the foundations in this area for as long as possible.

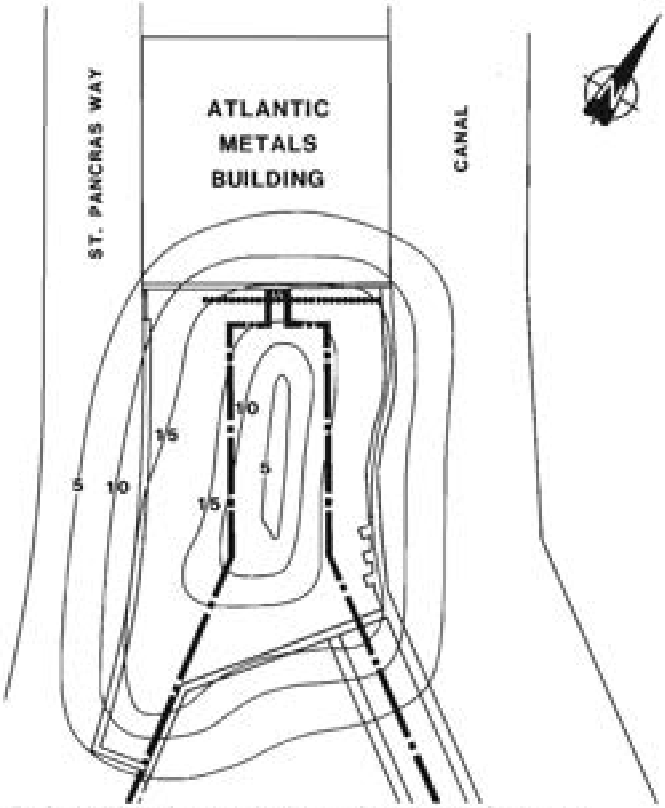


Fig 8. Anticipated unrestrained ground movement—horizontal

Design of foundations in ground movement areas

An analysis of the ability of the clay to flow horizontally around the piles indicated that this would be of low order and that the induced shear forces and bending moments at the top of the piles could become critical, leading to shear failure in them at the undersides of the pile caps. The literature search had revealed no reports of structures where measures had been taken to overcome this problem, except perhaps in the case of houses with deep strip footings in desiccated clay soils, where it is common to provide cross trenches internally to relieve horizontal swelling pressures. Such measures would have been unsuitable in this instance as the depth of trenches required would have been excessive and their presence in the vicinity of the canal wall would have seriously endangered its stability.

Clearly, the proposal for sleeved piles arising from the feasibility study, and conceived to cater for vertical ground heave only, was now inadequate and a different approach to protecting the piles was required.

The solution adopted was to create a zone immediately around each pile and of sufficient depth that the horizontal ground movement could take place in an unrestrained fashion without imposing excessive lateral loading on the piles. Calculations showed that this latter condition would be met if the annular space extended over the first 4 m of the piles and was bored some 100 mm oversize on the pile diameter. It was decided that the resulting void should be filled with bentonite both to prevent the clay surface from softening and also to inhibit construction material falling back into it. The design of the piles could now be completed making due provision for the effects of heave until such time as the loads from the new construction produced a compensating effect.

The piles supported caps which in turn received ground beams carrying the suspended ground-floor slab, the whole construction being protected against heave by 100 mm thickness of Clayboard (see Fig 9). The pile caps and ground beams were designed to resist the bending moments and shear forces induced in them by the lateral deflection of the piles. As the horizontal ground movement along the centreline of the administration block would be negligible, it was decided not to sleeve the piles adjacent to this line and to take the wind forces through them to the ground.

Foundations around the TWA sewer

The sewer is also located in the area likely to be affected by ground movement. Consideration was therefore given to the order and rate of change of vertical displacement which might occur along its length, but the view was reached that it could accept the calculated distortions without distress and accordingly no special protective measures were adopted. Its alignment also coincided with the stair and lift core at the south end of the administration block. There, at foundation level, beams positioned to allow the formation of the lift pits were designed to bridge the sewer spanning onto pile caps running along each side. Their supporting piles lay within the defined zone of significant vertical and horizontal ground movement, and also had to meet the TWA requirement that no load from the new construction be imposed on the sewer. Accordingly, they were designed to be friction-free down to invert level,


Fig 9. Clayboard protection to ground beams

to be achieved in construction by overboring and inserting within the bore an externally slip-coated permanent steel sleeve. The overbore annulus was to be filled with bentonite/cement grout up to a level 4 m below the pile caps to give the pile lateral support, with bentonite filling only the space above.

Foundations outside ground movement areas
The piled foundations in these areas, comprising the whole of the sorting hall and about half of the welfare block, were designed as standard 600 mm friction piles of approximately 20 m length bottoming just above the Woolwich and Reading beds. The design of the ground-floor construction was carried through as determined in the feasibility study.

Ground works adjacent to the Atlantic Metals building
In the feasibility study, provision was made to minimise the effects of vertical heave on the Atlantic Metals building by introducing a contiguous piled wall just within the Granary site. While this concept was in itself satisfactory, the requirement to consider the possibility of horizontal movement in the underlying clay now gave rise to the necessity to reappraise the aptness of such a wall.

The piles, of nominal 600 mm diameter, were envisaged as being designed unrestrained laterally but, being relatively flexible, they were liable to deflect with the movement of the earth mass, thus doing little to prevent horizontal earth movement. Accordingly, the possibility of substituting for the contiguous piles with a stiffer form of construction was reexamined, but was found to be most unattractive on cost grounds.

An assessment was then made of the effect on the Atlantic Metals building of horizontal earth movement in the range of 10-20 mm, this being the anticipated deflection at surface level of the contiguous piled wall, and it was judged that the induced strains could be tolerated by the structure without distress. It was decided, therefore, to continue with the installation of the contiguous piled wall and to monitor the ground movements locally.

Ground works at the canal wall
The site investigation had revealed that the construction of the canal wall was suspect, particularly at the north end where it was for the main part founded at relatively high level. In the feasibility study the favoured solution was to bore a contiguous piled wall through the rubble berm adjacent to the canal. However, as a result of the doubts about the integrity of the wall revealed by the site investigation, the options for supporting the wall were reviewed.

Alongside the Granary the canal was unusually wide, reflecting the earlier construction of the covered loading wharf, and the idea of constructing a wall within the canal became attractive. Approaches to the canal authority were well received and, as a result, approval was given for a new permanent sheet steel wall to be constructed in the canal 1 m from the wall face over the whole length of the site and sealed back into the existing wall at each end. The sheets were to be driven by a silent and vibrationless pile-driving system and the new wall was designed to act as a cantilever founded in the London clay. At the location of the TWA sewer the much shorter piles were assumed to act as propped cantilevers supported by a walling carried to high modulus cantilever piles on each side of the sewer. In the case of the small sewer, which was only just below the bed of the canal, it was decided that the risk of the pile driving

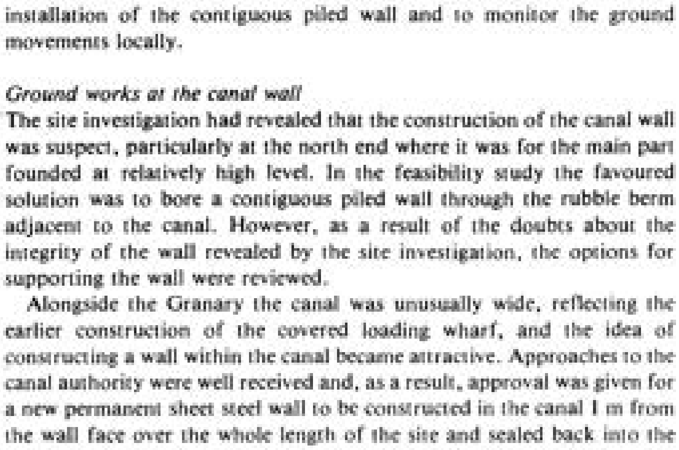


Fig 10. Section through canal wall in heave zone

causing collapse of the sewer was too great to accept. Accordingly, as the sewer was disused, provision was made for it to be filled with PFA grout so that the piles could subsequently be driven down to it.

A walkway was to be provided between the canal and the completed building (see Fig 10). It was to be about 2 m above the ground-floor level of the new building, and was to be retained by L-shaped free-standing reinforced concrete walls. Since the drains in this vicinity would be vulnerable to ground movement, shelves, protected by Clayboard on the underside, were designed to cantilever from the bottom of the perimeter ground beams with the drains and manholes built on them. However, that construction required deep excavations at the foot of the canal wall, so that, in certain areas, its stability was examined in relation to slip circle failure. This was viewed as being more than just a short-term problem, since the purpose of Clayboard is substantially to leave a permanent void. To restore some of the surcharge lost by forming the deep trench, it was decided to substitute a stiffer material for the Clayboard locally under pile caps and ground beams. Laboratory tests were carried out on selected grades of expanded polystyrene which showed that, once the initial yield stress of the material was overcome, creep effects became significant and compression would continue but at a reduced rate. However, the residual strength in the polystyrene was sufficient to enable a small surcharge to be generated. The presence of this material had the additional benefit of inhibiting the collection of water in the void which would have had softening effects on the clay.

The final boundary condition to be considered was that adjoining Granary Street. With the benefit of the information gained from the site investigation (i.e. the nature of the masonry, the wall thicknesses, and extent and position of foundations), it was possible to analyse the stability of the wall. On completion of the development some 2 m of fill material was required to be placed which would improve the wall stability sufficiently to enable the raking shores inserted at the time of demolition to be removed. However, as a slight movement of the wall would occur if the shores were removed, it was considered prudent to leave them in place to avoid the possibility of damage to the road and services running in it. To increase the life of the shores, which would be buried, they were to be encased in concrete.

Superstructure
The building was given visual cohesiveness by its principal cladding material, a curtain wall system of glass and stove enamelled sandwich panels (see Fig 11). As an exception, however, in the two-storey carpark and the lower level of the sorting office, closely spaced precast concrete mullion units were incorporated, thus allowing natural ventilation to occur in vehicle movement areas. The roof cladding was a shallow pitch corrugated aluminium system, clipped to supporting members from below, thereby avoiding the need for through fixings and also having the benefit of being free to move thermally.

The final design developed the structural form determined at feasibility stage. In the sorting office the reinforced concrete frame was carried up to first-floor level (see Fig 1) and locally beyond to form the central plantroom. The slab at first-floor level was of conventional solid reinforced concrete construction, 175 mm thick, designed for a superimposed load of $5 \cdot 5$ kN/m^2 and carried by secondary reinforced concrete beams at 4 m centres. Openings were provided to allow for the installation of chutes and conveyors to take the mails between the sorting office and the receipt and dispatch areas in the loading bays at ground floor. The secondary beams were supported by primary beams normally 1·5 m deep 0·53 m wide, carried to columns on the 12 m grid. However, where the columns were offset in the vehicle movement area at ground floor, the span of the primary beams over was increased to 16 m and their size to 2·5 m by 0·5 m so as to carry the first-floor columns.

In the sorting office the concrete columns continued up to roof level to provide the seating to the structural steel shallow roof pitch of overall dimensions some 73 m 78 m. Provision was made in the bearings at the column heads to accommodate out-of-position tolerances. The external concrete columns were designed as cantilevers against lateral loading. The roof steelwork therefore required no fire protection, since it gave no lateral restraint to the enclosing walls. The steelwork was blast cleaned and then painted with a zinc phosphate epoxy paint system.

Beneath the sorting office roof a steelwork subframe was suspended to carry the mechanical and electrical services (see Fig 12). A further subframe was provided to the side walls to give lateral support to the curtain walling, necessitated by the high head room and wide column spacing.

Fig 11. Canal elevation cladding nearing completion

Fig 12. Services subframe in the sorting office

Fig 13. Canal elevation structural frame nearing completion

In the welfare block, there was conflict between the ideal structural grids for the user and carparking facilities. The lower two floors of carparking were served by ramps. The floors were supported by a grillage of 750 mm × 1000 mm beams at 4 m centres spanning 12 m onto 900 mm 1500 mm edge beams of similar span. A nominal 175 mm solid concrete slab formed the floor deck with an additional 12 mm concrete thickness as a wearing allowance. The upper floors of the block were of similar framing, but with 200 mm traditional hollow clay pot ribbed slabs. The structural steel roof system was similar to that for the sorting office.

The administration block structural arrangement remained as envisaged at feasibility stage with the slab panels again being formed in hollow pot construction. The top storey comprised a series of steel portal frames (see Fig 13) clad in curtain walling and aluminium roof decking. The reinforcement from the lower columns was carried through into the concrete casing of the portal frame legs to provide stability under fire conditions. This enabled a waiver to be obtained from the Greater London Council against the statutory requirement to provide fire protection to the portal rafters.

The construction

Although the scheme has been with the consultants since January 1980, the Post Office was unable to give instructions to proceed to full design and construction until February 1983. At the same time a completion date was set for June 1985, with the earlier date of March 1985 for the sorting hall so that the letter-sorting machinery could be installed and commissioned. The contract was to be the normal JCT 1980 with Quantities edition. Assessment by the consultants made it clear that, allowing for a 2-year construction period, there was insufficient time available to permit a full design to be completed and normal competitive tenders to be obtained for a start on site in June 1983.

Clearly, early construction would concentrate on overcoming the problems arising from building on a derelict site, many of which would benefit from being worked on during the better weather conditions of the summer months. There was also the requirement to remove the railway embankment at the earliest possible time so that ground movement could take place for as long as possible prior to the permanent works being undertaken. Accordingly, it was decided to run a preliminary contract of 16 weeks' duration for the piling and earthworks, but with stage completion at 12 weeks when sufficient piling would be installed to enable the second and main contractor to commence work.

In recognition of the fact that the majority of the works in the first contract were piling operations, it was decided to offer the work to piling contractors, who would acquire such other support as they deemed necessary for the general works of breaking out, demolition, and ground excavation. It was intended that, after week 12, the first contract would be assigned to the main contractor who would take on the piling company as his subcontractor with the parties now in their traditional roles.

In accordance with Travers Morgan's normal procedure the piling contractor was required to provide the piles and guarantee their loadcarrying capacity. The piles were to comply with a performance specification based on the consultant's design, compliance being proved by testing in the usual manner. It was during the tender period that the piling contractor, with his advisers, offered an alternative design for the contiguous piles which allowed for 600 mm diameter piles to be spaced at 1 m centres.

The above proposal was accepted and the piling works commenced on time. It was during the early stages that problems from the lack of complete knowledge of the site history and ground conditions manifested themselves. Alongside the Atlantic Metals building the contiguous pile wall was to be installed from the top of the embankment prior to its removal. As the work progressed away from the canal wall, massive masonry construction was discovered which made normal pile boring impossible. The problem was overcome by using a heavy coring machine which enabled an oversize metal liner to be inserted through the masonry into the natural ground and through which the piles were bored and cast.

The intention to monitor ground improvement adjacent to the Atlantic Metals building was put into effect. Inclinometer tubes were installed in three piles at approximately 10 m centres and a baseline was established for reference. Additionally, level pins were fixed to the wall of the building. Monitoring of the inclinometers has shown slight movement of the wall towards the Granary site, but they have not risen, suggesting that the pinning action is successful.

As described, the main bored piling was provided by the piling contractor to a performance specification. A total of 460 piles was proposed, with diameters of 500 mm or 600 mm, having lengths varying from 14 m to 21 m into the clay. The piles were to carry vertical loads in the range of 500 kN to 1700 kN and, additionally, certain of the piles were designed to resist horizontal wind forces of up to 80 kN. Two preliminary load tests were undertaken to prove the design and results from the first test on a 600 mm diameter pile, 21 m long, met the settlement criteria under both working and twice working load. The second test, on a 500 mm diameter pile 14 m long, met the settlement criterion at working load but at twice working load just failed to achieve the appropriate value.

From the two sets of test results it was judged that there had been an overestimation of the cohesion of the upper levels of the clay, and the lengths of the shorter piles were increased by up to 1 m. A further preliminary load test confirmed the adequacy of this revised design. Subsequent tests on working piles also proved satisfactory, these being taken up to 1·5 times working load.

The installation of the working piles was completed successfully, although, as is not uncommon, a few were found to exceed the positional tolerance of 75 mm. Piles designed to resist vertical heave forces required heavy reinforcement cages throughout their full length but, where sleeving was provided over the upper 4 m, the amount was reduced. Since the piling rig was unable to cope with the size and weight of the cages, additional craneage was brought to site.

Forming the annular space to protect the piles from horizontal soil movement was generally straightforward. However, problems were encountered in some areas of made ground where it was necessary to top up the bentonite, presumably because the open nature of the fill provided an initial path for leakage.

Alongside the canal wall the installation of the expanded polystyrene beneath certain pile caps and ground beams presented no problem. However, before placing the much larger quantity of Clayboard under the rest of the construction in that area, precautions were necessary to prevent its premature collapse. This could have occurred from moisture ingress prior to the completion of the permanent concrete works immediately above. The required protection was achieved by sealing the slabs of Clayboard in large polyethylene bags. Tubes were then led from each bag through the concrete slab to enable water to be introduced subsequently into the Clayboard to assist in its collapse.

With the difficulties in the ground overcome, the construction of the superstructure progressed well (see Fig 14) with only occasional problems arising. One of these concerned the contractor's intention to cast the 7m-high columns of the lower storey of the sorting office block in one lift. To facilitate compaction of the concrete the contractor adopted a mix incorporating a superplasticising additive on the advice of a specialist supplier. However, early results were disappointing, column faces showing signs of excessive bleeding and segregation of the concrete mix. After taking further expert advice the contractor reverted to a standard concrete mix and cast the columns in two lifts.

The concrete for the project was premixed and there was frequent sampling of deliveries. Test cube results, in general, showed 28-day strengths much higher than the specification requirements. Principally, this resulted from the need to incorporate in the mix, for reasons of durability, a larger quantity of cement than was required for strength alone.

Fig 14. Aerial view—superstructure in progress

As soon as the concrete works for the sorting office block had been completed, the erection of the structural steelwork for the roof began. Both the fabrication and the erection of the steelwork might have been expected to present difficulties owing to the complex geometry of parts of the building, particularly the roof. However, the work proceeded uneventfully and the usual concern about the fit between the concrete and steel frames proved unfounded.

The works remain on programme for completion by mid-summer 1985.

Acknowledgements

The authors are grateful to Mr Derek Reeves, Controller, Buildings & Management Division, London postal region, for his agreement to the publication of this paper.

Client: The Post Office

Design team:

architect:	T. P. Bennett & Son
quantity surveyor:	Cyril Sweett & Partners
structural engineer:	Travers Morgan & Partners
services engineer:	Engineering Division, Major Projects, London postal region

The main contractor is Costain Construction Ltd. and the piling contractor was The Expanded Piling Co. Ltd. The structural steelwork subcontractor was Boulton & Paul Ltd.

UDC 725.16:69.054

Discussion

The Granary site—design and construction of a mechanised letter-sorting office

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A. H. Dutton, MA(Cantab), CEng, MIStructE, MICE

This paper was presented at a meeting of the Institution of Structural Engineers, 11 Upper Belgrave Street, London SW1X 8BH, on 18 April 1985, with Professor E. Happold, RDI, BSc, FEng, FIStructE, FICE, FCIOB, HonFRIBA (Vice-President), in the chair, and published in The Structural Engineer, *Vol. 63A, No. 4, April 1985, p109* et seq.

Mr D. A. Langdown (F) (Costain Construction Ltd.): First, I would congratulate the authors on their paper which describes a job with a number of very interesting features, particularly in relation to ground works.

From the contractor's temporary works point of view, the initial problem was the stability of the canal wall during excavation works at the rear. As little was known about the condition of the wall, which varied considerably in construction and condition along its length, the analysis for stability had to be checked in numerous places. Generally, by restricting the lowest level excavations for bases at the rear of the wall to isolated pockets on the 'hit and miss' principle, it was possible to construct the work without propping the wall.

The temporary stability of the canal wall did, however, rely on maintaining a dry 'void' between the line of sheet piles in the canal and the old wall. This is the space which is shown finally filled in on Fig 10 of the paper.

One emergency situation arose during the construction of the ground works when, because of an operating failure in controlling the canal water level, the canal water unexpectedly rose and over-topped the row of sheet piling and started to fill up this void between the sheet piles and the wall, thus endangering the wall stability.

Pumping that had already been installed to deal with leakage through the sheet pile clutches was fortunately on hand to keep the surcharge on the wall from water pressure to a minimum, but, as a matter of site safety, work adjacent to the canal was halted until normal conditions had been restored.

One significant programming problem that we had was the need to construct the first-floor beams and slabs before the ground-floor beams and slabs were cast and cured. In the areas where the *in situ* ground was poor and the ground-floor slab was itself supported by piles, the temporary works to support the first-floor framwork were very extensive and a grillage of temporary steel beams between pile caps was needed to carry the falsework for the first-floor formwork.

In other areas, as the first-floor wet concrete load was greater than the design load of the permanent ground-floor slab, a number of loading tests were undertaken on the ground to assess likely settlement and allowable groundbearing pressures. The problem was partly overcome by casting the heavy beam section first and the slabs later.

I would like to refer to the casting of the 7m-high columns about 600 mm square, which, as the paper says, we originally hoped to cast in one pour, using Cormix SP1 'superplasticiser'. This did, however, result in some segregation in some of the columns. It was considered that revising the mix design to include more sand might have helped the situation, but this was not possible in the time.

Finally, three sides of the column shutter were erected for full height and the fourth side half height; the concrete was placed for half height and vibrated and then the remaining shutter fixed and the concreting completed to the top.

The Cormix 'superplasticiser' was used successfully elsewhere on the site for concreting the strongroom walls which were 350 mm thick and 3 m high and exceptionally heavily reinforced.

I was very interested in the figures quoted for the anticipated heave of the soil.

I peruse a large number of soil reports during the course of a year and I am often surprised at the relatively high figures quoted both for ground heave during demolition and excavation and for the long-term differential settlement of pad foundations, sometimes in what appear to be fairly good soils.

I wonder whether the geologists who write these reports realise how many buildings are piled specifically because of the large settlement figures predicted by their theory. I would therefore be pleased to learn whether any records exist of the actual ground heave experienced in practice to compare with the figures predicted by theory.

Finally, noting that fairly large spans were required in some areas, I wonder whether prestressed concrete was considered as an option for the construction.

Mr Dutton: I agree that many soil reports predict large ground movements for both heave and settlement and that it is important to have as many records as possible to determine whether the predictions are borne out in practice. At present, there is very little information on actual movements generally available.

We certainly considered the possibility of monitoring ground heave on site. However, when we assessed the practicality of so doing, it became apparent that, owing to the complexity of needing to relocate continually the levelling points as first excavation and then construction took place, it would prove an expensive and unreliable procedure. We therefore decided to concentrate effort on the more important aspect of monitoring the movement of the contiguous pile wall.

Mr Myers: Consideration was given, at the preliminary design stage, to the feasibility of using reinforced concrete, prestressed concrete or structural steelwork as the structural medium for the project. An early decision was reached that the most appropriate structure would be a hybrid with the longer roof spans in structural steelwork. More detailed investigation was undertaken for the remainder of the superstructure, and certainly structural steelwork, reinforced and prestressed concrete were considered for the longer-span beams of the sorting hall floor. However, particularly on the basis of costing advice, the use of reinforced concrete was finally selected as the most practical and economical solution.

Dr. B. Simpson (Ove Arup & Partners): At the north end of the site, an area of about 2100 m^2 was overlain by about 6 m of fill above the general site level, as indicated by the stippled area in Fig 3 of the paper. This fill was to be removed.

The contract required that the piling contractor was responsible for the prediction and limitation of ground movements so as to avoid damage to the proposed structure and to the adjacent Atlantic Metals building. Expanded Piling Ltd. commissioned Ove Arup & Partners to carry out a study of the possible ground movements, to advise on their effect on the adjacent building, and to design the piles for the new building, taking account of vertical and horizontal ground movement.

Finite element computations were carried out for this purpose, using linear elastic parameters. The assumptions adopted were pessimistic, being chosen so as to ensure a safe design rather than a 'best estimate' prediction of behaviour.

The possible influence of the removal of overburden was computed by considering removal of a circular area, 90 m in diameter. The contract required that a row of piles be built on the northern boundary of the site, as shown by the dotted line in Fig 7 of paper, before the overburden was removed. These piles were therefore included in the computational model. Referring to the undrained shear strength of the London clay, c_u, Young's modulus was taken to be 400c_u in the short-term (undrained) case and 130c_u in the long-term (drained) case. No account was taken of the stiffening effect of the foundation piles.

The computed horizontal movements for the line of piles at the northern boundary of the site are shown in Fig D1. It was found that the presence and length of these piles made no difference to the computed horizontal movements. This occurs because the bending stiffness of the piles is not significant compared with the stiffness of the deforming ground. The tensile stiffness of the piles is significant, however, and their effect on computed vertical movements can be seen in Fig D2. Following the method of Burland & Wroth[D1], tensile strains, ε, for the Atlantic Metals building were calculated, and it was shown that these were reduced below the critical value of 0·075 % when the row of piles was included in the computation.

In areas where the heave which would occur before the piles were loaded was computed to be more than 20 mm, tension steel was provided. This was designed using the conventional assumption that shaft adhesion would be equal to αc_u, but α was set to the pessimistically high value of 0·7. A permissible steel stress of 250 N/mm^2 was adopted in order to prevent excessive elongation and cracking of the piles. This combination of parameters is considered to be very conservative.

Piles subject to lateral ground movement were designed using a beam and spring analogue, in which the head of the pile was displaced, without rotation, by an amount equal to the computed lateral movement of the ground.

Ground movements were monitored by the engineer during construction, and it would be of interest to compare measured movements with the pessimistic computations described above.

Mr Dutton: As Dr. Simpson mentioned, we have monitored the movement of the contiguous pile wall. Three of the 15 piles had inclinometer tubes cast into them during their construction. Since then their movements have been monitored over a period of approximately 1 year, beginning with a base set just prior to the commencement of excavation and then at increasing intervals of 2 weeks, 2 months, 4 months, and 6 months. The monitoring was carried out by a specialist soil instrumentation company.

The movements of the piles have been small and are, in general, of the same order as the accuracy of the instruments. However, some trends are apparent and can be seen in Figs D3 and D4.

For movement to and from the Atlantic Metals wall (see Fig D3), inclinometers nos. 2 and 3 seem to indicate the movement as predicted by Dr. Simpson in that the tops of those piles have moved towards the Atlantic Metals wall in the short term. In the longer term, however, the piles have tended to drift away from the Atlantic Metals wall over their full depth, contrary to Dr. Simpson's prediction. Inclinometer no. 1 would appear to be the inevitable rogue and has gone its own way but then it is located very close to the canal and in that area the mound was excavated in stages with long time-intervals between. Virtually the whole of the movement in that pile took place in the short term with little change since. We believe the general movement away from the site may be attributable to the following:

(i) The proximity of the pile to the canal which exerted a continuous lateral pressure on the head of the pile above the excavated level elsewhere.

(ii) The Atlantic Metals building in that location, being two storeys

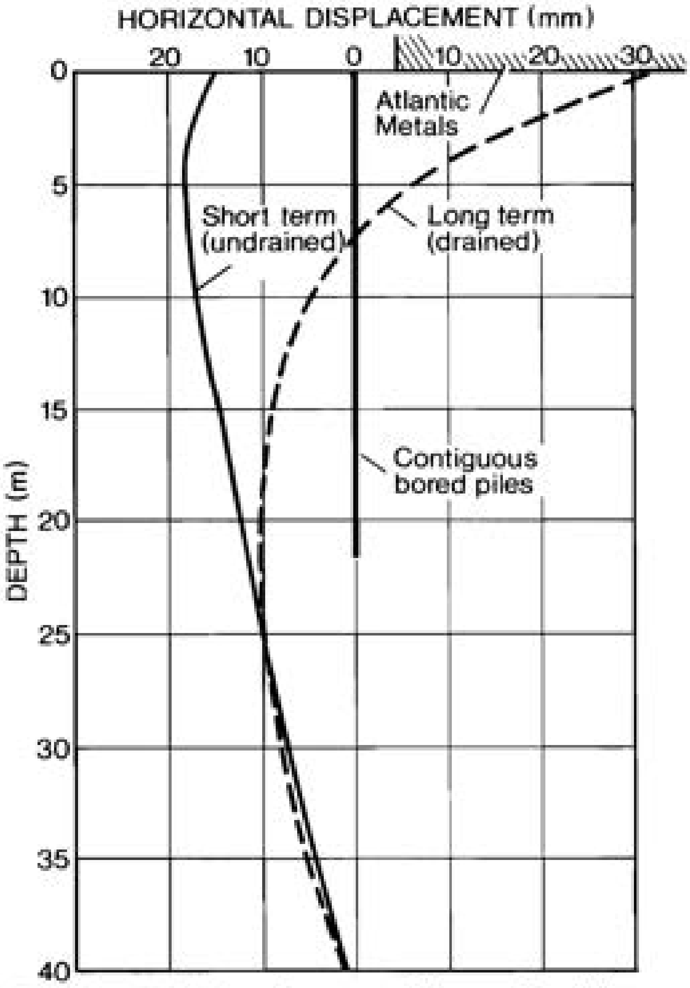

Fig D1. Computed horizontal movements of the row of bored piles

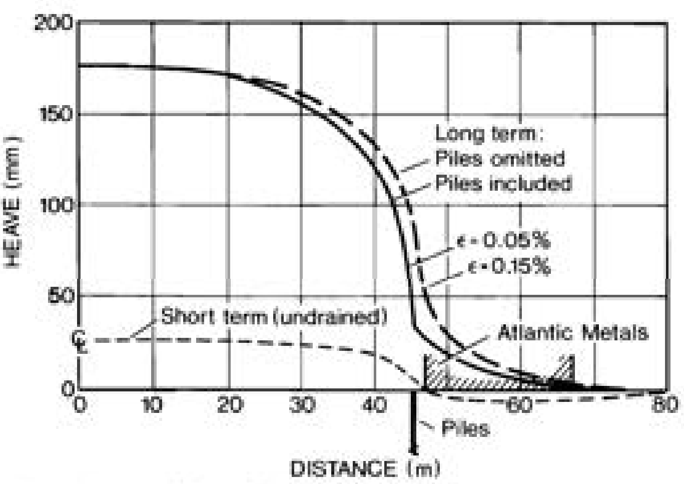

Fig D2. Computed heave of the ground surface

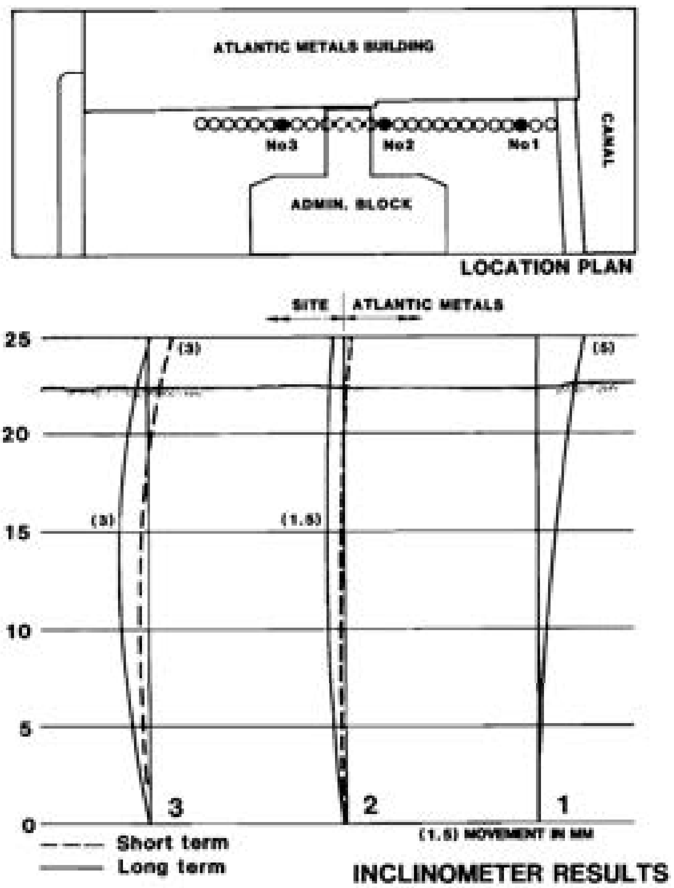

Fig D3. Movements to and from Atlantic Metals wall

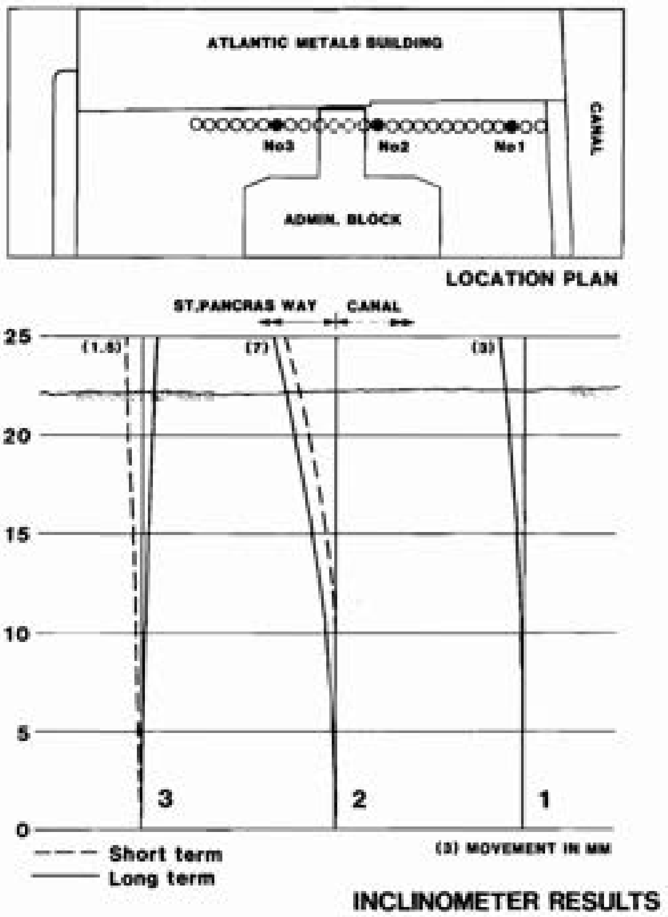

Fig D4. Movements to and from canal

instead of five, exerts a smaller loading and therefore provides less restraint to movement.

For movements in the direction to and from the canal (see Fig D4), the trend would appear to be for the outer piles to move towards each other, though the movement is quite small.

Inclinometer no. 2, however, has moved 7 mm towards St. Pancras Way. The majority of the movement happened in the early stages and is probably attributable to a deep pile cap excavation being made adjacent to the pile.

In addition to monitoring the horizontal movements of the pile, we also monitored the vertical movement of the Atlantic Metals wall. To do so involved relating levels to a safe datum some 75 m away, northwards up St. Pancras Way. Owing to a large number of change points being required, accuracy is no better than ±3-4 mm. Even so, a trend is apparent (see Fig D5).

To the left-hand side, the wall has risen in the order of 6 mm. To the right-hand side the amount is nearer 10 mm. That is probably due to the lighter loading of the Atlantic Metals building on the right-hand side. In the centre the movement is only 4 mm, contrary to the expectation that the amount of vertical heave would be a maximum at the centre. We consider that the discrepancy is probably the result of the local concentration of piles for the new building adjacent to that area restraining the vertical movement.

We also tried to monitor the vertical movement of the heads of the piles, but found that the accuracy of measurement was larger than the amount of movement, which was somewhere in the 0-5 mm range and is much less than the movement of the Atlantic Metals building. So the piles would appear to be performing their intended function of pinning down the ground, though some limited heave is still taking place on the other side of the wall. We are pleased to say that the vertical rise of the Atlantic Metals building is still less than the amount that Dr. Simpson calculated the building can tolerate before it is distressed.

Dr. Simpson also mentioned his approach to designing the piles subject to horizontal earth movement. The example he discussed was for piles without sleeving which were required to accommodate up to 10 mm horizontal movement. Referring to Fig 8 of the paper it can be seen that soil movements in the order of 0-20 mm were anticipated. Piles in the 0-10 mm movement zone were left unsleeved. Those included a group of piles down the centreline of the Administration Block. In areas where the soil was expected to move by more than 10 mm the piles were provided with an annular space around the upper 4 m to permit the soil to move without exerting what would amount to a very large lateral force on the pile. The annular space was formed as shown in Fig D6 and as described in the paper. Even with that sleeving, we estimated the shear force induced at the head of the pile would be up to 75 kN and the bending moment up to 150 kNm. Those are not inconsiderable forces and moments which needed to be accommodated by the substructure. They were resisted by making use of the grillage of ground beams.

Of particular difficulty were the piles adjacent to the TWA sewer (see Fig D7). The TWA had stipulated that the piles should not exert any load on the sewer and that they must be defrictionalised down to the level of the sewer at a depth of 6·6 m. An annular space was formed around the piles in a similar way to the piles subject to lateral ground movement.

However, it was felt that the pile should receive some lateral support, so the lower 2·6 m of the annulus was filled with a bentonite/cement grout. The sleeving was defrictionalised by coating it with a layer of visco-elastic bitumen compound which was stiff enough to resist damage during the placing of the sleeving but had considerable long-term creep capacity.

Mr G. R. Holland (The Expanded Piling Co. Ltd.): By most standards, this was a somewhat unusual contract for a piling contractor to undertake, especially one whose main operations are to construct large diameter piles using crane-mounted auger rigs.

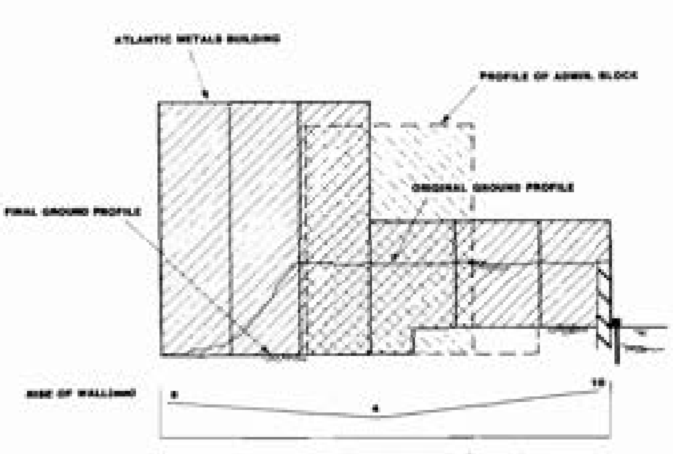

Fig D5. Vertical movements along Atlantic Metals boundary

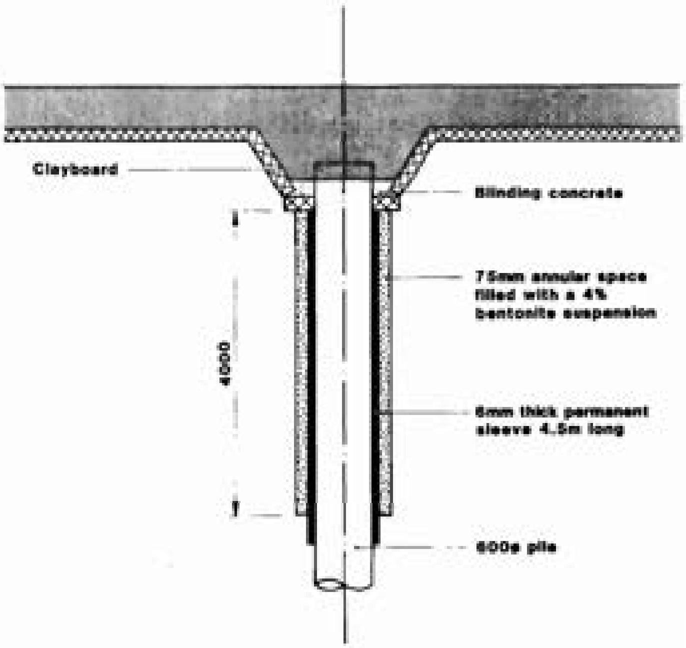

Fig D6. Sleeving to piles: typical detail

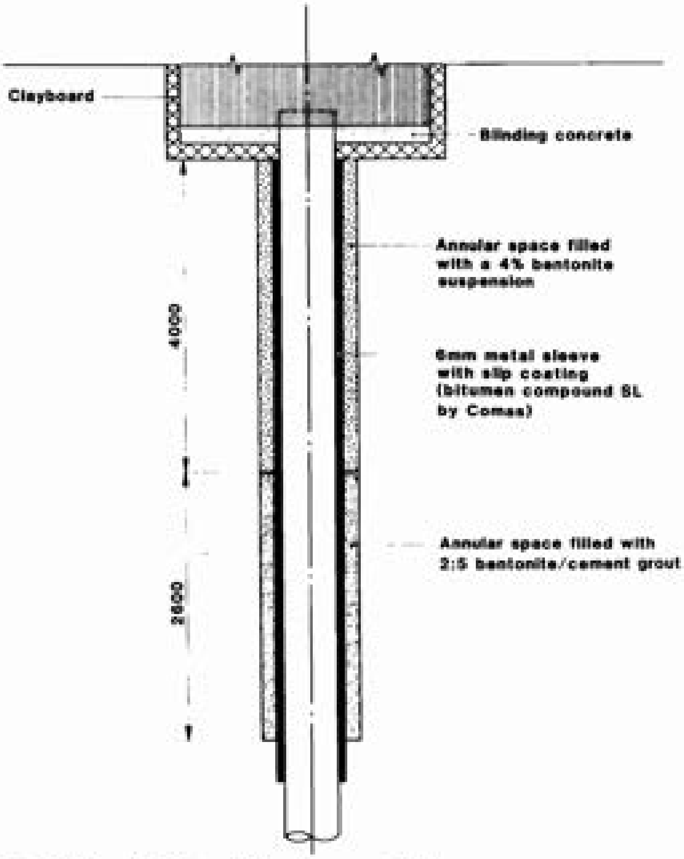

Fig D7. Detail of sleeved pile adjacent to TWA sewer

There were a number of ancillary works, whose total value exceeded the value of the actual bored piling. The site preparation and the steel sheet piling, together with the preliminaries, constituted the bulk of this, and an arrangement was made between the Expanded Piling Co. and Messrs Lilley Construction that all work other than that immediately associated with the bored piling should be carried out by Lilley Construction. This was a rather unusual arrangement where the main contractor's attendances and other works were carried out as a subcontract. In fact the arrangement worked quite well. The Expanded Co. administered the contract and dealt with all contractual matters throughout.

The subsequent contractual arrangement, described in the paper, of assigning the contract to the subsequent main contractor did, however, present administrative problems. In retrospect, it might have been better for the piling contract to have been completed under the orginal arrangement. It is not uncommon these days for more than one contractor to be working on the same site, each with 'trade contracts' with the employer.

Technically, the main problems arose in dealing with the design of piles to resist heave and consequent vertical and lateral loads on the piles.

There had been some slight problem with the basic design of the piles, to carry vertical loads. Problems do tend to arise when the engineer's specification requires the pile to achieve a limited settlement at the designed ultimate load. It seems to me that a pile designed for a specific factor of safety theoretically should just fail at that factor of safety. Consequently, with accurately designed piles, to achieve a limited settlement, when the pile is at the point of failure, is, to say the least, tricky. It would seem preferable to have a specified performance at (say) working load and $1 \cdot 5 \times$ w.l, and then to prove the ultimate load with a CRP test.

The question of limiting heave was a much more complicated matter. In the event, we engaged the services of the Geotechnical Section of Messrs Ove Arup & Partners to carry out this design work for us. I think that it is not always fully appreciated by the professional team what difficulties these rather more complicated matters present to piling contractors, especially at the time of tender. There is quite a different approach by different consulting engineers on these questions of design. Many piling contractors do not have the resources, in-house, to carry out all of the detailed designs within the time available, the tender period being usually very limited. In this particular case, it was necessary to obtain details of the adjacent structures, and these were not all that readily available.

Not surprisingly, most consulting engineers require to be paid for carrying out the work and, at the tender stage, there is no guarantee that the job will be won and those costs recovered. It would seem to me that, if the piling contractor is to be responsible for design, then a more equitable arrangement might be for the engineer to indicate the order of the requirements—sizes of piles, reinforcement, etc.—and for all tenderers to price on that basis. The piling contractor who is successful in winning the contract could then check (and maybe even improve) on the design and assume responsibility. It is perhaps worthy of note that the revised draft of the ICE Model Procedures for piling states that the ultimate responsibility for design rests with the engineers.

The comment in the paper regarding piles out of position raises another interesting point. The large majority of piles were within tolerence, but some were out by a matter of a few millimetres. These could usually be accommodated within the pile group, and were probably the result of the rig moving off-centre. Others were significant amounts out, requiring corrective piles to be constructed, and were the result of incorrect setting out. I think the whole question of responsibility for setting out—and certainly the arrangements for checking by somebody else—should be reassessed. In the long run, it does not help anybody if piles are out of position, no matter who has to foot the bill. Competent checking by a main contractor or the engineer—and being paid for it—would avoid a lot of the errors, and the consequent problems. In today's increasingly 'fast track' construction projects, the pressures on the setting-out engineers are getting out of hand and need review. The arrangement whereby the main contractor sets out, and the piling contractor is required to check and agree it, seems to cover the matter.

The piling contract was carried out during the summer months, and was completed successfully and, I believe, to the satisfaction of all parties. I would like to express my company's appreciation of the cooperation and assistance given by the Post Office, architect, engineer, quantity surveyor, the main contractor, and, of course, our subcontractor, Messrs Lilley Construction.

Mr Myers: We do not disagree with the principle of Mr Holland's suggested procedure for testing a preliminary pile. CRP testing was one of the originally specified options, but was ruled out when the District Surveyor was unwilling to accept its results without those of a full maintained load test as well. In general, District Surveyors seem sceptical over the interpretation of CRP tests, and static tests remain the favourite.

Mr White: Mr Holland raises the issue of whether the engineer or the piling specialist should design the piles. While I have some sympathy for his views, the industry's traditional approach, which we follow, is that, as the piling contractor is required to guarantee that his piles will carry the required loads to a specific load factor, it is proper that he be asked to make the design and subsequently prove it by a predetermined testing requirement. We recognise that this approach is not altogether popular with piling contractors, but believe it firmly places responsibility for the works as constructed where it should properly lie.

Against this background, we provide the contractor with all the factual soils information that we have, together with any relevant information we may have on adjoining or underlying structures relating to the site (in this case, that for Atlantic Metals building, the canal wall, and the main sewer passing beneath the site). We also define the type of piling required.

However, for this project we had to add to our normal performance specification some further criteria to take account of the anticipated soil movements following the removal of the dumpling at the north end of the site. These criteria were selected so as to avoid loads being imposed locally on the TWA sewer and, more generally, to avoid the piles themselves being subjected to high order lateral and vertical loads resulting from the earth movements, which would in turn affect the substructure. We believe it is these further criteria that have given rise to Mr Holland's concern.

Mr G. R. Browne (Cyril Sweett & Partners): Mention has been made of the need to appoint a piling contractor in advance of the main contractor and a question raised as to why such an arrangement was necessary and, in view of the content of this earlier contract, why a main contractor was not appointed rather than the piling subcontractor.

The proposal to construct this building had been with the Post Office for many years, and during the early stages we had carried out a multitude of varying cost plans related to different types of building and on other sites. Eventually, when the Post Office confirmed their wish for the present building works to commence, with the date by which they needed completion, time was not sufficient to enable a full tender based on Bills of Quantities to be obtained. Various methods were debated, such as two-stage tendering, target cost, to enable a contractor to be appointed at an early stage.

Because of the complexity of this type of building, we felt that proper cost control would not be possible unless the tendering inquiries reflected all elements of the design, and quite clearly that would take some time. The option, therefore, was to bring about a preliminary contract to deal with the ground works. In addition to the actual piling, there were other civil works such as work to the canal wall, extensive earth removal, which had to be carried out in advance and in conjunction with the piling.

Thought was given to appointing a contractor for these preliminary works but we were concerned that establishing a contractor at such time would prejudice the obtaining of subsequent competitive tenders, and the recommendation was that the preliminary works should be carried out under the auspices of a piling subcontractor, the contract eventually being assigned to the successful main contract for the superstructure. It was felt that this would enable the employer to obtain the best tender and would also enable the consultants to pursue their design for the superstructure. In fact, things did proceed in a reasonably harmonious manner, and the employer will have his building completed by the desired date and within the original budget figure.

Mr E. L. Moss (Cormix Ltd.): With respect to the use of Cormix 'superplasticiser SP1' in the construction of the columns:

(1) No specific reason was found for the apparent poor results on the six or eight columns using the method. Possibly, the geometry of the column, which resulted in a long fall of the highly workable concrete, was a factor, together with the dense reinforcement present.

(2) No full-scale tests were possible because of pressure of time, though laboratory mixes were first made to determine the workability and compressive strengths.

(3) All the concretes delivered to the site were tested for slump cone workability before the addition of 'superplasticiser SP1' and for flow value (to DIN 1048) after the addition. These showed the concretes to be satisfactory and without any signs of segregation or bleeding.

(4) Despite the difficulties, no columns were rejected. Simple remedial work was sufficient to render them acceptable.

(5) It is possible that, with slight mix design adjustments and/or modifications to the filling technique, a satisfactory result would be achieved.

(6) The contractors were later able to devise a method to give a good column by using a 10mm-size aggregate (without Cormix SP1) instead of the 20 mm aggregate utilised for the Cormix SP1 concretes. A modified filling technique was also imposed. This consisted of filling the column to half height (3·5m) and immediately assembling the top lift followed by filling with concrete. This procedure ensured that no delays were suffered in constructing the columns.

(7) It is certain that this method and the 10 mm aggregate concrete could have successfully been combined with Cormix 'superplasticiser SP1' to give a good column.

The superplasticiser would have imparted very good flow characteristics at a reduced water content, thus yielding stronger, more durable concrete. We have a number of highly successful contracts using this type of concrete combined with Cormix SP1.

(8) A successful outcome was achieved using Cormix 'superplasticiser SP1', and a 20 mm granite aggregate in concrete for the strong room. Very congested reinforcement was present at this point.

Reference

D1. Burland, J. B., and Wroth, C. P.: 'Settlement of buildings and associated damage', *Conference on settlement of structures*, Cambridge, Pentech Press, 1976, pp 611-654.

Building Institution's Liaison Committee (BILC)

Annual report 1984-85

Meetings

During its second year of existence, the Building Institution's Liaison Committee—comprising IStructE, the Royal Institute of British Architects, the Chartered Institute of Building, the Chartered Institution of Building Services Engineers, and the Royal Institution of Chartered Surveyors—has held four meetings—on 1 October 1984, on 8 January, 22 April and 9 July 1985. The meetings have continued to be held at IStructE, which has again provided the Chairman and the secretariat of the committee for the year.

Topics considered

The major topics considered by BILC have been as follows:

Development of cooperation among the constituent institutions

BILC has, with some success, continued to encourage member institutions to develop closer working relationships at all levels, although cooperation among Institution branches varies in degree.

Education

Through the good offices of CIOB, 147 questionnaires were issued to heads of departments of building, civil engineering, architecture and related subjects in a total of 81 universities and colleges, with a 72 % return. The questionnaire was designed to establish common education already existing in those teaching institutions offering courses for degrees in construction-related subjects and to gauge the attitude of departmental heads to the concept of joint education. 78 % of responses expressed approval of the concept, and a small majority of respondents felt that the amount of joint education was likely to increase. A number of teaching institutions expressed the view that the individual requirements of the professional institutions tend to inhibit joint education. The results of the survey and their distribution are still under consideration by the committee.

Careers

BILC has also reviewed existing sources of advice on professional careers in the building industry and has noted that these are chiefly the individual institutions and that joint action in this area has not often proved successful. Nevertheless, the committee is hoping to issue a joint careers leaflet giving brief information to pre-O level schoolchildren. The committee has also noted the work of the Civil Engineering Careers Service.

Building Regulations

Developments in connection with the Building Regulations and the approval of persons and inspectors have been kept under review during the year.

Clients' guides

The committee has concluded its review of clients' guides to the various professions in the building industry and has decided that no action is required either in respect of existing guides or in the possible production of a new guide.

Maintenance manuals

BILC has also reviewed the present range of maintenance manuals for buildings, and has concluded that greater encouragement should be given to the use of the 'Building Centre maintenance manual and job diary', and a recommendation to this effect has been made to institutions.

Presidents' Committee for the Urban Environment

BILC has maintained liaison with PCUE on all the preceding matters. It was with considerable regret that the committee learnt of the death, on 20 July 1985, of Sir Hugh Wilson (Convenor of PCUE).