Case Study of Barrette Retaining Wall, Kent, UK

Etude de cas d'un mur de soutenement en barrettes, Kent, GB

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ABSTRACT This paper presents a case history of a retaining wall designed by Arup and constructed in Kent, UK, between 1996 and 1998, as part of a highway cutting. The design methodology which was applied for this novel wall type is introduced, and measured horizontal displacements are presented and discussed. The design requirements specified that the wall had to be capable of supporting up to 8.8 m of retained height and was not to be propped or anchored. To attain sufficient bending stiffness, Arup introduced a series of discrete barrettes perpendicular to the excavation. Arches of sprayed concrete retain the soil between the barrettes above dredge level. The barrettes were designed applying the general design principles for embedded retaining walls. Pore water pressures on the wall were reduced and controlled by installing a back of wall drainage system. Inclinometers in one of the instrumented wall sections were recently re-read. These data are shown and compared to the original predictions. Overall the wall showed only minor movements and satisfies the stated design requirements.

RÉSUMÉ Cet article présente l'étude chronologique d'un mur de soutènement conçu par Arup et mis en place a Kent, Royaume-Uni, entre 1996 et 1998, sur la portion en tranchée d'une voie rapide. Une fois la méthode de conception utilisée pour ce type de mur innovant présentée, les déplacements horizontaux enregistrés sont exposes et discutes. Le cahier des charges spécifiait le nécessité pour le mur de supporter 8.8m de soutènement sans butons ni ancrage. Afin d'obtenir suffisamment de rigidité en flexion, Arup a proposé une série de barrettes perpendiculaires a la tranchée. Au-dessus du fond de l'excavation, le terrain est retenu par des arches en béton projeté s'appuyant sur les barrettes. Le calcul des barrettes est effectué suivant les principes généraux des parois moulées. La pression hydrostatique sur le mur est réduite et contrôlée par la mise en place d'un système de drainage. Des mesures ont été prises récemment sur les inclinomètres de l'une des sections de murs instrumentées. Les relevés sont exploses et compares aux prévisions d'origine. Elle révèlent que les mouvements ont été minimes et que le mur répond aux spécifications du cahier des charges.

1 INTRODUCTION

Retaining structures for deep excavations generally need horizontal supports to retain the soil pressures whilst at the same time restricting the ground movements behind the wall. For this purpose, propping is only viable if the opposite side of the excavation is reasonably close, or the excavation is of confined width so the walls can be propped using perpendicular walls. Inclined props which transmit the forces to single foundations below the final excavation level can form major obstructions for the excavation works and cause difficulties if water-tightness of the base slab is required. Permanent ground anchors can be an option, however, these are often difficult to use in practice due to adjacent underground structures, obstructions and potential legal land ownership issues. In addition to this, accessibility of anchor heads for monitoring and maintenance purposes is often challenging in the long-term.

In relatively shallow excavations, in which the retaining wall can support the soil acting as a cantilever in the short-term, using the base slab as a permanent prop is a widely used arrangement to provide design security for long-term conditions.

This paper introduces a novel retaining wall solution using discrete structural elements, similar to a soldier pile wall. Rather than piles, discrete barrettes perpendicular to the excavation are used as vertical elements to provide the structural resistance. Thanks to their orientation, the stiffness of the barrettes perpendicular to the excavation is much higher than that of soldier piles or diaphragm walls and the barrettes can withstand much higher lateral forces.

The paper describes the original analysis and design of the barrette wall M1T2 Thanet Way, and presents measured wall performance data. Deformations of the wall and the soil behind the wall at the end of construction are shown for the cross section with maximum retained height. Aside from the case study presented in this paper, only one other wall of this type, which is described in Deschamps (2008), is known to the authors.

Five inclinometers installed in the wall section described have been read in June 2014. Despite interpretation of the new readings being problematic due to a lack of original installation records, the new data suggest that the deformations behind the wall, 16 years after construction, are minor.

2 THANET WAY RETAINING WALL

2.1 *Selection of wall type*

The wall presented in this paper was constructed between 1996 and 1998, on behalf of Kent County Council (KCC), UK, as part of the upgrade of A299 Thanet Way. A grade separated highway interchange required design and construction of retaining walls of up to nearly 9m in height.

Stem and heel gravity walls as well as ground anchored walls were discounted for a number of factors, in particular land take. The wide span to the opposite wall and headroom requirements along the steadily increasing wall height meant that the wall could not be propped above road level. As the design had to allow for excavation adjacent to the wall for laying and maintenance of services, propping below road level was problematic. Due to these constraints, a cantilever wall solution was selected. Arup designed an embedded cantilever wall; For wall sections higher than 6m, the wall consists of 0.6m wide, 6m long barrettes installed perpendicular to the excavation, spaced at 4m. Above excavation level, the soil between the barrettes is retained by 250mm deep sprayed concrete arches.

Maximum allowable wall deflections of 20mm horizontal movement at verge level and 1:200 tilt above verge level, following construction of a non-structural facing wall, were specified by KCC in order to minimize the risk of future damage to the block work. In order to ensure that these limits were not exceeded, a permanent drainage system was provided, consisting of fin drains behind the sprayed concrete arches and vertical drains located 6m behind the wall facing, at the end of the barrettes. The water level behind the shotcrete is controlled with intersecting horizontal drains which discharge through the verge beam into the highway drainage system. The general arrangement and the back of wall drainage system is shown in Figure 1.

The section considered in this paper, including the geology and basic design parameters is shown in Figure 2. The wall in this section is fully embedded in London Clay (LC).

2.2 *Failure mechanism*

Similar to a soldier pile wall, barrette walls are discontinuous and their overall stability depends on the mobilization of shear along the embedded wall length below excavation level.

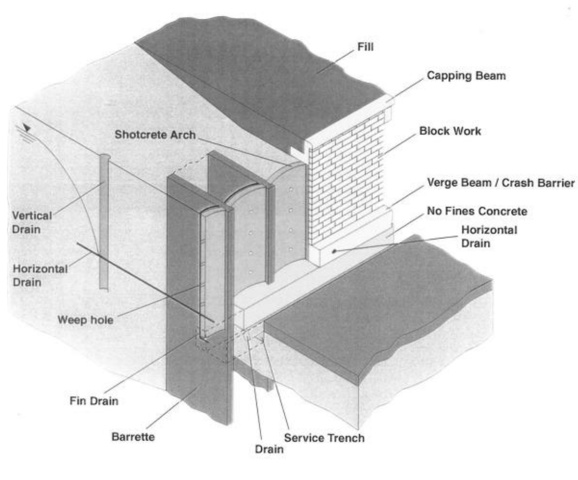

Figure 1. Isometric view of the wall and the drainage system.

Two sets of analyses were carried out to study the interaction between the barrettes and the soil. The forces on the sprayed concrete arch were derived at excavation level, in plan, using the 2D FE software SAFE. The calculations took into account stress relief in the ground as the barrettes are constructed, the unsupported face during excavation and the shotcrete arch construction.

Analyses below excavation level examined the forces acting, in plan, on a soil block, contained between adjacent barrettes at depth z. For the given wall geometry and soil parameters, the combined active earth pressure and shear resistance available between barrettes was calculated to be greater than the available effective passive resistance in front of the wall. This led to the conclusion that the barrettes would act as an integral unit with the soil between the barrettes and the overall resistance would not be limited by local failures around each individual barrette.

As a consequence of these calculations, it was determined that it would be appropriate to model the wall as a continuous wall with an equivalent wall thickness of 6m, equal to the length of the barrettes. The composite wall stiffness for subsequent analyses was assumed as the stiffness of a single barrette divided by the spacing between barrettes.

2.3 Applied design procedures: Stability

For barrette walls, similar to gravity walls, the stiffness of the structural elements is such that bending is not likely. On the other hand, the embedment length of the barrettes as well as the construction procedures are more akin to contiguous embedded retaining walls. During the original design of this wall, a number of analyses were undertaken to compare and determine which of the two common design procedures was applicable.

As a result of these analyses, the wall was designed as an embedded cantilever wall with allowance for vertical shear at the wall faces. The wall toe level was derived as the level at which the resultant moment about the center of the base is zero, taking into account shear forces at the face and at the back of the wall, with an interface friction coefficient of $\delta=\varphi$. To cover uncertainties associated with this uncommon approach, for this calculation the effective width of the wall was reduced to 2/3 or 4m.

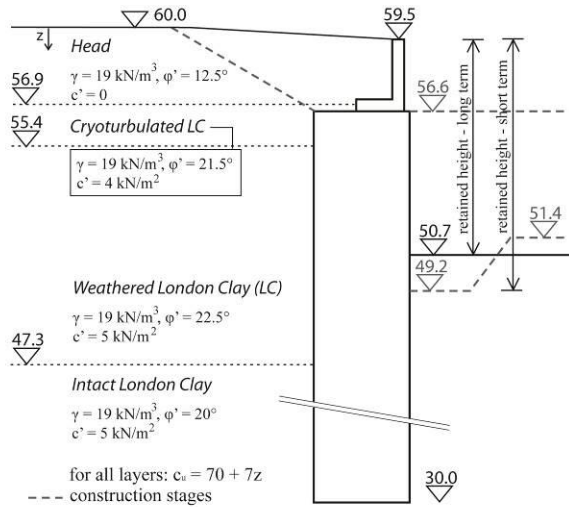

Figure 2. Wall section of maximum retained height: cross section and geology

Because the allowance of side shear was not covered by the commonly used design guide CIRIA 104, Arup adopted the published EC7 recommendations at that time and applied 0.5m overdig as well as safety factors of 1.25 on $\tan\varphi'$ and 1.6 on c'.

2.4 Applied design procedures: Displacement

Calculations of wall and ground movements were made using the 2D finite element program SAFE.

The original analysis was conducted using the BRICK soil model, developed by Simpson (1992). In this analysis, short-term and long-term deformations were analyzed applying undrained and drained conditions respectively, since at that time, the FE programme SAFE did not allow coupled consolidation during construction to be modelled.

3 WALL PERFORMANCE

3.1 Measured data

Given the novel wall construction and the associated need to verify the applied design procedures and calculated wall movements, a comprehensive monitoring system was installed along the wall. Deformations were measured using inclinometers and extensometers at various distances behind the wall as well as in the barrettes. Water levels were monitored both behind and in front of the wall using vibrating wire pressure transducers which were installed in standpipes. Figure 3 shows the location of inclinometers and piezometers installed at the wall section in plan. Magnetic extensometers were installed in conjunction with the inclinometer tubes.

Until July 2003 the wall was monitored bi-annually and readings of piezometers and horizontal displacement at verge level were reported. Since 2003 no records of further readings were available to the authors. Full records of the readings were only available until September 1999, 17 months after completion of construction. Installation records including reference levels could also not be obtained as part of the present study. Consequently only inclinometers installed in the particular section discussed here were read again in June 2014.

3.2 End of construction

In Figure 4, observed movements after completion of construction are compared to movements predicted from the original undrained model. The analyses conducted in SAFE showed movement of the inclinometer toes which, for the inclinometer readings, are assumed to be fixed. As there was no independent monitoring of the top of the inclinometers, it cannot be verified if toe movement actually occurred.

Water levels one year after completion of construction in the serviceable piezometer were found to be close to the expected long-term levels, indicating that the installed drainage system was operating efficiently (Figure 5). Compared to the plotted values, until 2003 an increase in P20 and a decrease in P21 were recorded, with all other piezometers being stable.

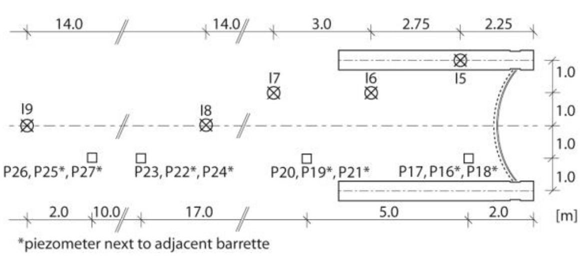

Figure 3. Installed instrumentation at critical wall section.

3.3 Long-term performance

In June 2014, inclinometers I5-I8 were read again. Deformations of these inclinometers at end of construction as well as the new readings are shown in Figure 6. It has to be noted that I5, in the barrette, was commissioned about 6months later than I6-I8 and consequently did not record all movements during construction.

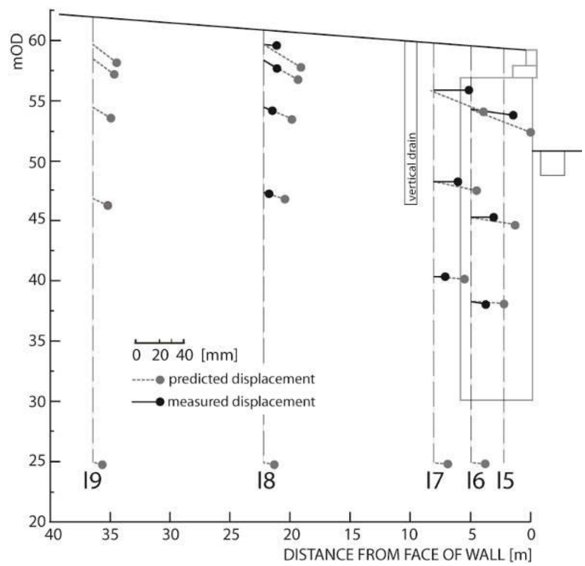

Figure 4. Predicted and measured displacements at end of construction (April 1998).

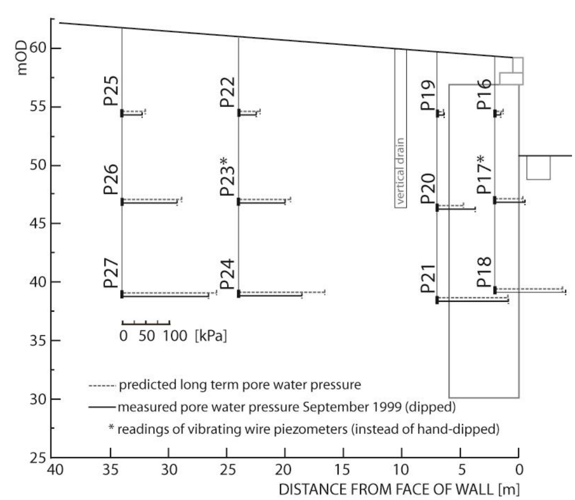

Figure 5. Long term prediction of pore water pressures and measured values 1 year after completion of construction (April 1999).

Even though the new readings were related to baselines scaled from a hard copy printout, agreement between the new inclinometer data with the original records as well as consistency with apparent deviations in inclinometer verticality shown in the historic data gives a good degree of confidence that the deformation profiles recorded reflect the actual ground movements.

To address the uncertainty in the relationship between the top of the tubes and the reference heights, the readings were tested for the influence of a depth positioning error of ±200mm. The check showed that the influence of this potential error in reference level is not significant with respect to the plotted output. Other sources of error were difficult to detect; Spiral survey data are not accessible. Historic hard-copies of the inclinometer readings with and without correction for spiral suggest that the influence was minor. Potential bias errors in the new as well as in previous readings could not be detected with certainty as this would require a number of data points above the inclinometer toe which can be assumed as fixed. Scatter around the lowest data points as well as model predictions suggest though that the toe of the inclinometers is not stable.

Horizontal long-term deformations as predicted in the original analysis are plotted alongside the new readings in Figure 6. The plotted displacements are reduced by the predicted toe movements which were 20mm for I6 and 16mm for I7 as well as I8.

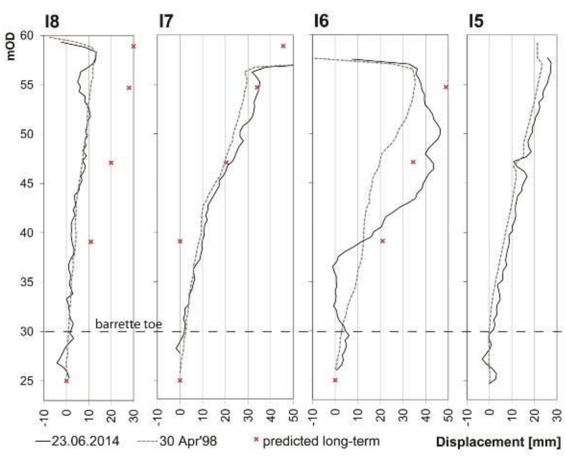

Figure 6. Horizontal displacements at end of construction, recent readings and long-term prediction.

4 DISCUSSION

It can be seen from comparison of the deflected profiles of the wall at the end of construction and some 15 years later that only minor movements of the wall have occurred in the interim. At the top of the wall, increases in deformation of 2mm in I5, in the barrette, and 3mm in I6, between the barrettes (see Figure 3), have been recorded.

Inclinometer I5, in the barrette, shows a kink at around +47 mOD. This is probably due to an inconsistency in the inclinometer tube as it appears in the prior readings and also is apparent in the checksums of the recent measurements, suggesting false readings around this level. Between +25 and +28 mOD, deformation of the inclinometer tube possibly indicates squeezing of the ground below the barrette. Both features are apparent in the B-axis as well, which is not plotted here. However, the recorded movements are very small (±3mm) and further evidence would be required to verify this conclusion.

Inclinometer I6 which is located 5m behind the wall face, adjacent to a barrette, shows backwards movement between +32 and +37 mOD compared to the reading in 1998. This could possibly be due to a bias shift. However, shifting the data to match the prior reading at this level would increase the post construction movement at the top of the wall to 12 mm, which does not seem plausible given the negligible displacement of I5 and I7 in the same period. Between +37 and +57 mOD I6 bulges in the direction of the wall face. Both, the bulge and the backward movement at lower levels seem to show local effects between the barrettes, as they don't appear 3m further back, in I7. In I8, 16m behind the wall face, no additional movements have occurred since construction was completed.

Even though the displacements had been overpredicted in the original analysis for the short as well as in the long-term (Figures 4, 6), horizontal strain in the soil behind the wall, calculated as $(d_{I6}-d_{I8})/17m$, with d_{I6}: deformation at inclinometer I6, level +55 mOD, was similar for predicted and measured values (short-term measured: 0.14%, short-term predicted: 0.12%, long-term measured: 0.14%, long-term predicted: 0.15%). It must be noted that taking into account the system accuracy of inclinometers there is some uncertainty in these values and they should only be considered as indication of order of the actual strains.

In order to demonstrate the performance of this barrette wall in the context of other case studies, the maximum horizontal wall deflection vs maximum excavated depth below ground level has been plotted against case history data given in Ciria C580 (Figure 7). The Thanet Way wall performance is within the range of data for other cantilever walls and is close to the line where deflection is equal to 0.4% of excavated height. It is important to note that unlike other cantilever walls this wall remained as an unpropped cantilever wall after construction and has no long-term propping from the final structure.

Careful record keeping of instruments including their installation as well as changes of monitoring devices can be challenging due to storage of large amounts of data and, as in this case, restructuring of companies or administrations. Future-proofing of storage of data is particularly challenging. This case study shows that long-term monitoring of geotechnical structures and record keeping is not only important for the reassurance of satisfactory behavior but also for the ongoing advancement of design procedures and construction techniques.

5 CONCLUSION

The successful construction and good long-term performance of the barrette wall at Thanet Way shows that barrette walls are an alternative for deep excavations in clay, where anchors or props cannot be considered. In urban areas, where the soil behind the wall is often occupied by other infrastructure, or provides support to other sensitive structures and in particular for sites where the width or arrangement of the excavation does not allow propping either at height or at the base slab, barrette walls can be a viable solution with relatively small wall deflections compared to the retained height. Nevertheless, the ground movements are likely to be larger than those from conventional propped retaining walls and the sensitivity of the nearby buildings and infrastructure needs to be appraised alongside the wall selection.

Because barrette walls can remain stable as an unpropped cantilever wall in the long term, they are a potential design solution for infrastructure applications where base slabs or other propping structures are not part of the final structure.

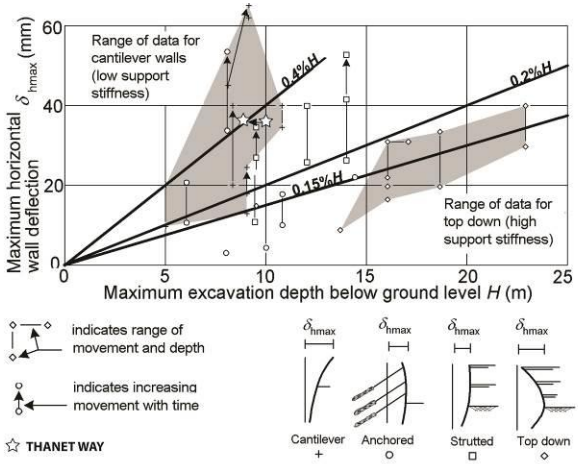

Figure 7. Comparison of barrette wall with case studies after CIRIA C580 Figure A2.1.

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