KINAXIXI MXD, ANGOLA – THE USE OF MICROPILES TO SUPPORT BASEMENT RETAINING WALL

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ABSTRACT

The Kinaxixi Multi Complex development in the centre of Luanda, Angola is currently under construction. It initially comprised two tower blocks for commercial and residential use, linked by a five storey podium with five basement levels for car parking and services. The development has subsequently been extended

In July 2011, Atkins was invited to review the foundations of the proposed development together with the retaining wall design for the five storey basement (~26 metres deep). At the time of the review construction was already underway and the temporary support for the basement excavation, in dense to very dense Luanda Formation sand, was provided by a soldier pile wall with multi level soil anchors. The anchors were installed through the pile shafts with shotcrete arches between the pile centres.

A review of this temporary retaining system showed that it did not have sufficient capacity to form part of the permanent works. Atkins proposed an alternative design comprising a 450mm thick permanent reinforced concrete wall which was accepted by the Client.

The design of the permanent wall indicated that the weight of the wall and loading from the five storey podium development could not be carried on a strip foundation at its base. The basement floor slab, designed by others, could not be sufficiently increased in thickness and therefore, another solution was required.

A piled design was developed and due to the thickness of the permanent wall, micropiles were considered to be the most efficient solution. The initial pile design considered 350mm diameter piles between 8 and 12 metres long. The installed piles consist of vertically drilled Ischebeck Titan 52/26 grout body anchors to carry working loads between 355kN and 753kN. This approach would not normally be employed in Angola. In total 353 piles were installed under some 550 metres of perimeter wall.

This paper describes the development of the pile design, on-site pile testing and pile installation of this novel method of structural support.

1.0 BACKGROUND

The Kinaxixi Multi Complex Development (MXD) is located on the western edge of the Luanda Central Business District on the site of the former Kinaxixi Market. Phase 1 of the development has a footprint of approximately 190 metres x 130 metres and comprises a five storey basement, five storey podium and two towers, each of 31 storeys.

An architect's impression of the completed development is shown in Figure 1 and an aerial view of the site is shown in Figure 2.

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Figure 1. Impression of completed Phase 1 Kinaxixi MXD development.

Figure 2. Site area around Kinaxixi MXD (Phase 2 extension shown as dashed line).

2.0 GEOLOGY & TOPOGRAPHY

The surface topography of the development area slopes gently from approximately 56 metres above sea level (ASL) at the eastern edge to approximately 53m ASL at the western edge.

The site is underlain by a dense sand sequence of the Luanda Formation of Miocene age that was proved to -15m ASL by boreholes drilled as part of the ground investigation for the current development. The upper 20 metres of the Luanda Formation underlying the site contains layers, up to 3 metres in thickness, of very stiff clay and weakly cemented siltstone/mudstone.

In the initial ground investigation SPT N values of greater than 50 blows were recorded in the Luanda Sand from 15 metres below ground level. Atkins specified two additional boreholes from the base of the excavation drilled to 50 metres deep. These indicated that density had reduced due to pressure relief as a result of the excavation and an SPT N value of 50 was achieved some 8 metres below the base of the excavation. The SPT N profiles with depth and the design line for piled foundations below the base of the investigation are shown in Figure 3.

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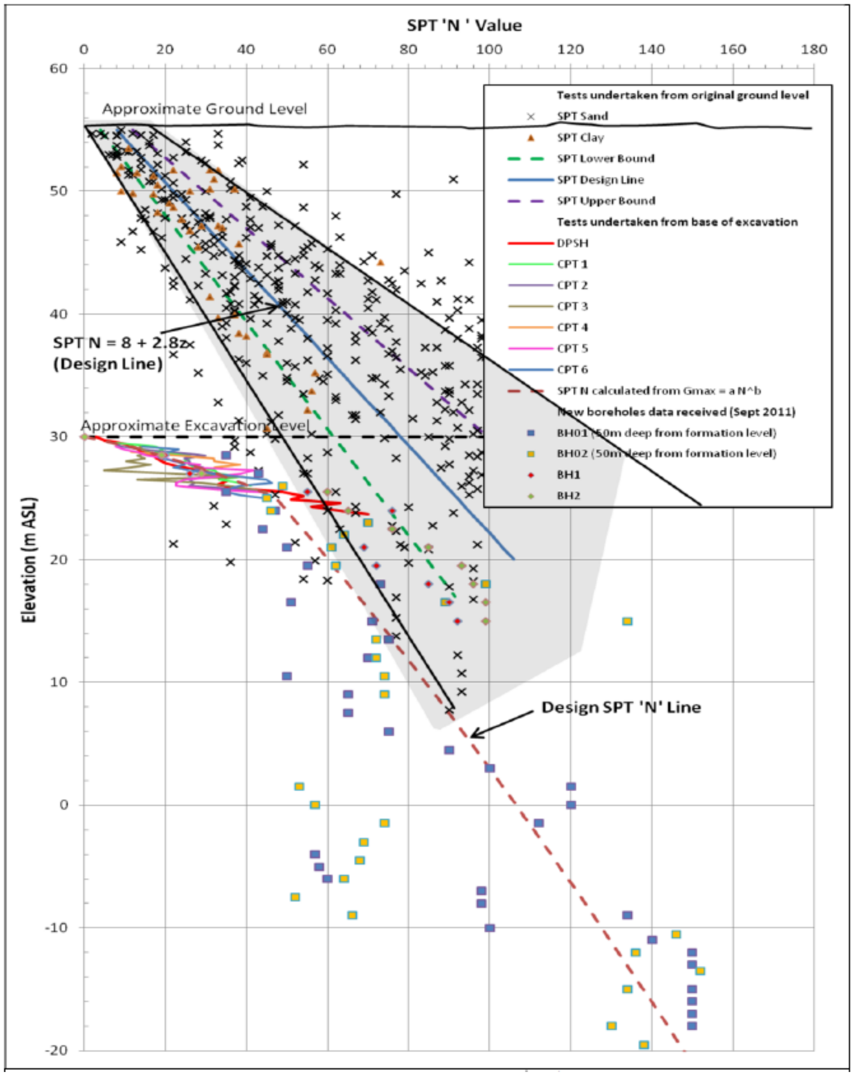

Figure 3. Plot of SPT N value with depth.

Groundwater was encountered in exploratory holes as seepage at approximately 5m ASL which relates to sea level to the west of the site.

3.0 ORIGINAL DESIGN SOLUTION

Korean architects carried out the original design of the development with Korean structural engineers and Brazilian geotechnical engineers. Atkins Ltd was invited in July 2011 to comment on the foundation solution for the main towers and also the design of the permanent basement retaining wall.

At this time, the basement was under construction and supported by a temporary retaining wall. The temporary wall comprises 650mm diameter Continuous Flight Auger (CFA) piles at 1.6 to 2.0m centre to centre with infill gunite arches 175mm thick. Pile length was approximately 20 metres and additional internal piles were installed inside the east and west walls. The wall is anchored with up to five levels of ground anchors drilled through the piles. These comprise 40mm or 52mm diameter bars installed in a 125mm diameter drill hole at 10° inclination to the

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horizontal. They were typically spaced vertically at 3.0 metre intervals with a 9 metre fixed length. Reinforcement in the piles comprises 4no Y40 bars and 4no Y25 bars. Typical details of the wall are shown in Figure 4 and photos of the as-constructed wall are shown in Figures 12 and 13.

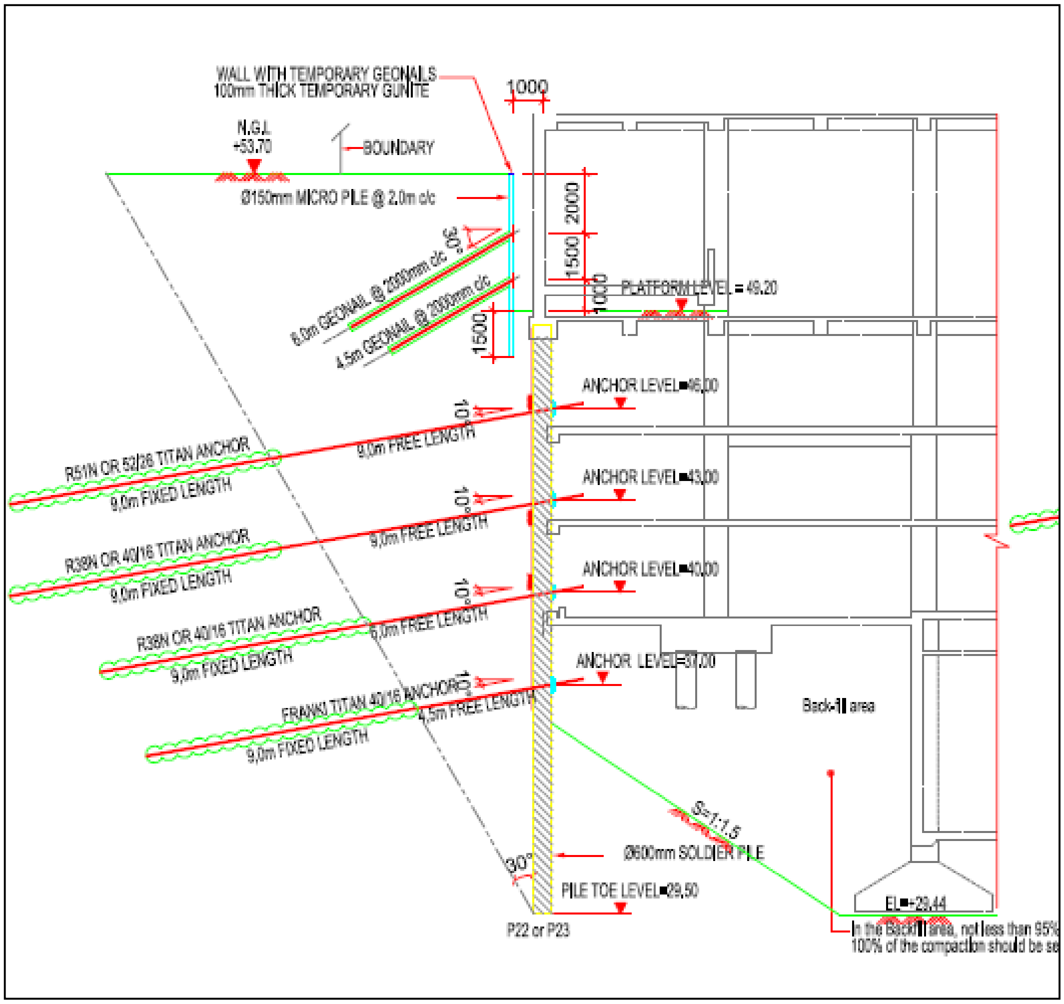

Figure 4. Typical details of temporary retaining wall.

This form of wall construction is common in South Africa but is always of short design life and only constructed in ground with a groundwater table below the toe of the piles. The form of construction was suited to the ground conditions at Kinaxixi MXD although a failure of the wall occurred in the North East corner of the site due to ingress of water close to ground surface from a fractured foul sewer.

The original proposal for a permanent wall was to use the temporary wall construction and key the base and floor slabs into the wall as shown in Figure 5.

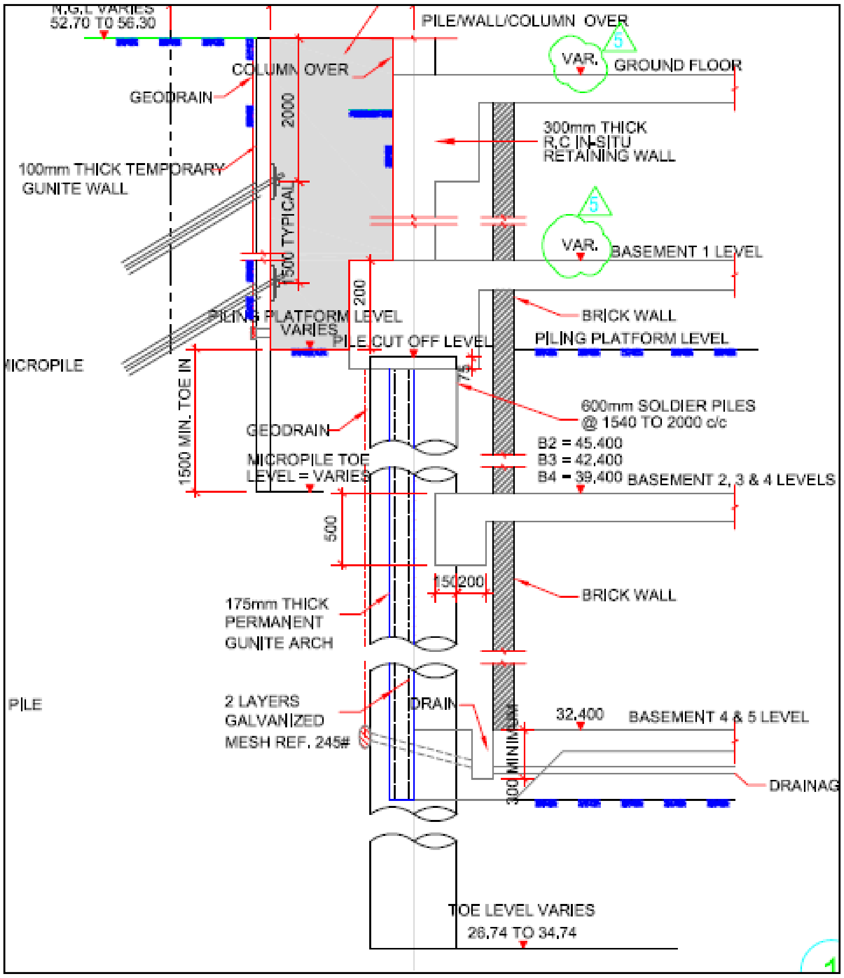

Figure 5. Original permanent retaining wall proposal.

4.0 REVISED WALL DESIGN

As a part of the assessment of the proposed foundation and retaining wall design the proposal to key the basement floor slabs into the temporary works lateral support system was checked using the PLAXIS Finite Element program to BS8110 [Ref 1]. The analysis results indicated that a partial factor on bending between 1.0 and 1.1 and a partial factor on shear between 0.5 and 0.4 depending on pile spacing. It was considered, therefore, that the proposed permanent lateral support system was unsatisfactory due to:

- Inadequate cover to the reinforcing steel
- Inadequate structural provision for shear and bending moment
- Concern regarding the long-term durability of the intermediate gunite support.

A new design solution was, therefore proposed comprising a 450mm thick reinforced concrete wall with B32 vertical bars and B16 horizontal bars at 150mm c/c spacing as shown in Figure 6 acting independently in lateral support from the existing temporary wall.

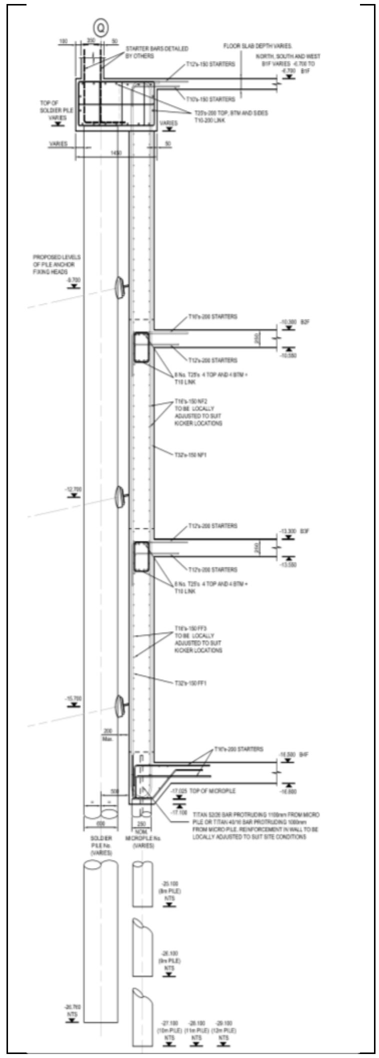

Figure 6. Elevation through proposed permanent retaining wall.

5.0 MICROPILE DESIGN

The design of the foundations for the permanent wall, however, had to be independent of the base slab foundations, as an independent structural engineer had designed these so a piled foundation solution was proposed. The piles had to be of suitable diameter to transfer axial load directly into the ground below the footprint of the wall. A piled solution installed inside the footprint of the wall would have acted in tension together with the beam connecting the pile head to the underside of the wall.

The perimeter wall is subject to additional vertical loading from site development above basement level with applied loads generally in the range 450 to 500kN/m but up to 710kN/m wall. It was considered that a micropile solution would not be capable of the total vertical load so a coupled system was analysed considering the axial capacity of the existing soldier piles together with the proposed micropiles. The design solution required consideration of a number of factors:

- The ultimate load capacity of the existing soldier piles
- The ultimate load capacity of the soldier piles

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- The relative vertical stiffness of the existing soldier piles and micropiles acting together
- Bending moments and shear acting on the micropiles from the proposed permanent wall.

The vertical stiffness of the soldier piles and micropiles were assessed using the PIGLET pile group analysis program and soil design parameters as set out in Table 1.

Table 1. Micropile soil design parameters.

Parameter	Design Value
Bulk Unit Weight (kN/m^3)	19
Angle of internal friction (°)	36
Critical state angle of friction (°)	33
Earth pressure coefficient at rest (k_o)	0.6
Active earth pressure coefficient (k_a)	0.26

Additionally, soil stiffness used in the analysis corresponded to 0.03% strain level as:

$E' = 10.6 + 30z$ (MN/m^2) where z was measured in metres below top of pile.

The axial loads applied in the analysis to the soldier pile wall were:

- Superstructure loads from the perimeter structural wall at ground floor level
- Mass concrete infill between the gunite arches and permanent wall.

If the applied superstructure loads at the capping beam level were over and above the capacity of the soldier piles then the excess load was assumed to be transferred to the permanent wall via the capping beam.

Additional to this load, the vertical loads applied to the micropiles were analysed as:

- Self weight of the permanent wall
- Sub-structure loads from the basement floor slabs (275 to 390kN/m)

The analysis of capacity the soldier piles considered a combination of 50% overburden pressure on the embedded section of the pile for the near face and 50% of the total pile length for the far face. For end bearing N_q was taken as 65 with overburden pressure calculated from the excavated side of the pile. The calculated safe working load of the soldier piles is presented in Figure 7.

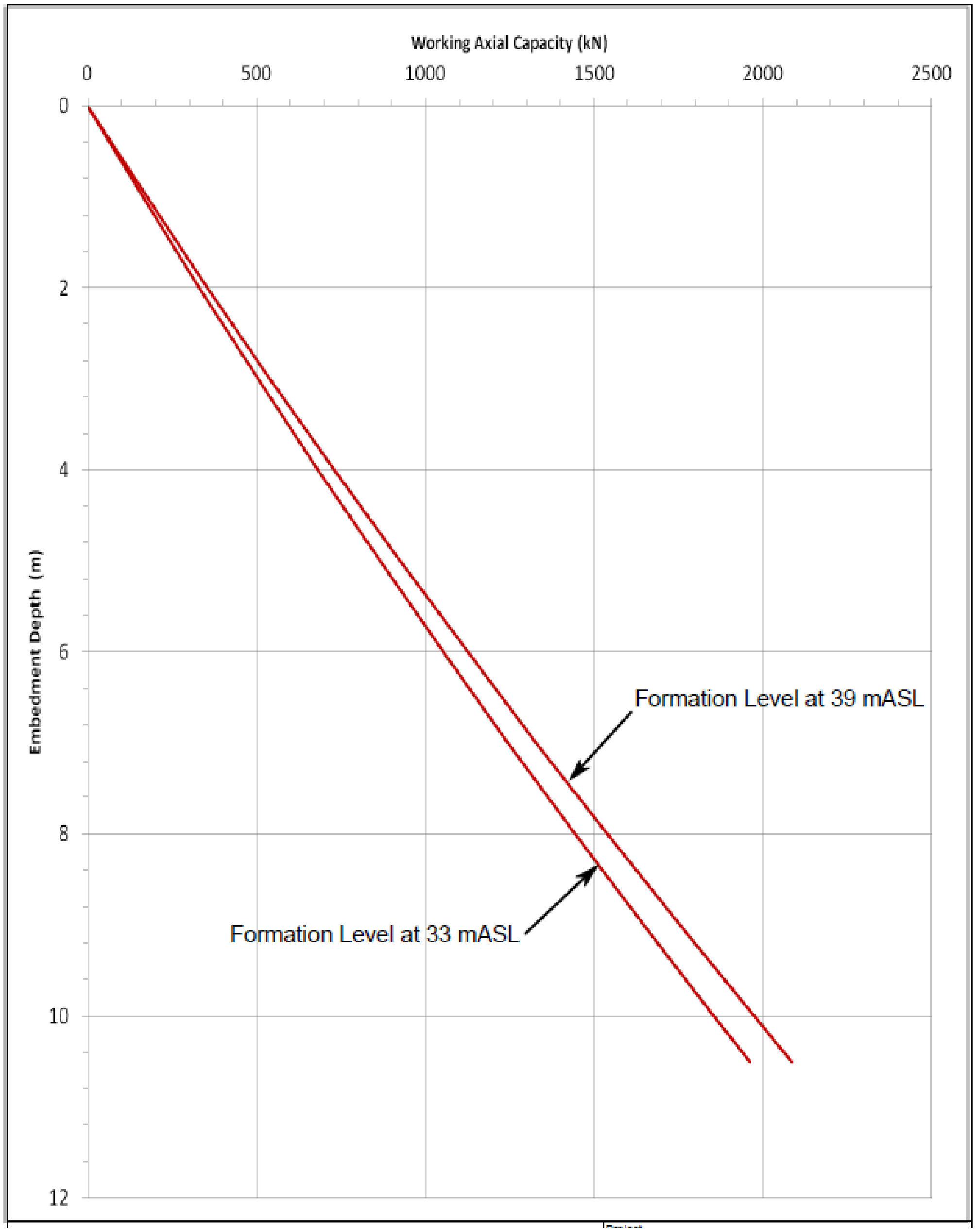

Figure 7. Calculated Capacity of 650mm Soldier Piles.

The micropiles were designed as 350mm, conventionally drilled, in a combination of end bearing and skin friction considering two approaches. Approach 1 considered K_s = 0.7 and an N_q value of 50. Approach 2 considered K_s = 0.9 and ignoring end bearing with a factor of safety of 1.1 as a check to reduce the risk of over reliance on end bearing. The calculated safe working load of the micropiles is presented in Figure 8.

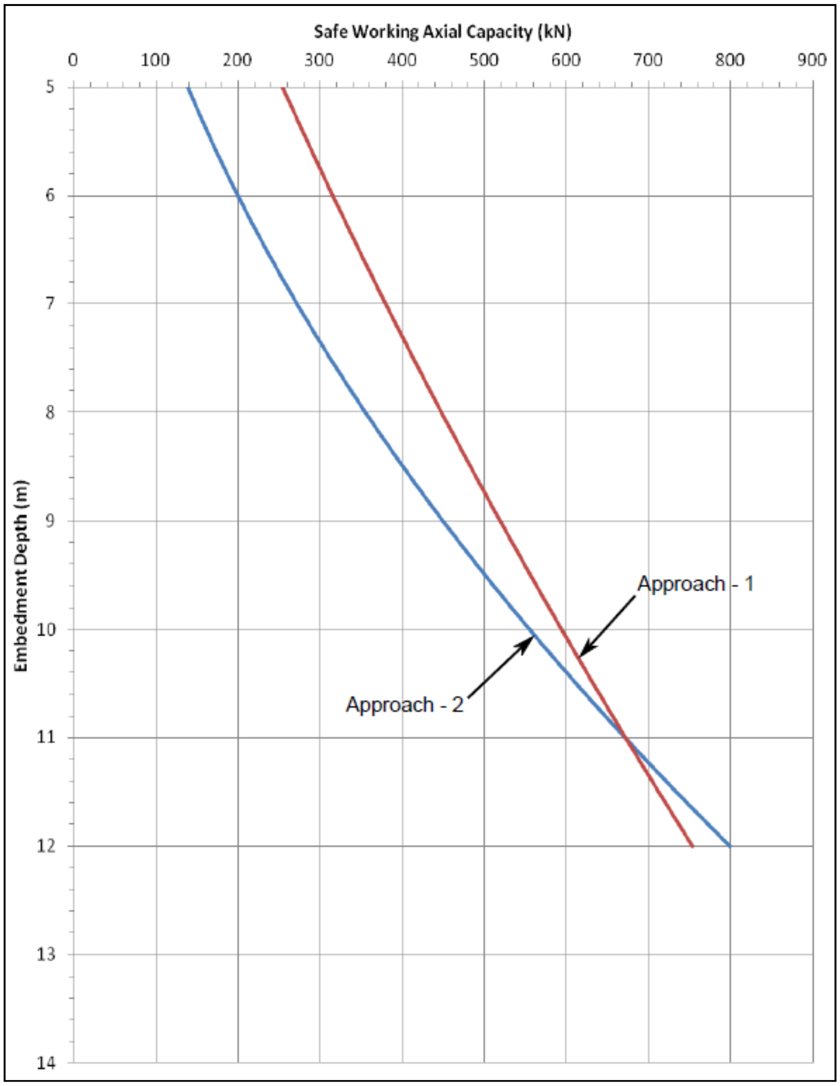

Figure 8: Calculated Capacity of 350mm Micropiles

6.0 MICROPILE TESTING

Micropile testing was specified to 1.5 x working load of 800kN and 600kN after the ICE Specification for Piles and Embedded Retaining Walls [Ref 2].

Two trial piles of 10 metre and 12 metre length were constructed by the piling contractor, Franki, using a Casagrande C6 drill rig in a convenient area of the site close to their intended permanent location in order to simulate construction conditions.

The piles incorporated an Ischebeck 52/26 hollow drilling rod and were of nominal 250mm minimum diameter. Two 12 metre piles were installed as reaction piles and the load was transferred to a reaction beam with a 200 tonne hollow jack. The test set up is shown in Figure 9.

Figure 9. Pile Test Set Up.

The test Load versus Deflection graphs are presented for the 12 metre and 10 metre long piles in Figures 10 and 11 respectively.

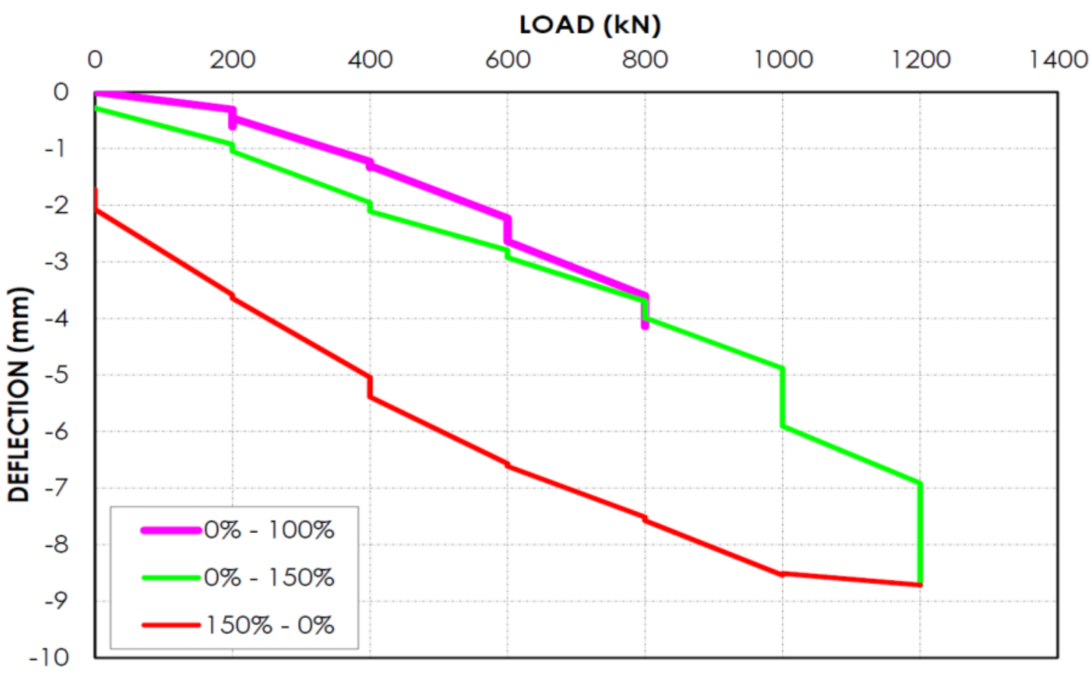

Figure 10. Load versus Deflection Results – 12 metre pile.

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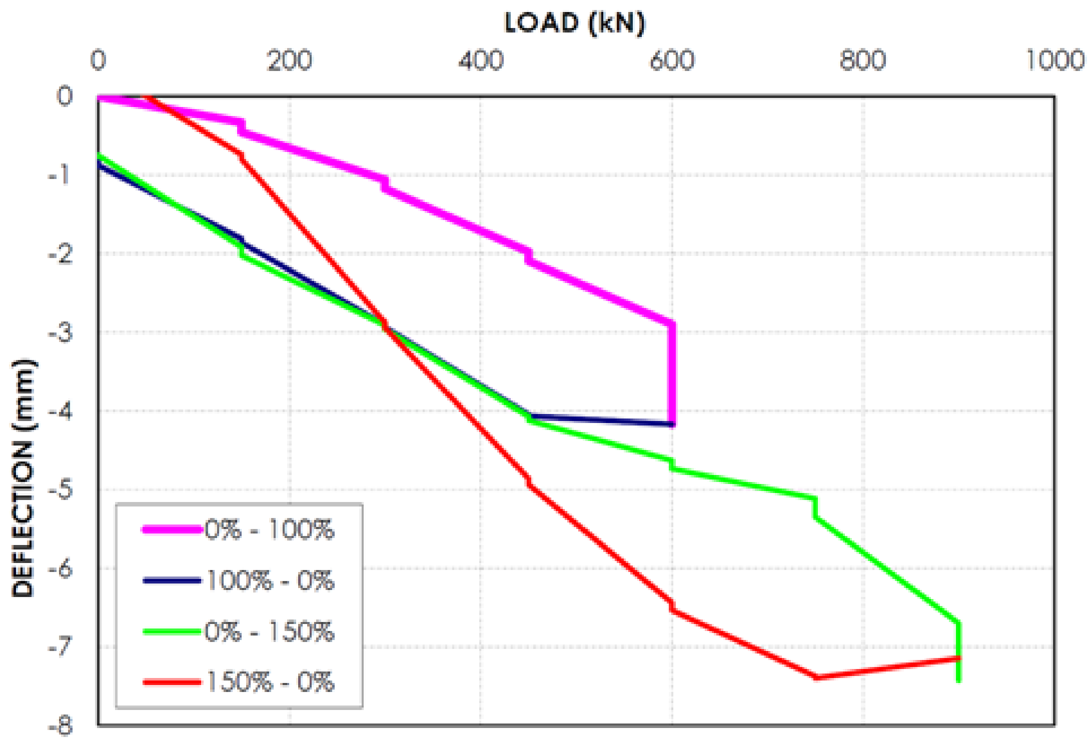

Figure 11. Load versus Deflection Results – 10 metre pile.

The 12 metre long pile showed settlement of 4mm at 100% WL and 9mm settlement at 150% WL. The 10 metre long pile showed settlement of 4mm at 100% WL and 7mm settlement at 150% WL.

Back analysis of the pile test results was carried out using the methods after Chin (Ref 3) and Fleming (Ref 4). The back calculated ultimate shaft friction, end bearing and Factor of Safety are presented in Table 2:

Table 2. Test Pile Skin Friction, End Bearing and Factor of Safety.

	12 m pile	10m pile
Chin		
Ultimate Skin Friction (kN)	1049	968
Ultimate End Bearing (kN)	719	815
Factor of Safety on Working Load	2.2	3.0
Fleming		
Ultimate Skin Friction (kN)	1193	803
Ultimate End Bearing (kN)	736	982
Factor of Safety on Working Load	2.0	2.3

The Factors of Safety were considered adequate to BS 8004 (Ref 5) particularly as the settlement at working load was in the order of 5mm which was the settlement required from the wall analysis to evenly distribute the applied load from the permanent retaining wall and the over site development between the temporary works piling and the micropiles.

The back calculation of ultimate skin friction and end bearing are generally in agreement using both methods. The contribution of skin friction is considerably greater than that calculated for conventionally drilled micropiles were it would be expected that most of the load was carried in end bearing. It is considered that this is due to the irregular surface of the pile shaft created by the drilling method that generates greater friction by dilatency along the shaft surface as the pile settles under applied load. This greater skin friction offsets the reduced end bearing due to the reduced shaft diameter as the Ishabeck anchors are nominal 250mm diameter compared with the 350mm diameter used in the original design calculations.

Back calculated N_q factors are in the range 60 to 100 considering a nominal 250mm shaft diameter. This is considerably greater than those back calculated from pile testing on the main 1200mm diameter structural piles constructed by the bored cast in situ method where a value of 20 was calculated. This indicates that the installation method effects a degree of compaction on the material at the base of the pile and is equivalent to the design values after Berezantsev [Ref 6] for driven piles.

7.0 MICROPILE CONSTRUCTION

A total of 353 no. micropiles were constructed using a Casagrande to lengths and working loads as detailed in Table 3.

Table 3. Summary of installed micropile length and working load

Pile Length (m)	No of Piles	Working Load (kN)
12	1	753
11	99	672
10	99	555
9	149	450
8	5	355

The piles were constructed at locations between the originally constructed soldier piles. Setting out of the microplies and the permanent basement wall took account of the current position of the soldier piles which, when surveyed showed up to 200mm deviation from the intended setting out position and also some rotation due to lateral earth pressure since their construction. A Casagrande C4 rig was used as, at many locations, access was restricted by constructed elements of the basement and podium. An average production rate of four piles per day was achieved, limited mainly by the logistics of construction on a site where main construction activities were being undertaken. Typical views of pile construction are presented in Figures 12 and 13

Figure 12. Pile Construction – North Wall.

Figure 13. Pile Construction – South Wall.

The piles were constructed in the same manner as the two test piles with a nominal minimum diameter of 250mm. In order to achieve this the contractor chose to weld 12mm reinforcing bar onto the head of the self-drilled anchor. Typical anchor heads with welded reinforcing bar are shown in Figure 14. Checks on the integrity of the welded bars were carried out by complete removal of the hollow stem following the first 3m run, for a number of the initial micropiles.

Figure 14. Typical Self drilled anchor heads for micropile construction.

Although the effectiveness of this innovation could not be checked by full exhumation of a completed pile, the tops of the piles were commonly excavated to trim off to some 500mm to pile cut off level and in all cases the pile diameter was found to be satisfactory as shown in Figure 15.

Figure 15. Exhumed top of constructed microplie.

8.0 CONCLUSIONS

The requirement for a permanent inner wall for the basement construction at the Kinaxixi MXD development was not envisaged by the original scheme designers. The design and construction has, therefore, to be independent of the remainder of the basement design that was already under construction when it was being developed.

The requirement for the foundation solution to accommodate the load from the wall and a component of the load from the over-site development led to the development of a micropile solution. The solution was initially designed as conventional bored piles. A suggestion by the contractor to use vertically drilled Ishabeck soil nails was verified by pile testing and lead to a faster rate of production than if conventional bored micropiling had been used.

9.0 REFERENCES

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