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Opening ceremony shaft for the Athens 2004 Olympic Games

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Key aspects of the analysis, design and construction of the retaining structure for a central circular shaft inside the Olympic Stadium of Athens are described. As part of the opening ceremony of the Athens 2004 Olympic Games, the shaft was used as an invisible underground entrance into the stadium, playing a key role in the ceremony. Given the significant depth and the large diameter of the shaft (both about 25 m), excavating and retaining under extreme time constraints and with presumed high water table constituted a challenge. It was accomplished with recourse to the observational method. The retaining structure comprised 1·2 m diameter contiguous piles capped with a ring beam. The initial design used a form of top-down construction of the final 0·75 m thick permanent concrete lining. Delays in starting the excavation necessitated that a depth of 17·5 m be reached before any permanent lining could be constructed. The paper presents the design of the retaining structure, the developed finite element analysis, and the theoretical predictions during the various excavation stages. A detailed monitoring programme was implemented and utilised in the spirit of the observational method. Comparisons every two days of measured against original (class A), as well as updated, analytical predictions led to the successful completion of the project.

1. INTRODUCTION

An invisible, yet substantial construction played a vital role in the spectacle of the opening (13 August) and closing (29 August) ceremonies of the 2004 Olympic Games: the cut-and-cover shaft, at the centre of the field of the central stadium. Approximately 25 m in both diameter and depth, the shaft was needed to drain (within only 6 min) the artificial lake that covered the play field (Fig. 1(a)), as well as allow 'flying' dancers and singers to enter the playing field in an apparently magical way (Fig. 1(a)). The gigantic statues could not have possibly made such a spectacular entrance without this shaft (Fig. 1(b)). Even the DNA hologram was projected into space from the shaft (Fig. 1(c)).

This paper describes the design, analysis, and monitored construction of the temporary retaining structure needed for this shaft.

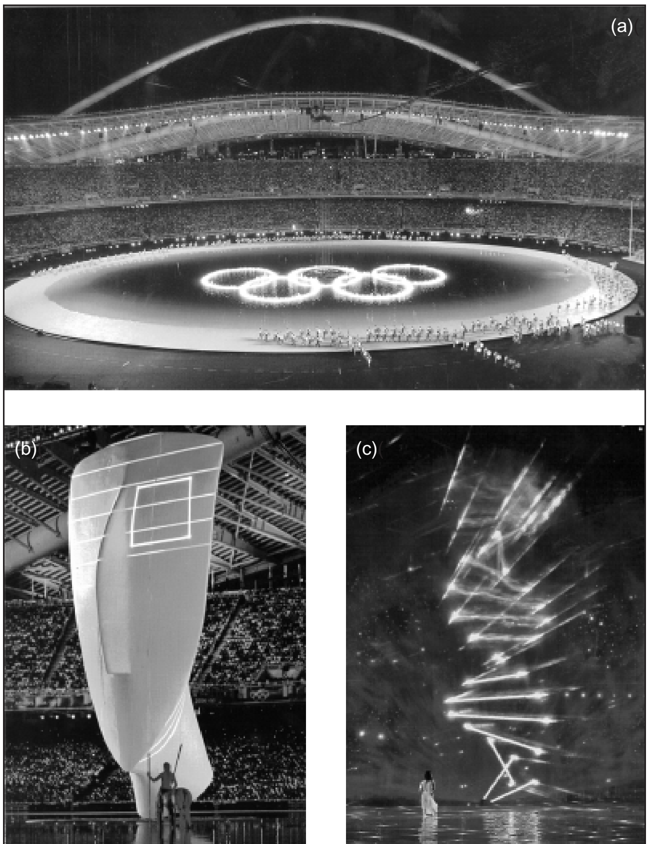

Fig. 1. Photos from the opening ceremony of the Athens Olympic Games on 13 August 2004. The shaft, at the centre of the Olympic Stadium, played a key role: (a) for draining 6000 m^3 of water from the 'artificial lake'; (b) as an entrance and exit for dancers, singers, and gigantic statues; (c) to project the DNA hologram

2. GEOTECHNICAL DATA

A conventional geotechnical exploration[1] comprised three boreholes along the perimeter of the shaft, G1, G2 and G3 (Fig. 2(a)), with standard penetration test (STP) measurements, and sampling (disturbed and undisturbed). The geology of the site includes mainly neogene formations, either of soil nature (stiff and hard clays) or rocky nature (conglomerates and hard marls). As depicted in Fig. 2(b), the soil profile consists of an upper layer of artificial compacted fill about 7 m thick, characterised as clayey sand to clayey gravel with a liquid limit LL $\approx$ 30, plasticity index PI $\approx$ 15, and natural water content $w_n \approx$ 18%. Consolidation (oedometer) testing provided the

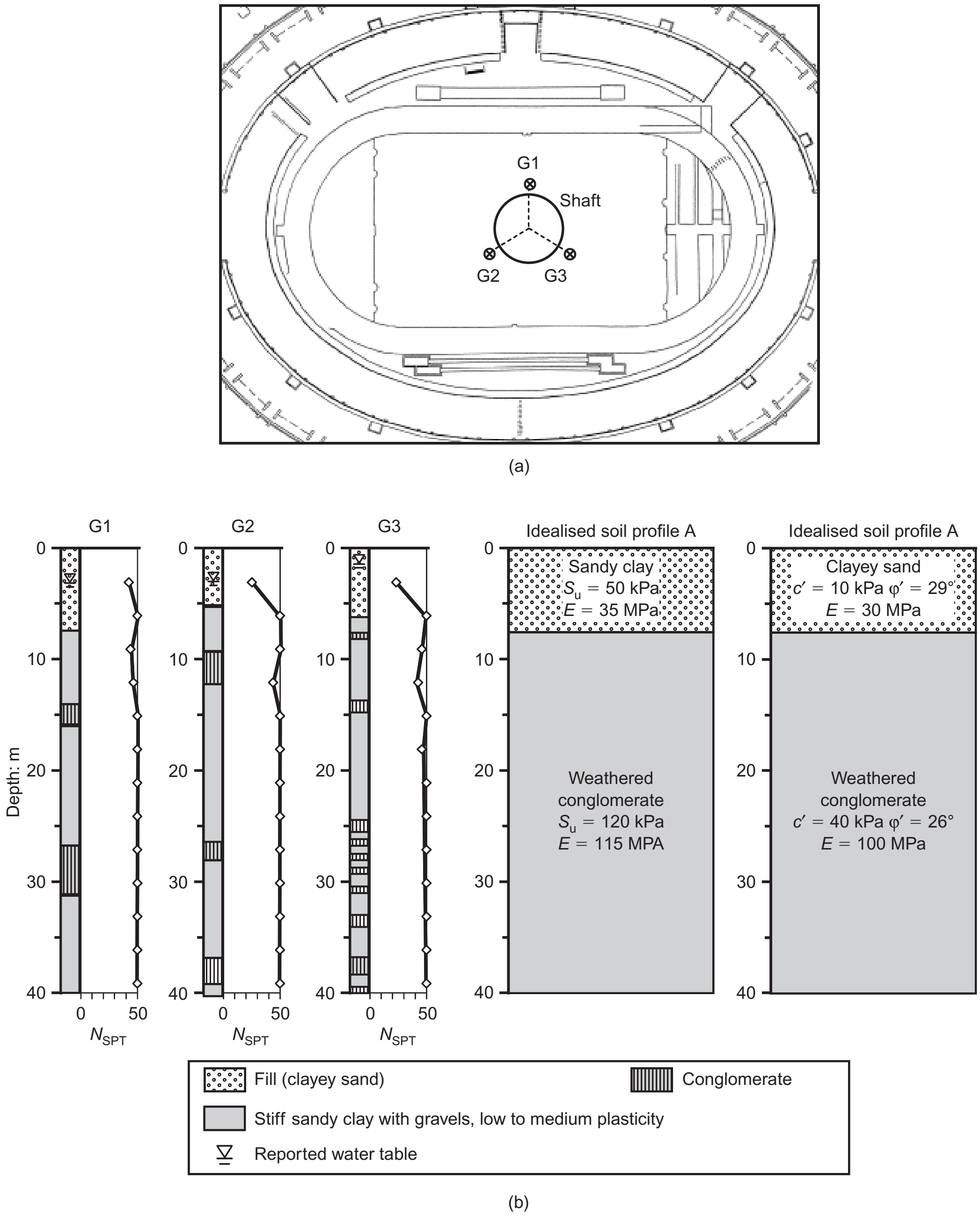

Fig. 2. Geotechnical exploration: (a) borehole locations, and (b) boring logs with NSPT blows, along with the two idealised soil profiles (based on the exploration—including oedometer, direct shear, and triaxial tests—by Istria, 2003)

virgin compression index—that is, the change in void ratio (e) per logarithmic cycle of vertical effective stress (σ'_v)[2,3]—at a depth of 5 m:

$$C_c = -\frac{de}{d(\log \sigma'_v)} \approx 0{\cdot}13 \quad (1)$$

and for the swelling–recompression index at the same depth, $C_r \approx 0{\cdot}01$.

The second layer is a deep layer of residual stiff sandy clay, changing to sandy and gravelly clay; it is most probably the product of weathering of the underlying conglomerate, as revealed by the presence of thin interlayers of conglomerate. The degree of weathering of the conglomerate and hence its cohesion were found to vary with depth and location along the perimeter of the shaft. The liquid limit was again of the order of 30, and the plasticity index of about 15; natural water content $w_n \approx 14\%$. From oedometer tests on samples from depths of 10–20 m the compression and swelling–recompression indexes are $C_c \approx 0{\cdot}07$ and $C_r \approx 0{\cdot}02$.

Standard penetration tests (SPT), conducted on all three boreholes, revealed $N_{SPT} \approx 25$–30 blows/30 cm for the first layer of artificial fill (clayey sand), and very consistently $N_{SPT} \geqslant 45$–50 blows/30 cm for the second layer of weathered conglomerate (sandy clay with gravels to low-plasticity clay with sand and gravels).

Surprisingly, the water table was reported to be very close to the surface, at only 2–3 m depth. But, as discussed below, the long-term interpretation of this measurement was questioned.

Strength characteristics were obtained from a limited number of conventional laboratory test results of direct shear and triaxial compression (both CU and UU type). Average properties are used from the two alternative idealised profiles shown in Fig. 2: for the artificial fill, cohesion $c' =$ 10 kPa, friction angle $\phi' =$ 29°, and Young's modulus of elasticity $E = 30$ MPa; for the sandy clay, $c' = 40$ kPa, $\phi' =$ 26°, and $E = 100$ MPa (soil profile B).

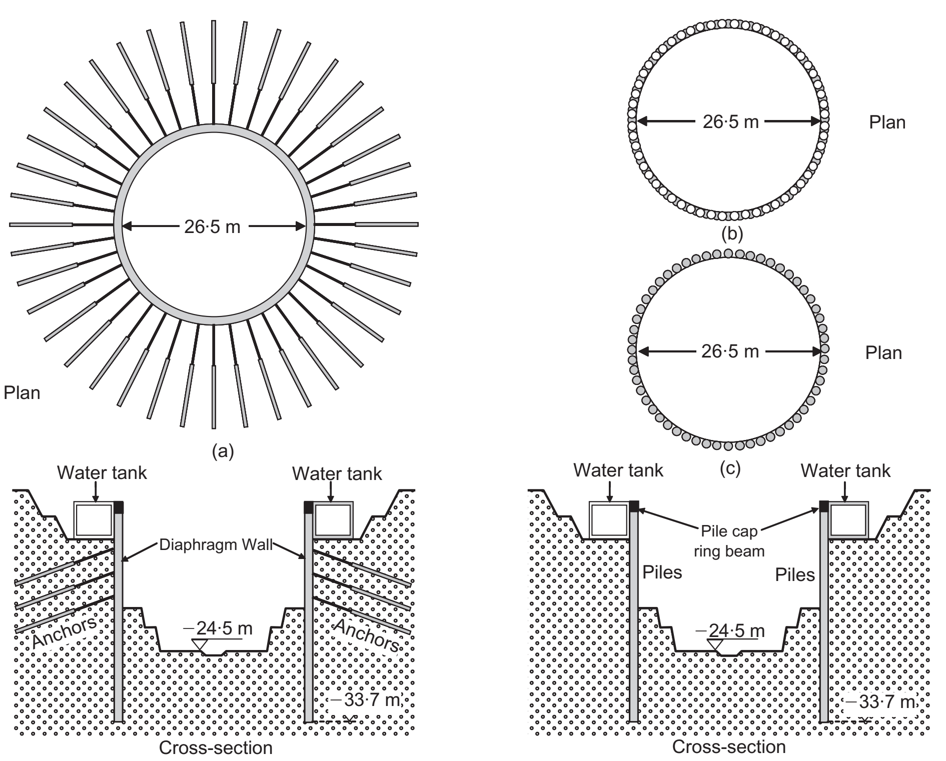

Fig. 3. The three temporary retaining structure alternatives: (a) tied-back diaphragm wall with three rows of anchors; (b) secant (interlocking) $D = 1{\cdot}2$ m piles spaced at 0·8 m; (c) contiguous $D = 1{\cdot}2$ m piles spaced at 1·5 m. All alternatives were considered in combination with a pile-cap ring beam at the top

Alternatively (soil profile A), we assumed undrained soil behaviour, with undrained shear strength $S_u = 50$ kPa and 120 kPa, for the first and second layer respectively. Young's modulus of elasticity was assigned values of $E =$ 35 MPa for the top layer and $E = 115$ MPa for the second layer. The selection of a constant Young's modulus with depth, $E = 115$ MPa, for the second layer is obviously a conservative approximation, as it is based on a few indirect measurements at depth up to 15 m. A logical alternative would be to take E as increasing linearly with depth.[4] A coefficient of lateral stress 'at rest', K_0, of 0·50 was assumed as the basic value in our analyses.

Finally, based on the results of in situ Maag tests,[1] the permeability k was evaluated to be of the order of 2×10^{-7} m/s and 5×10^{-8} m/s for the first (artificial fill) and second (weathered conglomerate) layers respectively.

Admittedly, soil testing (in situ and in the laboratory) was not 'state of the art'. All the above values of soil properties reflect simply our best judgement: hence a broad parametric investigation was unavoidably part of our analytical exploration into possible scenarios of response.

3. EXCAVATION AND RETAINING ALTERNATIVES

As illustrated in Fig. 3, three temporary retaining schemes were examined:

(*a*) a tied-back diaphragm wall
(*b*) a propped secant-pile wall (1·2 m diameter piles, spaced at 0·8 m)
(*c*) a propped contiguous wall consisting of 1·2 m diameter piles spaced at 1·5 m.

As a prop in the latter two cases, the piles were capped with a rigid ring beam, the hoop action of which would increase the radial rigidity of the retaining structure.

The first solution, of the tied-back diaphragm wall, was a logical choice given that

(*a*) the water table was detected close to the ground surface (at about 2 m depth)
(*b*) the depth of excavation was too large for an unpropped cantilevered wall
(*c*) the soil was sufficiently stiff to 'sustain' prestressed anchors.

There was only one problem with this solution: that of 'excessive' construction time (in view of the fact that the start of the games was only months away, and artistic trials using the shaft would have needed at least two months).

Interlocking (secant) piles, a second logical choice for securing watertightness[5] of the excavation, was also abandoned in an effort to save time, in the belief (based on local experience) among geotechnical designers, the constructor, and the checkers that the reported high water table was probably merely a seasonal perched groundwater surface above the low-permeability clayey substratum. Instead, the applied solution involved the following choices, some of which are seemingly non-conservative.

(*a*) Installation of *contiguous* bored piles, using a continuous flight auger (CFA) ring to drill successive but unconnected $D = 1{\cdot}2$ m piles (clear distance between them: 0·30 m).

Maximum verticality tolerance of 1·5% in depth was specified for pile installation. Their length: 9 m longer than the depth of excavation.

(*b*) As prestressed anchors and struts were beyond consideration, judged as time consuming choices, lateral support of the piled wall was provided (indirectly but efficiently) by a ring beam at the top of the piles. Its cross-section was 1·20 m × 1·20 m, and it played the role of a substantial stiff prop. Moreover, to further save precious time, a modified top-down construction sequence was to be carried out. When excavation reached 9·5 m depth, the top internal ring of the (final) permanent lining would be constructed (from the top of the ring beam down to a depth of 8·5 m). Then excavation would proceed, and construction of the second internal permanent lining ring would follow, and so on until completion.

(*c*) As leakage through soil between the (contiguous) piles could not be prevented, to minimise the potential groundwater hazard horizontal drainage pipes 10 cm in diameter would be installed between the piles as densely as the need arose with the progress of excavation. This would relieve the water pressures to some extent and allow for quick drainage (with sump pumps) from the bottom of the excavation.

(*d*) Finally, in view of the several non-conservative choices made, and the inherent uncertainties of the problem, the observational method, as advocated by Peck in 1969 in the Ninth Rankine Lecture,[6] was implemented. (For recent reviews of the method see refs 7 and 8.) A centrepiece of the method was the installation of a reasonably comprehensive monitoring system, described below. Key response variables were measured as construction proceeded, and were compared with the numerically computed values. Modifications to the design and construction were planned in advance, in case the most unfavourable of the foreseeable deviations of the measurements from the numerical predictions were to occur.

4. STAGES OF SHAFT CONSTRUCTION

The original schedule of top-down construction had to be slightly modified owing to some unforeseen delays. To save time, excavation would now proceed to 17·5 m depth before any internal lining was installed. Referring to Fig. 4, the most important stages in the construction of the shaft were as follows.

(*a*) Excavation to 1·5 m depth, followed by installation of the piles to 33·7 m depth, and casting of the pile-cap ring beam.

(*b*) External excavation to 3 m depth and internal to 9 m depth. External excavation was required for placing the water tanks (outside the shaft). Once the depth of 9 m was reached it was originally planned to construct the 9 m high internal (permanent) lining ring, in a kind of top-down construction. Measurements and analysis suggested that time could be saved without jeopardizing safety, if this casting of internal lining were postponed for greater depth.

(*c*) External excavation finalised at 7·2 m depth; internal at 11·5 m depth.

(*d*) Installation of a first row of drainage pipes at 9 m depth; internal excavation to 17·5 m depth.

(*e*) Construction of the reinforced concrete permanent internal lining from bottom 17·5 m depth to top. (Construction in two stages as originally planned, from 9 m depth to top, and from 17·5 m to 9 m depth, would have caused a few weeks' delay.)

(*f*) Excavation to the final depth of 24·5 m.

(*g*) Construction of the lower part of the internal permanent lining along with the bottom slab.

(*h*) Construction of the retractable roof.

The only constraint in the design of the excavation and the retaining system was to prevent yielding of the piles. This was equivalent to a maximum pile deflection not greater than about 3 cm.

5. ANALYSIS OF STAGED CONSTRUCTION

Despite the axial symmetry of the excavation, strictly speaking the problem was not axisymmetric in mechanics terms, in view of the fact that the piles were not in contact and hence they could not transmit hoop forces (i.e. tangential forces along the periphery). On the other hand, if (hypothetically) the radius of the shaft were very large (compared with depth) the problem could be reduced to a plane-strain piled-wall problem. Obviously, reality lies somewhere between the plane-strain and the axisymmetric idealisations. Also, the pile-cap ring beam at the top is essentially under axisymmetric conditions, carrying primarily hoop forces (i.e. axial compressive load).

So, from the analytical point of view, although things would have been really straightforward with a diaphragm wall, and even with a secant pile alternative (in first approximation), as axisymmetric conditions would be reality, the chosen supporting system necessitated a different methodology.

The analysis utilised the finite-element code PLAXIS.[9] To account for the aforementioned 'semi'-axisymmetric nature of the problem, we applied a hybrid methodology. The basic analysis was conducted in plane strain, but after making two simple intuitive corrections: (*a*) for arching (through several axisymmetric finite element analyses as described below); and (*b*) for the contribution of the hoop stiffness of the pile-cap ring beam. In the basic analysis the soil was modelled through three-noded plane-strain elements, and the piles as beams connected to the soil through a Coulomb sliding interface. The Mohr–Coulomb constitutive law was assumed for the soil, [4] and the piles were considered as elastic.

The aforementioned additional support afforded by the axisymmetry of the excavation was estimated by conducting

(*a*) an axisymmetric analysis of the soil without the piles

(*b*) an axisymmetric analysis of the pile-cap ring beam

(*c*) an axisymmetric analysis of the internal permanent ring lining.

For (*a*), full excavation to the maximum depth (24·5 m) was considered. Then a uniform horizontal radial load of 1 kPa was applied on the vertical excavation face. With the computed horizontal displacement of the excavation face, we derived an equivalent stiffness of the support, which included the contribution of soil arching (due to axisymmetry). The effect of the additional support provided by the pile-cap ring beam and

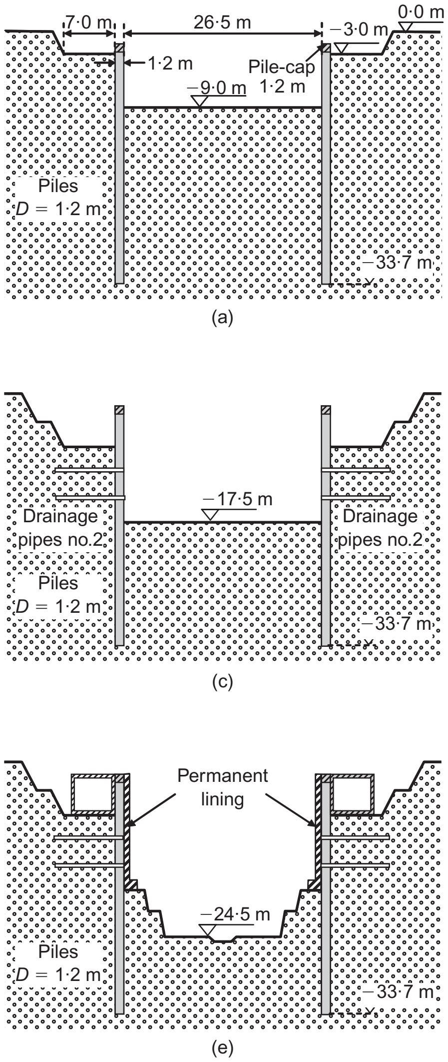

Fig. 4. Construction sequence. (a) After installing the piles, the shaft is excavated externally to −3·0 m and internally to −11·6 m. (b) The external excavation is finalised at −7·2 m. (c) The internal excavation continues to −17·5 m. (d) Part of the permanent internal lining, from −17·5 m to −1·8 m, and the external water tanks are constructed. (e) The internal excavation reaches the maximum depth of −24·5 m. (f) The lining is finalised, and the roof is cast and backfilled

the final lining (when installed) was estimated with a similar methodology.

Then these two additional support mechanisms (soil arching and pile-cap ring beam hoop stiffness) were incorporated in the plane-strain model as equivalent struts. Their stiffness was a suitable superposition of the stiffnesses computed using the two aforementioned axisymmetric models. The equivalent struts modelling soil arching were distributed as Winkler springs along the whole length of the piles, and those simulating the pile-cap ring beam were placed concentrated at the top of the piles. When excavation reached 17·5 m depth, additional struts (Winkler springs) were (computationally) added to the piles to simulate the effect of internal lining (which was cast at this stage).

Admittedly this hybrid analysis methodology would only model reality crudely. It was a choice prompted by the time requirements of a truly 3-D finite element analysis and the significant number of parameter studies needed to cover the uncertainties in soil parameters and water regime. As will be shown later, with this hybrid methodology the predicted performance of the retaining agreed well with the measurements. An alternative method of analysis would be to model the soil with axisymmetric elements and to consider plane-strain conditions for the piled wall. However, this option is not supported by the FE code used.[9]

The analysis was conducted in steps, replicating the construction sequence. In each step the water was assumed to drain partially, as the piles were at a distance from one another. Of course, if the water table were indeed very high and the top layer were highly impermeable, such a drainage would take too long. But, given the significant sand fraction of the top soil layer, in combination with the measured permeability (which was not dramatically high), one would expect some drainage to occur during excavation. The two rows of drainage pipes that were finally installed ensured that such drainage would at least show up.

Figure 5 portrays various snapshots of the deformed FE mesh. Note that the wall displacements increase as excavation

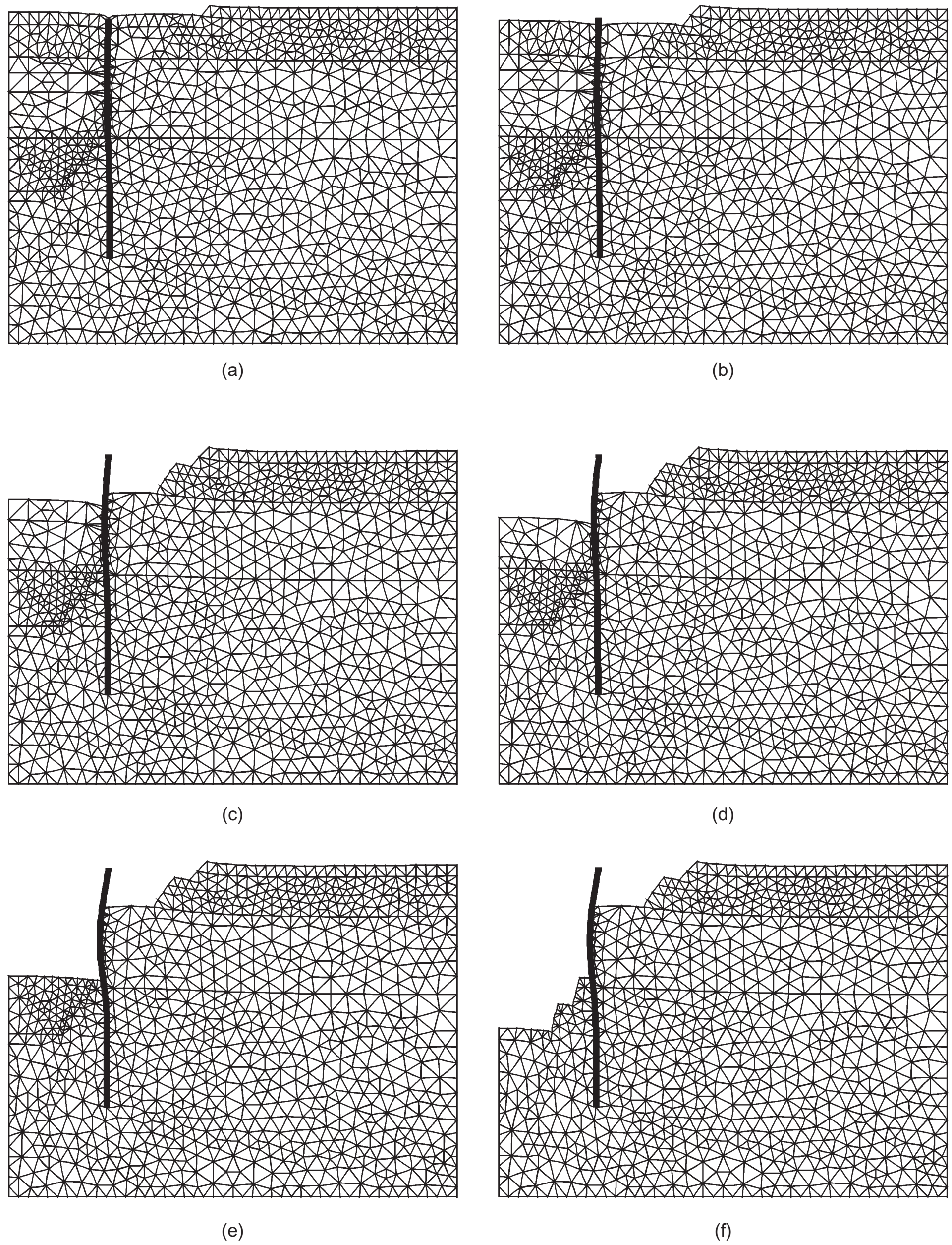

Fig. 5. Results of FE analysis. Snapshots of deformed mesh: (a) excavation to 1·5 m depth and installation of piles; (b) excavation to 3·0 m depth; (c) internal excavation to −9 m and external to 7·2 m depth; (d) internal excavation to 11·5 m depth; (e) internal excavation to 17·5 m depth; (f) installation of permanent internal lining to 17·5 m depth and internal excavation to 24·5 m depth

progresses. They reach a maximum (inward) displacement of $\Delta \approx 12$ mm at about 13 m depth (Fig. 5(f)). Observe the restraining role of the pile-cap ring beam. Also note that the bending moment M in the piles never exceeds 800 kN m per pile, easily undertaken by the 1·2 m diameter reinforced-concrete pile, and that the hoop force in the pile-cap ring beam reaches 8 MN, also easily accommodated with the 1·2 m square concrete cross-section.[2]

Evidently, compared with the secant-pile alternative, the adopted contiguous pile solution had some advantages. (*a*) By allowing the water to at least drain partially from between the piles, it reduced the water pressures on the piled wall (by an estimated 30% of their full hydrostatic value). (*b*) As there is no continuity between the piles, they retain the earth pressures as vertical beams deforming solely in bending. This allows for some deformation, and therefore an ensuing stress relief. On the other hand, if hoop forces could develop (as with diaphragm walls and secant piles), the wall would act like a cylindrical shell system—an extremely stiff support. Then K_0-type earth pressures would develop. In our case, the flexibility of the system allows for active conditions to develop below a certain depth. The only price to pay is the increase of the inward pile displacement. Fortunately, the computed 12 mm of displacement is insignificant (even if it is much larger than the 2 mm displacement of the secant pile alternative).

6. THE MONITORING SYSTEM

The instrumentation (as required by the observational method) is depicted in Fig. 6. The monitored response variables were

(*a*) displacements of the piled wall

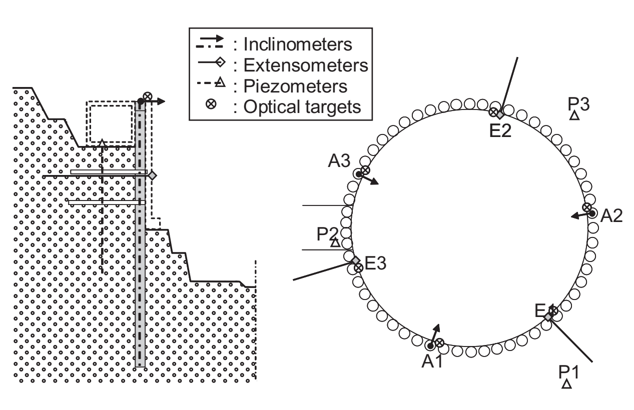

Fig. 6. The monitoring system, comprising: three inclinometers, inside the piles; three extensometers at 9 m depth; three piezometers along the perimeter of the external excavation; and six optical targets.

(*b*) soil displacements in the neighbourhood of the shaft
(*c*) changes in the level (depth) of the water table.

The displacements of the wall are the most direct measure of the retaining system's behaviour. In addition, as the pile bending moments are directly related to deflection, the level of distress can be assessed directly. As shown in Fig. 6, three inclinometers were installed to measure pile deflection. A fixed point was assigned to convert the recorded deflections relative to displacements. Hence, atop of each of the three piles equipped with an inclinometer, optical targets were also installed.[10,11]

By monitoring the displacement and deformation of the soil in the neighbourhood of the shaft, one could perhaps detect the beginning of a possible failure mechanism at an early stage. To this end, we installed three 15 m long extensometers at three different pile locations (Fig. 6), to measure relative displacements between the pile and a 'fixed' point far away from it. They were installed at $-10{\cdot}5$ m, as soon as the excavation had reached -12 m. It was expected that, if a failure plane started to form, the extensometer would show a sudden jump at some location behind the wall. However, the evolution of the potential failure mechanism would probably be rather ductile, given the reduced radial rigidity of the contiguous piled wall. This is an advantage over hoop retaining structures, the failure of which would tend to be brittle in nature.

Finally, monitoring the changes of water table depth was crucial for ensuring the stability of the retaining system. Our working hypothesis was that the water would drain substantially from the top layer and appreciably from the lower. This by itself was a non-conservative assumption, but (as already mentioned) it was based partly on the belief that the detected high water table was in fact of a temporary 'hanging' nature. If this belief had proved too optimistic, the developing hydrostatic pressure would have tended to make things significantly worse. To get an early warning of the imminence of such an unfavourable outcome, three piezometers were installed at different locations along the perimeter of the external excavation (Fig. 6).

7. OBSERVATIONAL METHOD AND CONTINGENCY MEASURES

A key ingredient of the observational method is the pre-planned ability to alter the design during construction.[6–8] Pre-planning means that, in addition to the main design, which is based on the most probable conditions and which the construction initially implements, contingency measures or modification of design have been worked out for the foreseeable critically significant deviations from the most probable conditions.

For the Olympic shaft the following unfavourable (worst-case) scenarios were envisaged.

(*a*) significantly worse properties of the second sandy clay layer: $c' = 25$ kPa, $\phi' = 25°$, and $E = 60$ MPa
(*b*) truly impermeable soil layers and a high water table, as reported in the geotechnical site investigation report
(*c*) negligible soil arching: truly plane-strain conditions.

Plausible combinations of unfavourable scenarios were also analysed to determine whether the retaining structure would be safe in all cases.

If the first scenario were the case—that is, if in reality the properties of the main soil layer were substantially worse than originally assumed—then the measurements would have shown almost double the deformation that actually occurred: inward pile displacement in the order of 25 mm. The presence and drainage of water would have been detected from the piezometers and the observed drainage of the horizontal pipes. Without only soil arching (not a likely outcome), the displacements would have reached 40 mm. Even in this case, piles and pile cap would have been capable of sustaining the imposed bending moments and hoop forces respectively. Even if the last scenario occurred in combination with one of the other two, the structure would marginally not have collapsed. However, if all three scenarios were combined, the originally conceived retaining system would have most likely failed. In such a case some early warning would come from the various measurements, and contingency measures deliberately designed in advance would have been applied, as explained below.

The consequences of the simultaneous unfavourable occurrence of the first two scenarios are illustrated in Fig. 7. Pile displacements Δ and bending moments M (per pile) at two characteristic excavation stages (11·5 m and 24·5 m depth) computed as in our original design (with the most probable conditions), are compared with the results for this unfavourable scenario. Observe that in the latter case, when excavation reaches 11·5 m depth, the wall displacement already exceeds 20 mm, in contrast with the actual observation, and that at the final excavation stage $\Delta \approx 40$ mm, which exceeds the serviceability limit state (~ 20 mm), and $M \approx 2$ MN m, which is equal to the design capacity (ultimate limit state) of the heavily reinforced 1·2 m diameter piles.

The trigger for applying contingency measures was set to be an observation of $\Delta \geqslant 20$ mm. Regarding the water, two extreme possibilities were identified.

(*a*) The piezometers showed that the water table had dropped

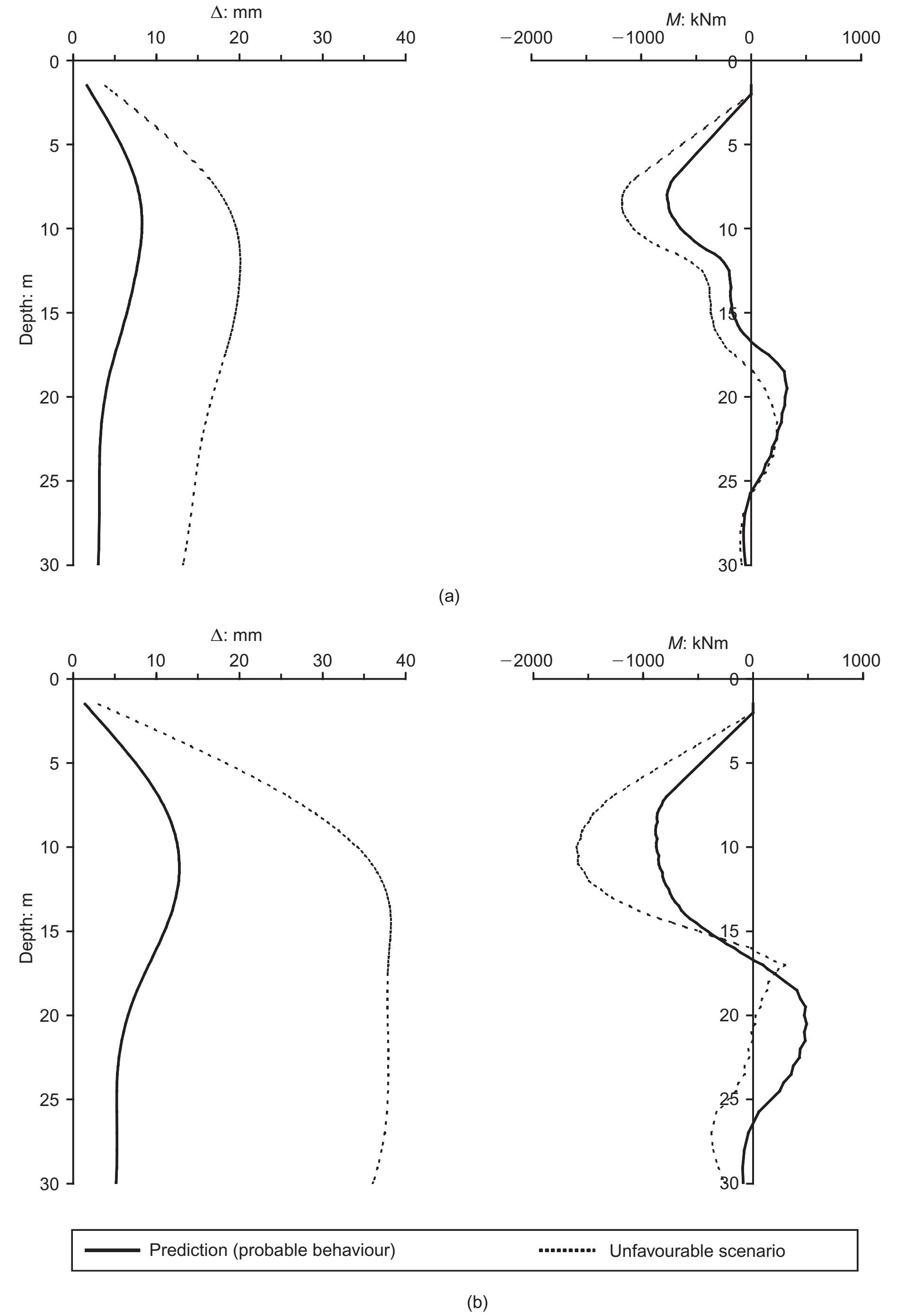

Fig. 7. Horizontal piled wall displacement Δ and bending moment *M* (per pile). Prediction under the most probable conditions compared with a worst-case pessimistically foreseeable scenario: (a) excavation down to 11·5 m depth; (b) complete excavation down to 24·5 m depth

(or it had never actually been as high as found in the geotechnical exploration).

(*b*) The water table remained close to the surface.

In the latter case, additional drainage pipes would have been installed. If proved successful in draining, the excavation would continue as planned. If not, a shift to a more genuine top-down construction scheme was devised: the internal permanent lining of the upper half of the shaft would be constructed as soon as a depth of about 9 m had been reached, to act as a huge ring (shell) before the remaining excavation could proceed.

8. CONSTRUCTION OF THE SHAFT AND COMPARISONS OF PREDICTIONS AND MEASUREMENTS

Figures 4, 8 and 9 highlight the various stages of construction, which began in September 2003. Without repeating what is already explained in these figures, we note that by December 2003, when the internal excavation had reached −9 m and the external had already been completed (−7·2 m) (Fig. 9), we were in a position to start evaluating some of the design assumptions. Although the deflection of the piles was still insignificant (in accord with analytical predictions based on 'most probable' conditions), the water table had already been

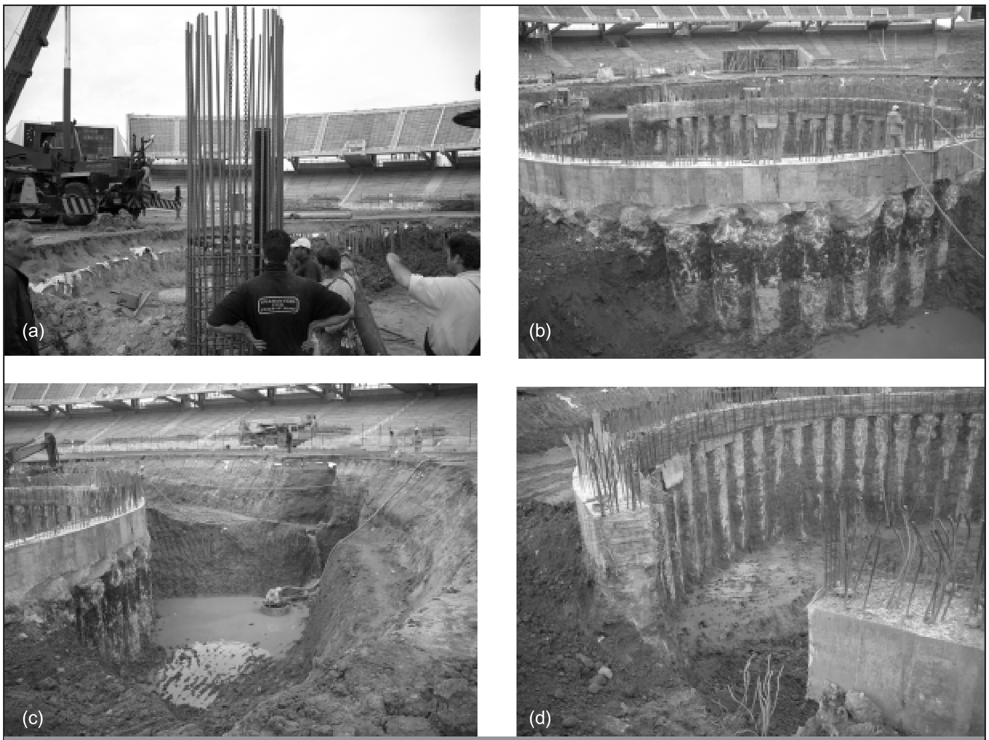

Fig. 8. (a) At the end of September 2003 the soil surface was excavated to 1·5 m depth and installation of the piles begun. 1 December 2003: (b) external excavation finalised at 7·2 m depth (note 'missing' piece of the ring to make a temporary entrance and hence speed up the initial stage of excavation); (c) as hoped, the water table was not at 2 m depth but at 7 m depth on average, essentially coinciding with the bottom of the external excavation; (d) pile-cap ring still 'open' at the location of the ramp

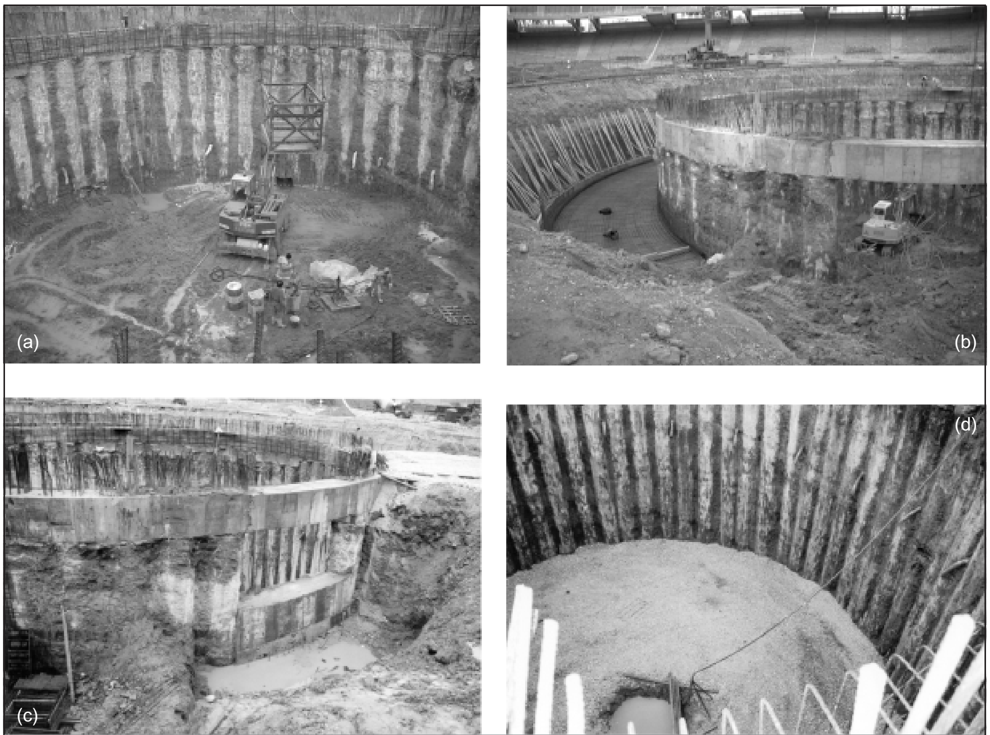

Fig. 9. (a) 8 December 2003: the internal excavation had reached −11·6 m internally and the first row of drainage pipes had been installed at −9 m. (b) 12 December 2003: casting of the water tank along the external perimeter of the shaft has begun. (c) 20 December 2003: the ring beam is now in place. Closing of the gap in the pile cap made excavation to greater depth possible without any further lateral support. (d) 2 January 2004: the excavation reaches −17·5 m inside the shaft. Drainage from the pipes is minimal

lowered substantially: on 1 December it was at about −7 m on average (−6·5 m on the north side and −7·3 m on the south). The piezometers were consistent with optical observations. One thing that was appreciably different from the design was the pile-cap ring beam. As seen in Fig. 8, the contractor was given permission to leave a 4 m wide opening on the pile-cap ring beam, as this would enable him to excavate at a faster rate. However, an open beam does not transmit hoop forces: therefore it does not act as a ring and cannot play the role of a prop. Acceptance of this unsafe modification during the early stages of excavation could be allowed only by a slight change in the schedule of excavations so that the difference between external and internal excavation remained merely 2 m.

Up to that point, construction had proceeded as anticipated. After the first row of drainage pipes had been installed, it became evident that the stiff sandy clay was either not very wet, or was not draining fast. One of the contributing factors to such behaviour was the fact that the water table was not as high as originally found, but was actually just above the interface between the upper artificial fill and the stiff clay. Nevertheless, to be on the safe side, drainage pipes continued to be installed.

On 20 December the gap of the pile cap closed, allowing the difficult part of the excavation to begin (Fig. 9). The measurements were within the limits of the analytical prediction, and the extensometers did not exhibit any sudden 'jump' in the deformation profile. Focusing closer on the deflection profiles, as measured with the inclinometers (and in full accord with the optical targets) (Fig. 10(a)), note that at this stage the three inclinometers, A1, A2, and A3, exhibited significant differences: A1 and A3 showed their maximum deflection at about −10 m (in accordance with analysis), but the deflection of A3 was different. Being closer to the earlier aforementioned gap of the pile-cap ring beam (see Fig. 8), the deformation shape of A3 was reminiscent of an unrestrained pile. By contrast, A1 and A2 behaved almost as 'fixed' at the top. In all cases, the measured inward displacements of the piles were slightly lower than predicted.

A large (but of lesser significance) difference between measurements and prediction refers to the displacement at the pile tip (bottom). Although the analytical results showed some displacement, in reality the piles behaved as completely 'fixed'. This difference could be attributed partly to the conservative modulus of elasticity that we had adopted for the sandy clay, keeping it constant even at large depths. A linearly increasing modulus at these large depths would have largely suppressed the computed displacement.

On 2 January 2004 the excavation reached 17·5 m depth (Fig. 9(d)). A drainage pump had been installed at the bottom of the shaft. However, given that only a small quantity of water was draining, and the piezometers were not detecting any measurable water pressures, we decided to forgo the installation of the second row of horizontal drainage pipes. The bottom-up construction of the permanent internal lining began. No significant differences could be observed in the readings of piezometers or extensometers. The inclinometers showed an increase of pile deflection. Notice that the deflection at the top of A3 (close to the former pile-cap gap) showed the 'sign' of restrained (pile-cap continuity had by now been restored). The deflection of A3 was practically identical to the analytical prediction down to 15 m depth. At further depth the piles behaved as 'fixed', in contrast to the analysis, which predicted 3 mm of displacement; a plausible explanation has already been given in the previous paragraph. For A1, the maximum displacement occurred a little deeper (13 m depth) than predicted (10 m depth), but the maximum values were practically the same.

On 3 March 2004 the excavation successfully reached the maximum depth of 24·5 m. The final inward displacements are compared with the corresponding predictions in Fig. 10. The maximum displacement reached 14 mm in A2, and only slightly exceeded 10 mm in A1 and A3. The comparison with our (Class A) analytical prediction (12 mm) was quite satisfactory. On 20 April the excavation was completed to −24·5 m, casting of the roof was about to begin, and the internal permanent lining had been cast down to −17·5 m—nearly one month ahead of schedule.

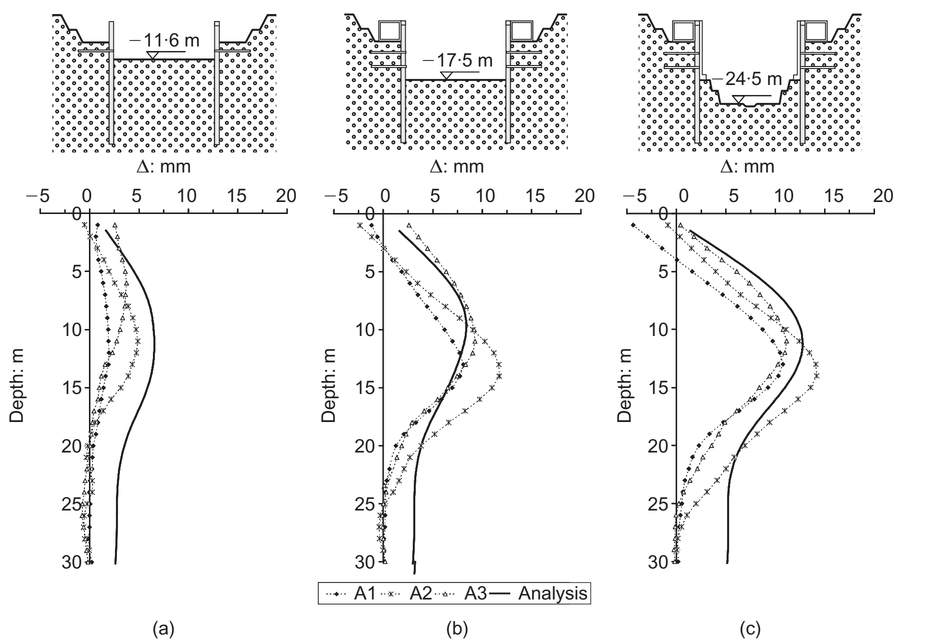

Fig. 10. Analytical prediction of horizontal pile displacements Δ against inclinometer measurements; (a) internal excavation at 11·6 m depth, 17 December 2003; (b) at 17·5 m depth, 2 January 2004; (c) at 24·5 m depth, 4 March 2004

9. CONCLUSIONS

The shaft for the opening ceremony was designed and successfully constructed under stringent time constraints, by utilising an observational method approach.[6] The choice of the retaining system was the least conservative for the prevailing soil and water conditions.[2] The analysis of the system of contiguous piles capped with a ring beam was not a rigorous truly three-dimensional analysis, but a hybrid methodology devised to simulate the effects of axisymmetry of the problem and of discontinuity of the non-secant piled wall. A detailed monitoring programme guided the staged construction. The measurements, compared with Class A predictions with the most probable soil conditions, verified the key design assumptions and helped introduce some modifications in the final design. The most unfavourable scenarios had been analysed in advance, and appropriate contingency measures had been studied. The shaft was completed on time, and contributed to the magic of the opening ceremony of the Olympic Games on 13 August 2004.

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