Instrumentation Results on Negative Downdrag Force of Abutment Piles Underneath the Reinforced Earth Wall Embankment

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ABSTRACT: This paper demonstrates a case history on the recorded negative downdrag load acting on the abutment piles induced by a 9m high Reinforced Earth (RE) wall. The RE wall is part of the approach embankment constructed over 35m thick granitic residual soil with SPT "N" values ranging from 10 to 20 with depth and located in Selangor State, Malaysia. Vibrating wire strain gauges and inclinometers were installed in the ϕ500mm prestressed spun piles at the middle and edge of the abutment respectively to measure the axial stress and the lateral movements of the abutment piles. A borehole was also sunk into the foundation soil for the installation of the inclinometer and the multi-level spider magnetic extensometers behind the abutment. Recommendations are given on the abutment pile foundation design.

1.0 INTRODUCTION

This paper summarises the instrumentation results obtained from an instrumented reinforced earth (RE) wall abutments of a flyover bridge for an expressway project. The instrumentation scheme is intended to provide factual evidence on the existence and also the magnitude of the negative downdrag on the abutment piles due to imposed loading from the RE wall embankment, in which there was a dispute on the adequacy of bearing capacity due to the relatively weak foundation soil for such heavy reinforced earth fill structure. To attain the objectives of this instrumentation, a few numbers of strain gauges, inclinometers and extensometers were integrated into the abutment piles and the foundation soil behind the abutment to record the actual behaviour of the abutment.

2.0 DESCRIPTION OF PROJECT

The approach embankment structure is located in Petaling Jaya of Selangor State, Malaysia. The instrumentation was implemented at the north abutment, which consists of about 200m long RE wall embankment with the highest vertical dimension of 9m at the abutment. There is a twin-cell reinforced concrete culvert sitting over the starting point of a valley, which is 15 m away from the abutment and underneath the RE wall embankment. The culvert is protected by piled RC slab supporting the overlying RE wall embankment. Figure 1 shows the elevation view of the abutment and the twin-cell culvert. Photo 1 shows the side view of the RE wall embankment.

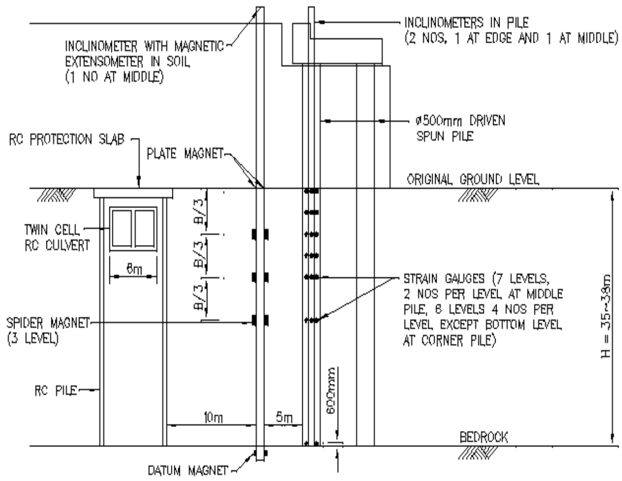

Fig. 1: Elevation View of Abutment and Twin-cell Culvert

Photo 1: Side View of the RE Wall Embankment

3.0 GEOLOGICAL CONDITIONS

The general subsurface soil profile of the site is shown in Figure 2. The residual soils weathered from granite consist mainly clayey silt with occasional silty sand layers. Typically, there is a 4 to 5m thick of hard weathered residual soil with average plasticity index of 25 % immediately overlying the granitic bedrock. The consistency of the granitic residual increases with depth as reflected in the SPT-N profile. Groundwater table is about 10m below existing ground level. There is no encounter of any boulders in the few boreholes sunk for this interchange. The average rock head encountered during pile installation is about 35m to 38m below original ground level, which is consistent with bedrock level as shown in the boreholes. Borehole BH-1 is located about 28m behind the abutment piles and Boreholes BH-2 is 20m away from the abutment towards the adjacent bridge pier.

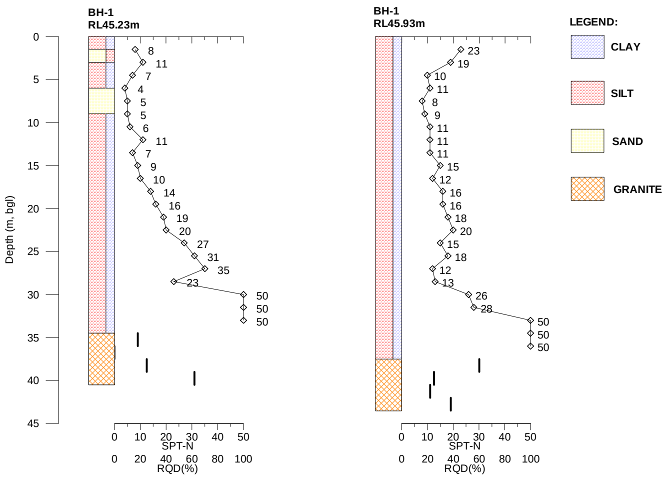

Fig. 2: Typical Borelog Profile

4.0 SEQUENCE OF CONSTRUCTION

The construction sequence of the RE wall embankment is tabulated in Figure 3 and Table 1. In general, the entire embankment including the piling at the abutment took about 4 months to reach the final finished road levels. The abutment piling was carried out before erection of the wall panels and instruments were installed not later than the erection of the second layer of RE wall panels. During filling, the inclinometer tubing was extended upwards at every 3.0m filling interval.

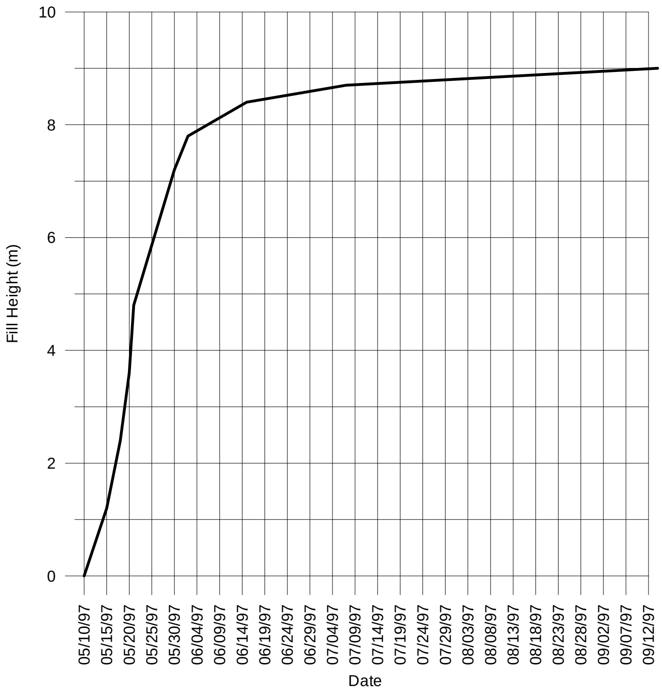

Fig. 3: Fill Height With Time

Table 1: RE Embankment Construction Records

DATE	DESCRIPTION
15/05/97	Quarry dust: 1^{st} & 2^{nd} layer (1.2m)
18/05/97	Quarry dust: 3^{rd} & 4^{th} layer (2.4m)
20/05/97	Quarry dust: 5^{th} & 6th layer (3.6m)
21/05/97	Quarry dust: 7^{th} & 8^{th} layer (4.8m)
22/05/97	Quarry dust: 9^{th}&10^{th} layer (6.0m)
28/05/97	Quarry dust: 11^{th} layer (6.6m)
30/05/97	Quarry dust: 12^{th} layer (7.2m)
02/06/97	1^{st} layer subgrade
09/06/97	2^{nd} layer subgrade
15/06/97	Final subgrade
07/07/97	1^{st} layer road base
14/09/97	2^{nd} layer road base

5.0 DETAILS OF INSTRUMENTATION

Two inclinometers, namely I6 and I7, with multilevel strain gauges were installed in the middle and edge piles (ϕ500mm spun piles) respectively. Another inclinometer, I5, with multilevel spider magnet extensometers was also installed in a borehole behind the abutment. The plan view of the instrumentation layout and the directions of the inclinometer movements are shown in Figure 4.

The arrangement of the strain gauges and the inclinometers in the spun pile are shown in Figure 5. Strain gauges were attached at various levels to the inclinometer tubing by the special designed spacers as shown in Figure 6. The empty annulus between the inner wall of the piles and the inclinometer tubing was subsequently grouted with Grade G30 cement grout to form a composite section of the piles. Readings at every stage of works were taken to indicate that the conditions of the strain gauges and the potential drift of the readings. The installation of the instruments in the abutment piles were carried out after the piles had been driven into the ground and reached the specified set criteria. The reasons of this installation procedure were to avoid potential damage to the instruments due to the driving impact and difficulties in extending the inclinometer tubing and wiring for strain gauges during the piling operation, particularly jointing of the piles. The multi-level spider magnetic extensometers were attached to the inclinometer tubing with sufficient clearance for sliding along the tubing. All the extensometers were referred to the base magnet installed at the depth of 0.5m into the granite bedrock. In this case, a firm reference datum can be established with minimum interference from the above ground construction activities. Figure 7 shows the perspective view of the instrumented spun pile. The installation details of magnetic spider extensometers are shown in Figure 8. The arrangements of closer spacing of extensometers at the upper soil layer and larger spacing of extensometers at the lower soil layer can provide a better resolution of the soil settlement profile as the upper layer is expected to settle more.

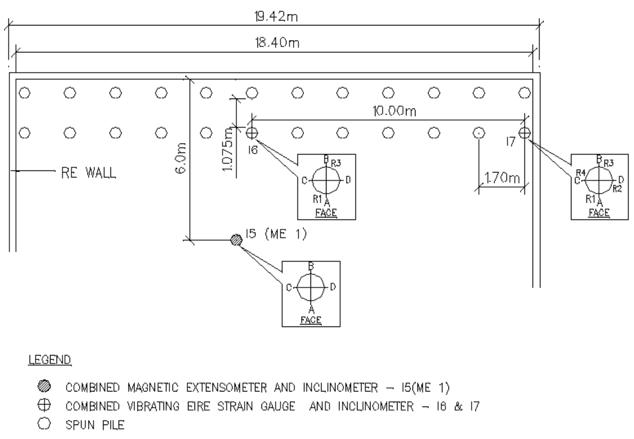

Fig. 4: Layout of Instruments

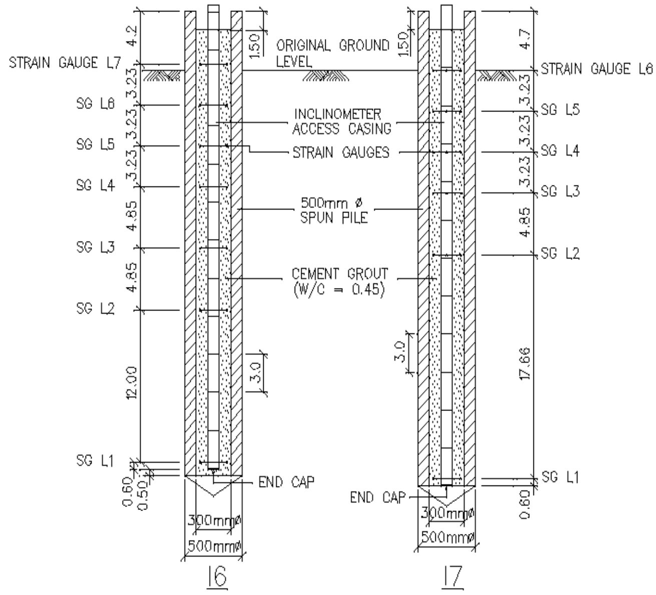

Fig. 5: Elevation View of Instrumented Test Pile

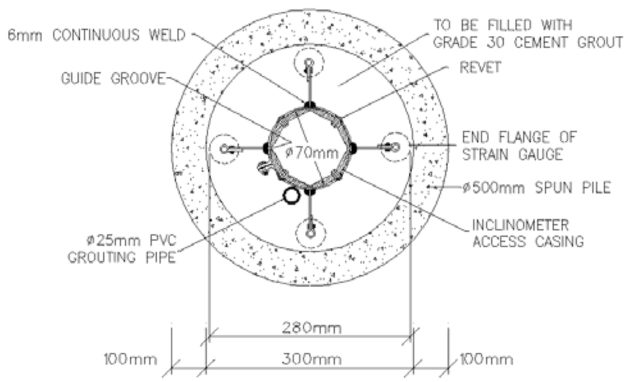

Fig. 6: Sectional View of Instrumented Test Pile

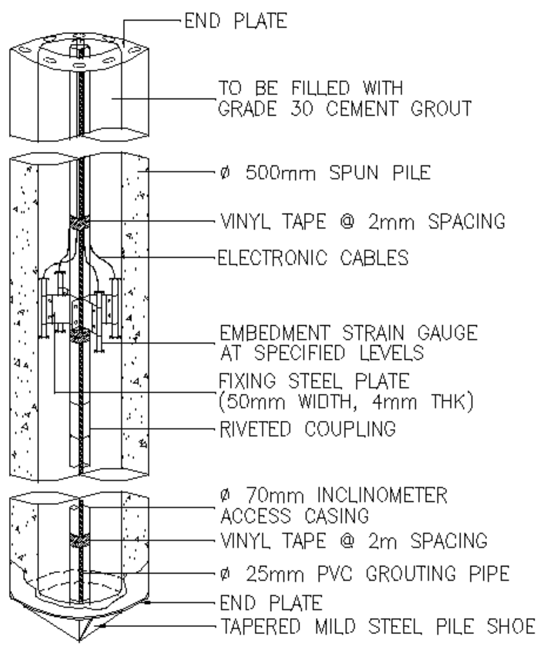

Fig. 7: Perspective View of Instrumented Test Pile

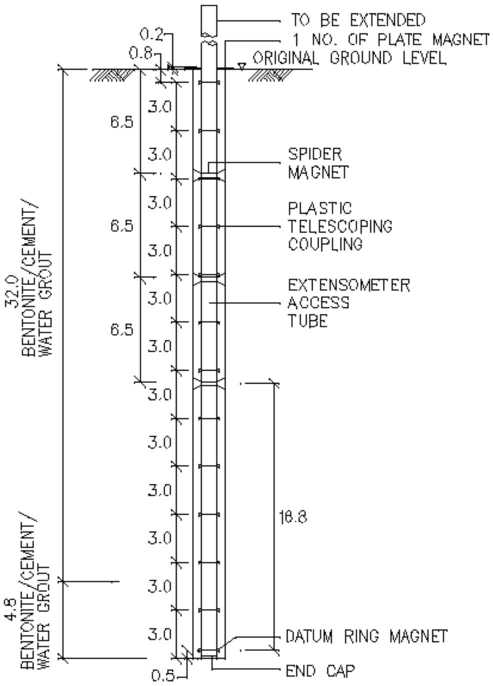

Fig.8: Elevation View of Magnetic Spider Extensometer

Photo 2: In the Process of Installing Strain Gauges and Inclinometer in a Spun Pile

6.0 INTERPRETATION OF RESULTS

The axial compression profiles for the ϕ500mm spun piles, I6 (middle) and I7 (edge), are visualised in Figures 9 and 10 respectively. The SPT-N profile from the two adjacent boreholes and the settlement profile measured from magnetic spider extensometers at Inclinometer I5 are also incorporated in Figures 9 and 10 to show the effect of ground settlement and the subsoil consistency on the induced downdrag force on the abutment piles. From the above results, it is not difficult to observe that the compressible soil strata for this bridge interchange is at the first 15m below original ground level, where the neutral planes for middle pile and the edge pile are 7m and 10m below ground level respectively. If one was to choose the neutral plane before instrumentation results are available, it is likely that deeper neutral plane will be assumed based on normal practise for end bearing pile, probably at the hard stratum at about 30m below original ground level.

The inclinometer profiles are shown in Figures 11 12 and 13 respectively. There were some disturbances on the inclinometer readings close to or above the pile top and above the original ground level due to free-standing length of the tubing and subsequent extension after every stage of filling. This is particularly obvious at the inclinometers installed in the borehole behind that abutment. Therefore, the readings at these disturbed areas were not interpreted in this study. The construction record of the RE wall embankment tabulated in Table 1 provides good indication of the imposed loading on the ground.

The estimated final imposed dead load from the bridge structures including deck, beams, bank seat and wing wall, transition slab and the imposed earthfill on the slabs on the abutment piles is estimated to be about 275kN per pile at the pile cut off level. The bank seat was taken as part of the imposed dead load is because there will be no contact support the bank seat from the fill after the fill has settled some time later though the full contact did exist during casting the bank seat. There is some minor drag load from the granular fill on the pile shaft between the pile cut off level and the ground level because of the settlement induced by the RE wall embankment.

Some significant values measured from the instruments are listed below:

- Maximum vertical movement = 65mm at OGL
- Maximum axial compression for I6 = 920kN at 10m BOGL
- Maximum axial compression for I7 = 1100kN at 7.5m BOGL
- Maximum lateral movement for I5 (non-pile) = 8mm at OGL
- Maximum lateral movement for I6 (pile) = 24mm at 8m above OGL
- Maximum lateral movement for I7 (pile) = 28mm at 6m above OGL

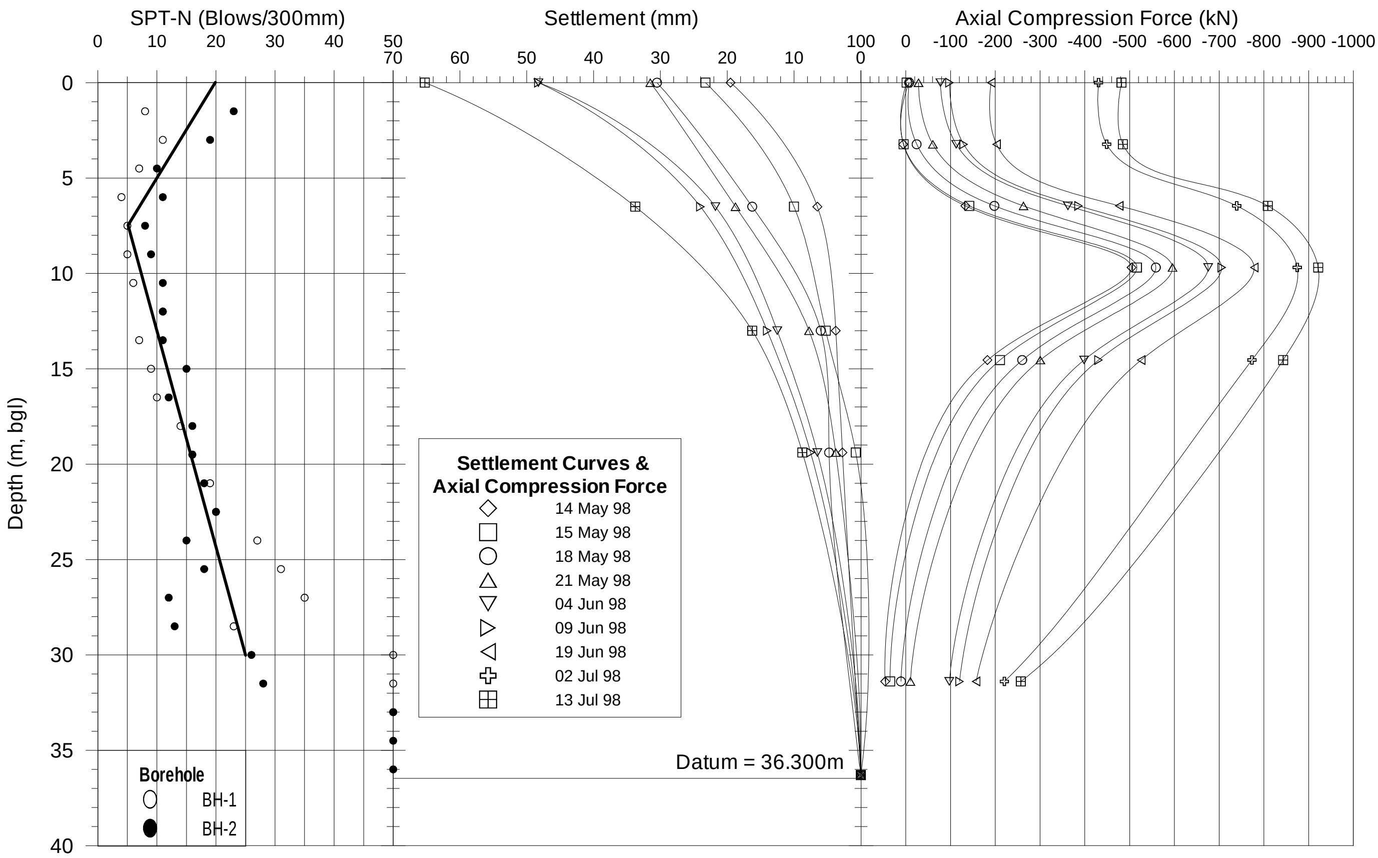

Fig. 9: Subsoil Consistency, Subsoil Settlement Profile and Pile Axial Load for Instrumented Pile (I6, Middle Pile)

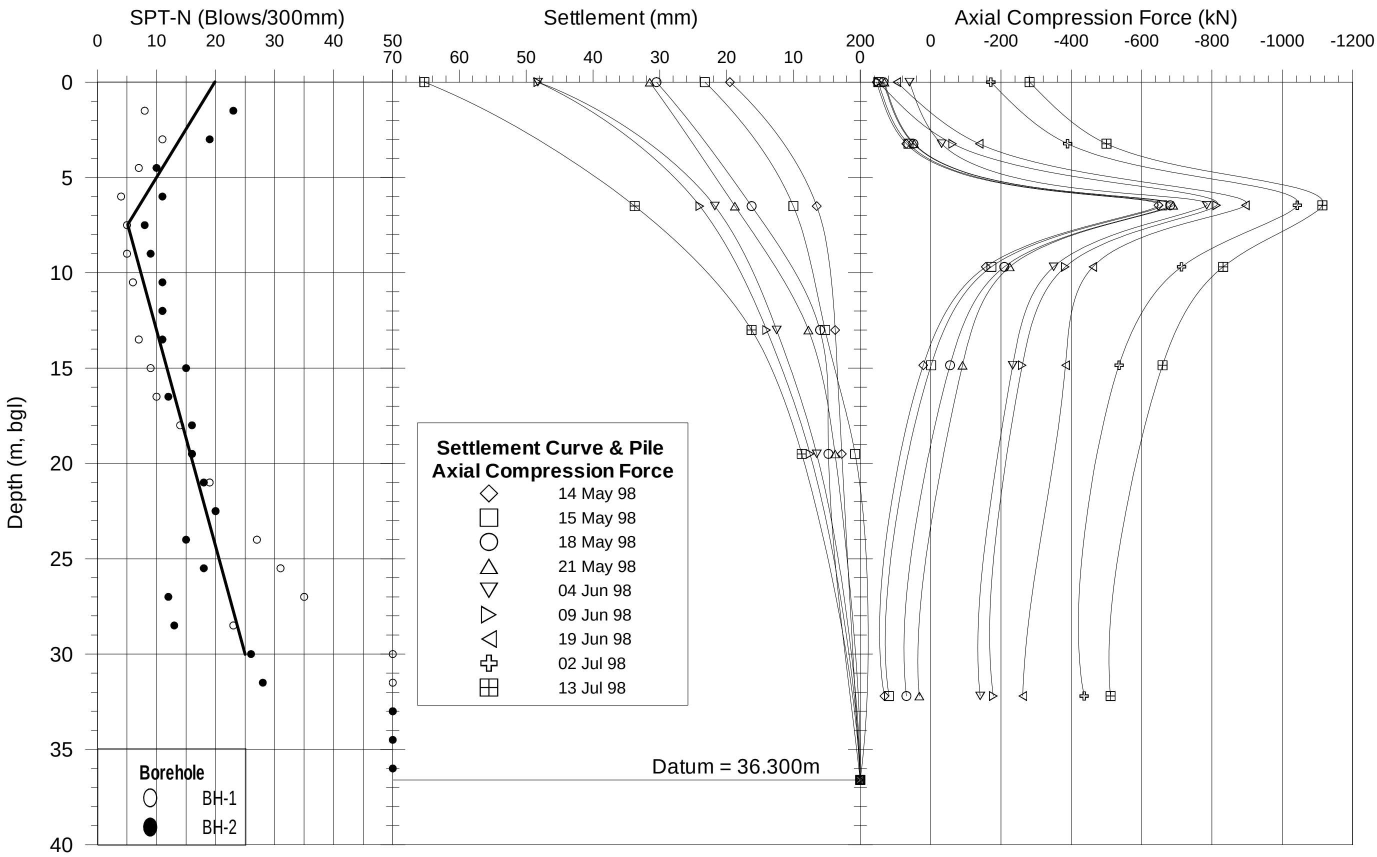

Fig. 10: Subsoil Consistency, Subsoil Settlement Profile and Pile Axial Load for Instrumented Pile (I7, Edge Pile)

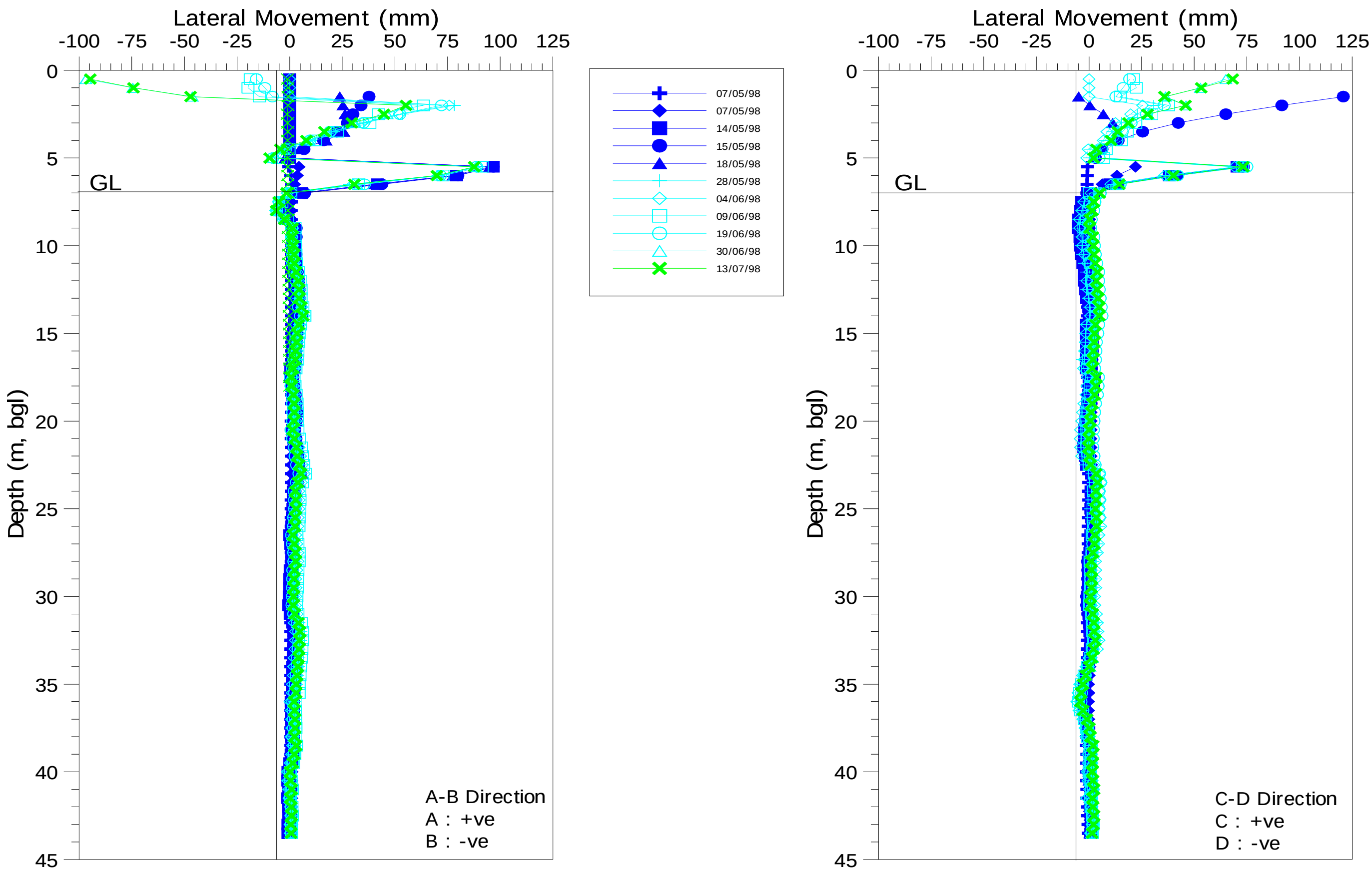

Fig. 11: Lateral Movement of Inclinometer I5

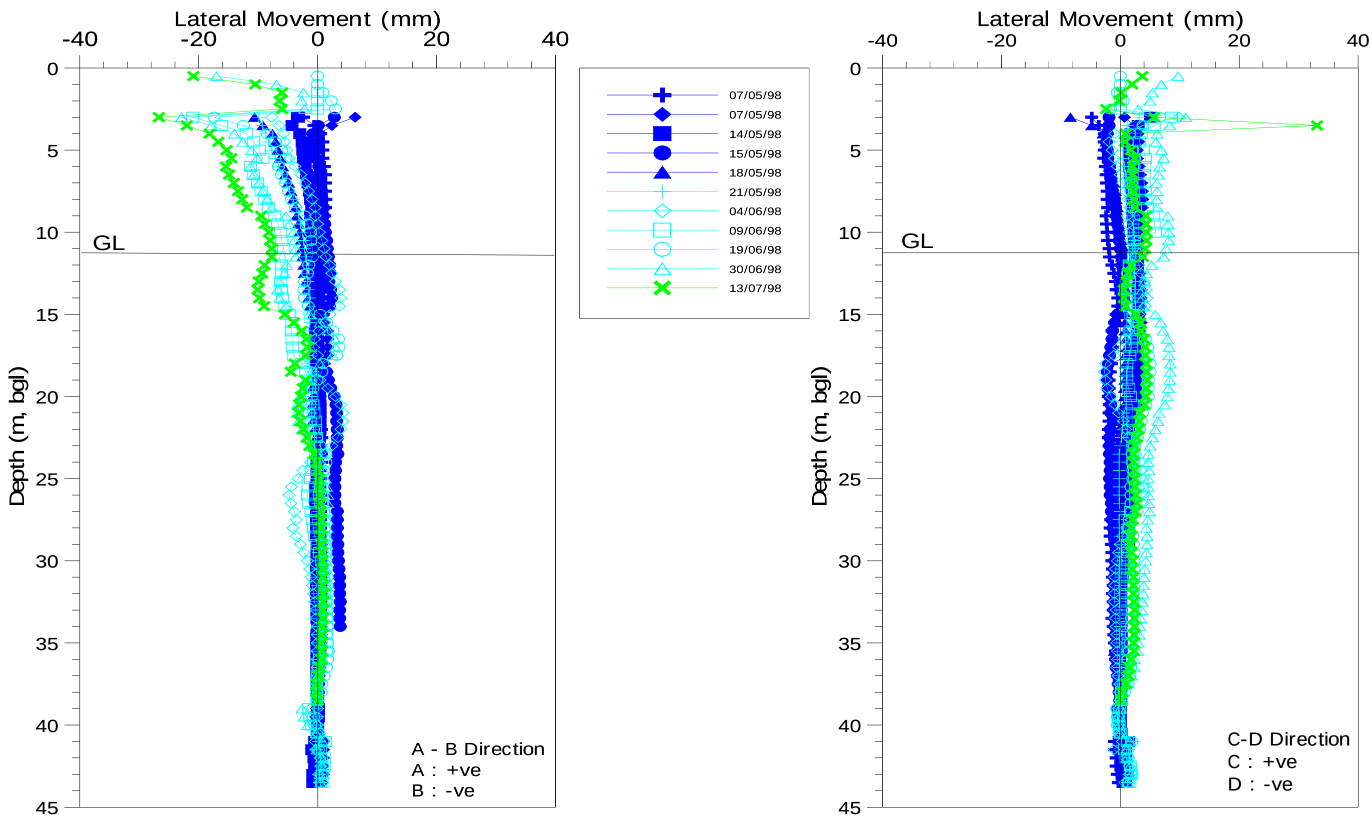

Fig. 12: Lateral Movement of Inclinometer I6

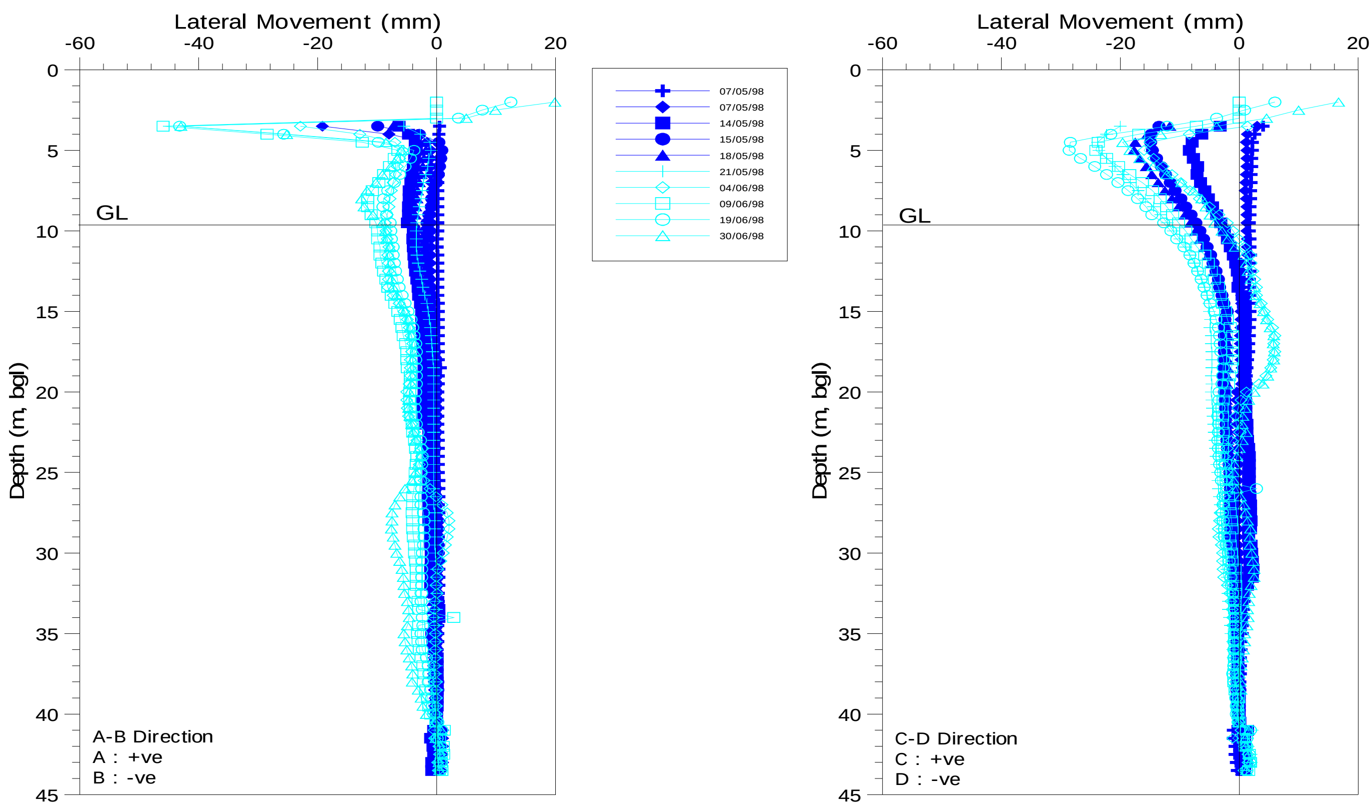

Fig. 13: Lateral Movement of Inclinometer I7

7.0 DISCUSSIONS

The following discussions will concentrate on the instrumentation results for the abutment.

The final measured negative downdrag loads induced on the ϕ500mm spun piles for portion below the ground and the observed neutral plane is between 440kN (I6) at the middle pile and 820kN (I7) at the edge pile, in which the skin resistance factors, β, in effective stress approach for estimating the negative skin friction are equivalent to 0.11 and 0.36 respectively. Little (1994) has given a very comprehensive review on the range of β values for various soil types. However, maximum pile axial loads of 920kN for I6 and 1100kN for I7 are still within the limit of the allowable structural capacity of 2200kN for ϕ500mm prestressed spun pile even including the estimated additional live load of about 180kN per pile in service conditions. In fact, the application of the imposed live load will reduce the negative downdrag force on the pile as highlighted by Fellenius (1972) and Leifer (1994).

The middle pile has less drag load than the edge pile in this case. This phenomenon is somehow consistent to the "hang up" effect, which is taken the reduction of vertical effective stress, σ_{vo}', in the soil is decreased to σ_{vf}' at the vicinity of the pile, as highlighted by Zeevaert (1959). Kuwabara and Poulos (1989) also demonstrates that the outer piles in the pile group develop less downdrag force than the inner piles. In the extreme cases, tensile load can be developed at the outer pile if the pile cap is sufficiently rigid. Briaud et al (1991) summarise various techniques to compute the downdrag force on pile group. It is also observed that the top 3m subsoil has little contribution to the downdrag force. This may be due to the laterally enlargement of interface contact between the pile and the soil by the driving process and therefore leads to lower contact stress on the pile shaft at this portion. However, this enlargement did not occur at every pile point.

The normal hand calculation procedure using rigid-plastic model is likely to over-estimate the negative downdrag significantly as full mobilisation of ultimate skin friction is usually assumed for both the negative and positive skin frictions as highlighted by Matyas and Santamarina (1994). The location of the neutral plane is also usually assumed at the rigid stratum for end bearing pile or some fraction of the pile embedded length for frictional piles, in both cases, over-estimation of the downdrag force is inevitable. The hanging effect from pile group may also contribute some reduction of the downdrag force, which is not considered in normal analytical procedure. For a given free field settlement profile in the subsoil, the estimation of the skin friction profile along the pile shaft, and the mobilisation of the skin friction before ultimate skin friction and after post failure skin friction, especially for the skin friction above the neutral plane, have significant influence on the estimated downdrag force. Wong and Teh (1995) has presented a simplified numerical procedure for the analysis of negative downdrag on pile using non-linear hyperbolic soil spring to model the load transfer behaviour between the settling soil and the pile elements. All load transfer behaviour of pile should be calibrated against the soil types, method of pile installation, size effects and pile types. Tan et al (1998) has presented the load transfer behaviour of cast-in-place piles in tropical residual soils of Malaysia. For an economic design, these parameters should be carefully interpreted and appropriate analytical model should be selected.

The lateral movements of the piles in the foundation soil in this case are small. Therefore, the bending moment on the piles should be insignificant.

8.0 CONCLUSIONS

A case history of negative downdrag at the two instrumented abutment piles of an RE wall embankment on granitic residual soil is presented. Details of a successful implementation of instrumentation in the abutment piles have been discussed. The following conclusions are drawn:

1. Negative downdrag does exist in residual soil, which can adequately support a typical 9m high RE wall embankment, if there is loading over the relatively compressible subsoil around the pile foundation. Therefore, appropriate allowance on downgrading the pile capacity catering for such consideration should be made.
2. The calculated skin resistance factor, β, in the effective approach for downdrag estimation of the two instrumented abutment piles ranges from 0.11 to 0.36.
3. The observed neutral planes are located at 7m to 10m below original ground level, which is significantly higher than the hard stratum.
4. "Hang Up" effect is observed between the middle pile and the edge pile. The difference of the downdrag forces between the two piles is about 380kN.
5. No structural integrity problem of the abutment piles at this interchanges is observed.

9.0 REFERENCE

1. Briaud, J. L., Jeong, S. and Bush, Randy, (1991), Group Effect in the Case of Downdrag, *ASCE GSP 27, Geotechnical Engineering Congress 1991*, pp.505-518.
2. Fellenius, B. H., (1972), Downdrag on Piles due to Negative Skin Friction, *Canadian Geotechnical Journal*, 9(4), pp.323-337.
3. Kuwabara, F. and Poulos, H. G., (1989), Downdrag Forces in Group of Piles, *Journal Of Geotechnical Engineering*, Vol. 115, no. 6, June, ASCE, pp.806-818.
4. Leifer, S. A., (1994), The Effect of Live Load on Downdrag Forces, *ASCE GSP 40, "Vertical and Horizontal Deformations of Foundations and Embankment"*, pp.949-961.
5. Little, J. A., (1994), Downdrag on Piles: Review and Recent Experimentation, *ASCE GSP 40, "Vertical and Horizontal Deformations of Foundations and Embankment"*, pp.1805-1826.
6. Matyas, E. L. and Santamarina, J. C., (1994), Negative Skin Friction and the Neutral Plane, *Canadian Geotechnical Journal*, 31, pp.591-597.
7. Tan, Y. C., Chen, C. S. and Liew S. S., (1998), Load Transfer Behaviour of Cast-In-Place Bored Piles in Tropical Residual Soils of Malaysia, *13th South East Asian Geotechnical Conference*, Taipei, R.O.C., November 1998, pp.563-571.
8. Wong, K. S. and The, C. I., (1995), Negative Skin Friction on Piles in Layered Soil Deposits, *Journal of Geotechnical Engineering*, June, ASCE, pp.457-465.
9. Zeevaert, L. (1959), Reduction of Point Bearing Capacity Because of Negative Friction, *Proc. 1st Pan-America Conference Soil Mechanics Foundation Engineering*, Mexico City, 3, pp.1145-1152.