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Title of paper: Soil nailing in Dublin Boulder Clay

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ABSTRACT

Extensive areas of Dublin City are underlain by a very stiff to hard lodgement till, known locally as Dublin Boulder Clay. Over the past number of years there has been increasing use of soil nails mostly to support temporary slopes for excavations, road widening and cut and cover tunnels. Although guidance for nailing in stiff clays exists, the very high strength and stiffness of the Dublin soils mean that it is conservative to use these established techniques and further optimisation may be possible. This paper presents data from the Dublin Port Tunnel project for a large set of short term pullout tests carried out on nails from a slope some 12 m in height together with slope monitoring data. The significant influence of drilling technique is discussed. Optimisation of temporary soil nails in Dublin Boulder Clay is appraised. The relationship between bond strength and actual nail performance is discussed drawing on data from a heavily monitored slope. Measured data on the vertical and horizontal movements of the slope together with data from piezometers confirm the excellent behaviour of the soil nailed slopes.

KEY WORDS: Dublin, Boulder Clay, Soil Nails, Slopes, Observational Method, Pull-out, Creep, Pore Water Pressures

1 INTRODUCTION

The Dublin Port Tunnel (DPT) provides a critical link between a major sea port in Dublin and the motorway network serving the Republic of Ireland. Construction involved a design & build contract that encouraged innovation by the Contractor. In an approximately 850 m length of the northern part of the DPT (Chainage 700 m to Chainage 1550 m), where the excavated depth for cut and cover construction was between 7 m and 12 m, shotcrete faced soil nailed slopes were used to retain steep cuts (65^o – 80^o). The site imposed difficult constraints, notably that the diverted M1 motorway at the slope crest must remain safe and operational throughout (Figure 1). However the temporary slopes were only required to stand for 6 months to allow construction of the tunnels and backfilling. High lane rental charges also applied during the motorway diversion. The contractor wanted a solution that was fast to construct but did not want to pay for a longer stand-up time than was strictly necessary.

An observational approach, aided by sophisticated finite element modelling, was developed (Long et al. 2003; Kovacevic et al. 2008). It provided a design stand-up time of 1 to 1.5 years by using a partly nailed slope. The slope would collapse after this period as stability relied on excavation induced soil suctions, augmented by soil nails. Nails were always provided in the top 5 m (Figure 1). Additional nails were provided at depth if indicated by the observed ground conditions or monitored results. In the end, some 1700 linear metres of slope was constructed with 18530 m^2 of shotcrete facing and 7460 soil nails. This represented savings of more than 20% in nail quantities relative to a fully nailed slope with a long design life. There were addition savings in time and costs equivalent to 12 nail-rig weeks of construction, equipment and personnel.

As part of this work, careful nail pullout tests and detailed monitoring of the nailed slope were undertaken. The results of these tests together with the quality of the grout and shotcrete used in addition to the data from the slope monitoring are the subject of this paper. As soil nailing is increasingly being used in glacial tills in Ireland the data and lessons from this paper could be of wider interest in Ireland and other areas with glacial deposits.

2 GROUND CONDITIONS

Full details of the site location, geology and ground conditions are given by Skipper et al. (2005) and Long and Menkiti (2007). In summary, the strata sequence comprises 0.3 m to 1 m of made ground or loess over Dublin Boulder Clay. The boulder clay is a low plasticity, very stiff to hard, gravelly, sandy clay. Permeable lenses and more cohesionless zones are occasionally encountered.

The boulder clay has been sub-divided into units reflecting the degree of weathering and different depositional phases (Skipper et al. 2005). At the site (Figure 2), the top 0 to 1 m of the boulder clay is the upper brown boulder clay (UBrBC) (undrained shear strength ≈120 kPa). This is underlain by the upper black boulder clay (UBkBC) (undrained shear strength ≈300 kPa). and the lower brown boulder clay (LBrBC) respectively, see Figure 2.

Values of water content versus depth for a single borehole from the site (BH1020W) are shown on Figure 2b. In general water content is highest in the UBrBC, sometimes exceeding 20% but on average is about 13%. For the UBkBC water content is generally less than 10%. Bulk density (ρ_b) values (Figure 2a) are generally high and are on average 2.3 Mg/m^3 for all 4 units of the DBC. Liquid (w_L) and plastic (w_P) limit values (Figure 2b) are relatively constant with depth and show average values of about 29% and 15% respectively.

Liquidity index [$I_L = (w-w_P)/I_p$] can sometimes be a very useful parameter for distinguishing between different strata, e.g. as demonstrated by Hight et al, (2003) for London clay. I_L values for the BH1020W at the DPT site are shown on Figures 2c. For the UBkBC values are more or less constant with depth and fall in the range –0.4 to –0.5. For the UBrBC I_L usually tends to be closer to zero but is also relatively constant with depth. These slightly lower values reflect the weathering processes subjected to this unit. Data for the LBrBC and are more scattered but on average are also less than zero. The LBkBC has on average the lowest values being about –0.66, consistent with this material having been subjected to the most intense shearing. All of these negative values reflect the highly "overconsolidated" state of the material.

The boulder clay is very stony. Its gravel content is typically 30±10%, its clay content is 15±5%. The ground water level is about 2m below ground level.

3 PERFOMANCE OF SOIL NAILS

3.1 Previous experience of soil nailing in Dublin boulder clay

To the knowledge of the authors there are only two other previously published reports of soil nailing in Dublin boulder clay. The first is associated with works at Dublin Airport, which was reported by Pedley (2000). Here the nails were required to temporarily support an approximately 10 m near vertical slope. Nails comprised 25 mm or 28 mm threaded bars installed in a 114 mm borehole. They were 6 m to 8 m long and were installed at a constant vertical spacing of 1.5 m. Horizontal spacing varied between 1.25 m and 2 m. Pull out tests demonstrated bond stresses of more than 200 kPa, well in excess of the design ultimate value, confirming the standard design procedures, as used for other materials, are conservative in these conditions.

Peters et al. (2012) describe the use of soil nails for temporary works during the upgrade of the Dublin orbital (M50) motorway widening. For the section described the slope was inclined at 85° and was approximately 3.6 m high. Nails were 3.5 m long spaced at approximately 1 m centres both horizontally and vertically and mostly installed at an angle of 12.5° to the horizontal. Slope movements were modest and mostly less than 5 mm.

3.2 Dublin Port Tunnel project - pullout tests

For the Dublin Port Tunnel project, the observational method adopted was verified using sacrificial soil nail pullout tests. Nine preliminary tests were carried out prior to construction, to inform the design work. Additional nail tests were carried out during construction to: (i) verify conformance; (ii) allow optimisation of the design; and (iii) optimise construction by assessing and accommodating changes in installation methods, ground conditions and construction programme. The number of nail pullout tests implemented averaged about 1 test / excavation lift / 100 m run. Excavation was in lifts of 2.5 m to 3 m depth.

The sacrificial test nail layout is shown in Figure 3, with nails inclined at 10^o to the horizontal. Pullout tests were undertaken at various depths, on nails installed in exactly the same manner as the main works nails. Drilling was by rotary methods with rock roller bit or down-the-hole hammer. Air / foam flush was used, powered by a 7.5 bar or 12 bar compressor. Figure 4 shows a typical borehole before nail installation. Gravel and cobbles in the stony soil become incorporated into the grout, giving a very rough interface.

The nail hole was grouted, using a tremie extending to the base, and then the tendon inserted. For the pullout test, a 32 mm diameter steel tendon was specified to promote failure at the soil-grout interface. The grout mix was specified as neat OPC and water, with a water/cement ratio less than 0.45 and characteristic 3-day and 28-day cube strengths of 15 MPa and 40 MPa respectively. Pullout tests were generally carried out 7 to 13 days after grouting.

Some tests were carried out at 3 to 5 days, to verify nail performance in a contingency scenario and to allow faster construction. Some of these early tests failed at the tendon-grout interface, not at the grout-soil junction ("T/G", Figure 9). Test nails were hydraulically jacked at a rate of 1 mm / min. Monotonic tests with load-unload loops and creep tests with four maintained load stages were conducted.

3.3 Pullout performance

The pullout tests revealed the following characteristics (Figures 5 to 9). Figure 5 shows the measured unit skin friction (soil-grout interface). A very high ultimate skin friction (or bond strength) was mobilised at the soil-grout interface, which was higher than the undrained strength. This is consistent with the rough soil-grout interface obtained.

Lower bond strengths were mobilised in cohesionless zones / lenses (Figure 9). Mobilisation of skin friction was rapid, within 5 mm to 10 mm displacement. This implies that working loads would be generated at low displacements. Failure at the grout-soil interface exhibited a very ductile response, without strain softening. The maximum skin friction was sustained over large displacements of 50 mm or more. The exception was a few tests in silt or cohesionless zones, which showed a slightly brittle response. These characteristics are compatible with the observational design used.

A photograph of an exhumed test nail is shown on Figure 6 and confirms the good adhesion between the soil and the grout body. The rough soil-grout interface was also visually confirmed. Good adhesion was also observed between the ribbed steel tendon and the grout body. Tensile load in the tendon was efficiently transferred to the grout body and thus to the soil. This is illustrated by the cracking observed in the first 500 mm of the distal section.

Creep tests indicated increasing creep deformations with load level (Figure 7). They also highlighted the risk of significant creep, possibly leading to instability, if the mobilised skin friction is more than about 60% to 70% of the ultimate value (Figure 8).

Figure 9 shows the results of all the pull-out tests, the measured bond strength and the design assumptions. Tendon/grout failure is denoted as "T/G" and the associated arrows show soil-grout bond strength derived by extrapolation. Based on the good performance of the pullout tests, nail spacing (horizontal x vertical) was optimised from 1.5m x 1.2m to 1.9m x 1.5m.

4 INSTRUMENTED CUT SLOPE SECTION

The tensile forces within working nails in the constructed slope were monitored using strain gauges at two cross sections (Chainage 980E and Chainage 940E). Full details of theses trials and the instrumentation used are given by Menkiti and Long (2008). The highest forces occurred at the end of the life of the slope with peak loads of 47 kN (Figure 10). Examination of the peaks in the tensile force profiles suggest that a potential failure surface that would have intersected the diverted motorway was successfully restrained.

Shear stresses mobilised in the instrumented nails have been inferred from the measured tensile forces and shown in Figure 11. The maximum mobilised skin friction during the life of the slope was typically 42 kPa, but was up to 87 kPa in Row 6.

These skin friction values are much lower than the available bond strength by a factor of say 3 to 6 (see Figures 5 and 9). The observed movements of the instrumented fully constructed, partly nailed slope were small (Figure 12). Maximum horizontal displacements were typically about 16 mm, being 0.13% of the slope height. This is within the range of 0.1% (rock) to 0.4% (clay) suggested in Clouterre (1991) and FHWA (2003) for nailed slopes. Measured crest settlement were also small, typically 3 mm or 0.03% of the slope height.

5 OVERALL PERFORMANCE OF SOIL NAILED SLOPES

Some photographs of the soil nailed slope at Chainage 1300E under construction are shown in Figure 12. The photographs show the nails and shotcrete protection for Lift 1 (Figure 12a) nearing completion and work underway for Lift 2 (Figure 12b). It can be seen that the material is very competent and stands unsupported allowing the works to proceed without difficulty. Similar good conditions existed over the majority of the soil nailed section. There were some incidents of local overbreak in the cut slope as can been see in Figure 13. Here this was caused by the blocky and highly fissured Dublin Boulder Clay found in the top 3 m at Chainage 1250E to Chainage 1265E.

5.1 Surface settlements adjacent to soil nailed slope

Measured surface settlements behind the soil nailed slope are shown on Figure 14. The data were obtained using conventional geodetic surveying. Not all points are shown but the data have been chosen to reflect the range of measurements. It can be seen that the movements are very modest and in general do not exceed 3 mm to 4 mm, values which are close to the accuracy of the system used. The settlements on the west side of the excavation were in general less than those on the east. The reason for this is not clear.

In particular the measurements are well within the observational method “amber” and “red” trigger levels which were set at 0.1% H (H = maximum slope height) and 0.15% H respectively.

5.2 Lateral slope movements

The development of lateral movement in the slope is shown for a typical profile (Chainage 1295E) in Figure 15. It can be seen that more or less all of the movement is concentrated in the upper half of the excavation. Most of the movements occurred during the excavation for Lift 1 (3.3 m) and the subsequent nailing of Lift 1 and the excavation for Lift 2 (5.8 m). Subsequently little additional movement occurred until final excavation of 10.9 m was reached.

Selected maximum lateral slope movements (for the same profiles as selected for surface settlement) are shown on Figure 16. A similar pattern, as observed for Chainage 1295E, can be seen for all the profiles. Again movements on the west side of the excavation are less than those on the east.

The measured movements do not exceed 0.25% of the excavation height. This is within the range of 0.1% (rock) to 0.4% (clay) suggested in Clouterre (1991) and FHWA (2003) for nailed slopes.

5.3 Pore water pressures

Pore water pressures in the slope were monitored using vibrating wire piezometers which had been installed in fully grouted boreholes as described by Long et al. (2004). Generally piezometers were installed at 4 m, 8 m and 12 m depth. Some typical piezometer data, for three profiles where the excavation depth was about 11 m, are shown on Figure 17. Initially the piezometer readings correspond to hydrostatic conditions with a ground water table at about 2 m depth. Subsequently the values decrease on excavation due to the release of total stress. The decrease in pore pressure is most considerable for the 12 m piezometer and correspondingly less for the 8 m and 4 m piezometers respectively.

On reaching a minimum value towards the end of the excavation period the values remain sensibly constant for the remainder of the monitoring period and completion of the works. Some modest suction values, up to about -7 kPa, were recorded. Similar depressed and negative values were reported by Menkiti et al. (2004) for a trial excavation at the site. These depressed pore water pressures (and suctions) are a major contributory factor to the stability and good performance of the slopes.

6 GROUT AND SHOTCRETE QUALITY

6.1 Soil nail grout

Soil nail grout comprised a neat cement / sand cement mixture with a required minimum 3-day compressive strength of 15 MPa and a minimum 28-day compressive strength of 40 MPa. The water/cement ratio did not exceed 0.45. Testing was carried out at a frequency of one test series per day.

Test results shown on Figure 18a confirm that the material satisfied the design criteria for strength. A small number of test cubes showed 3-day values less than 15 MPa. However as the average of 4 samples was greater than 18 MPa, and the lowest result exceeded 12 MPa, the material was deemed acceptable. The average measured grout density was 1940 kg/m^3.

6.2 Shotcrete

The shotcrete was required to have a minimum thickness of 75 mm and to be placed on a single layer of A252 mesh. The requirements for the 24-hour characteristic compressive strength and the 28-day strength were 8 MPa and 40 MPa respectively. One series of samples was taken for every 300 m^2 of shotcrete placed. Weepholes were provided through the construction facing to drain water from behind the shotcrete on a nominal 2.0 m centres grid. A face plate and nut was attached to each nail after the application of the shotcrete as shown on Figure 13b.

Measured shotcrete strength data is shown on Figure 18b. Again the progressive increase in strength with time is clear and the material was deemed acceptable. Average measured shotcrete density was about 2300 kg/m^3.

7 DISCUSSION

The significant excess bond strength available in Dublin Boulder Clay highlights the conservative nature of current design methods for temporary works in this soil. These design approaches are largely based on effective stress methods to calculate factored-down bond strength for use in design (Phear et al. 2004; Clouterre 1991;

FHWA 2003). In this project, the adopted design bond stress was increased from 50 kPa to 90 kPa – 110 kPa, based on the site testing (Figure 9). This still gave a significant margin relative to the measured pullout strengths. In comparison, the unfactored bond from effective stress considerations can be seen to be significantly smaller. Thus further refinement of common design methods may be possible for short term nails in Dublin Boulder Clay.

It is noteworthy that the optimisation of soil nailing cost and programme is a complex interaction of many factors. For example, available high bond strength may not be fully utilisable due to limitations on nail spacing by the capacity/ cost of the slope facing. Wider nail spacing also implies larger diameter tendons and thicker face plates. These may be prohibited by high steel prices. Heavy steel bars pose health and safety challenges (manual handling). Close interaction between the designer and the contractor is needed for effective optimisation.

Independent experience with continuous flight auger (cfa) piles in Dublin Boulder Clay supports the observation that a high bond strength or shaft friction is available in this soil and that there is a margin for further optimisation of current design methods. This is illustrated by tests on 450 mm to 762 mm diameter instrumented cfa piles (Gavin et al. 2008) in which the mobilised pile shaft friction was as high as 225 kPa – 280 kPa, even though the piles had not been taken to failure. In this case history, mobilised pile shaft friction was about 75% of the undrained strength of the clay. This is comparable to (but less than) the equivalent value measured in the reported soil nail pullout tests.

It should be noted that the results and experience presented in this paper should be carefully considered before extrapolation to permanent nails with much longer design life. Phear et al. (2004) argue that a high factor should be used to reduce the measured bond strength of cohesive soils to derive the design pullout resistance. The reasons they give are that: (i) in the long term soil suctions induced by slope excavation dissipate leading to lower strengths; (ii) the pullout tests themselves may generate local suctions that may cause overestimation of soil-grout bond in the short term; and (iii) the development of shear stress along the nail is progressive and non-uniform.

In dense granular soils, effective stress assessments would underestimate the bond strength, due to constrained dilation around the nail (Phear et al. 2004). A similar mechanism of interlock appears be in operation at the rough, stony soil-grout interface observed in Dublin Boulder Clay nails, constrained in this instance by the very stiff boulder clay. Furthermore, the interface behaviour is ductile. For these reasons, the arguments presented by Phear et al. (2004) for limiting the long term bond of cohesive soils may not fully apply to Dublin Boulder Clay or similar materials.

DPT pullout tests showed that the highest bond values were obtained from clayey soils using air flush. Use of foam to reduce dust emission slightly reduced bond resistance. Lower bond values were derived for granular soils/ lenses, where polymer flush was used to support the hole. Improved drilling practices were developed to complement the observational method applied. Compressed air pressures had to be throttled down to about 3 bars. Routine cleaning of the drill hole was implemented to avoid blockages by running the drill string in and out of the hole. These steps were necessary to avoid pneumatic fracturing of the boulder clay, usually along existing but tightly closed fissures. Site trials showed that such pneumatic fractures caused large horizontal movements, increased soil permeability and increased the potential loss of soil suction.

Results of monitoring of the slopes during and post construction indicate that very modest movements occurred. Lateral movements were confined mostly to the upper two lifts of soil nailing and did not exceed 0.25% of the slope height. Vertical settlements were more or less negligible. Monitoring of piezometers, installed in fully grouted boreholes, showed that a significant reason for the very good stability of the slopes was the depressed pore pressures caused by the excavation induced total stress relief. These depressed pore pressures (and occasional suctions) remained more or less constant for the duration of the project confirming the very low permeability of the Dublin Boulder Clay.

8 CONCLUSION

For temporary soil nails in Dublin Boulder Clay high bond strengths have been measured relative to values specified in common design methods. Monitored performance of instrumented piles confirm this excess capacity. For temporary soil nails in this soil, optimised design may be justified by pullout tests, and a significant population of test data is reported here. Some data is also presented on creep behaviour.

For permanent nails with a long design life, greater conservatism is needed as other important factors (discussed here) come into play.

The development of depressed pore pressures in the slope due to stress relief was a very significant factor in the very good behaviour of the slopes, as can be seen in the photograph of the scheme nearing completion on Figure 19.

ACKNOWLEDGEMENT

The data presented in this paper was obtained for a large infrastructure project. It therefore represents the culmination of effort from large number of people, all of whom we recognised and thank.

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Figures for paper by Menkiti et al on Soil Nailing in Dublin Boulder Clay

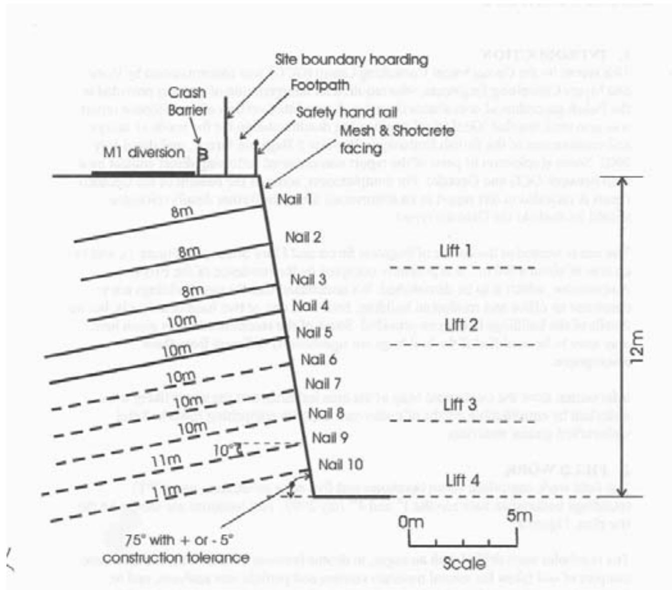

Figure 1. Observational method for soil nailed slope at DPT. Support used: 75 mm to 100 mm thick C40 mesh reinforced shotcrete facing and 100 mm diameter soil nails with 20 mm to 32 mm diameter steel tendons.

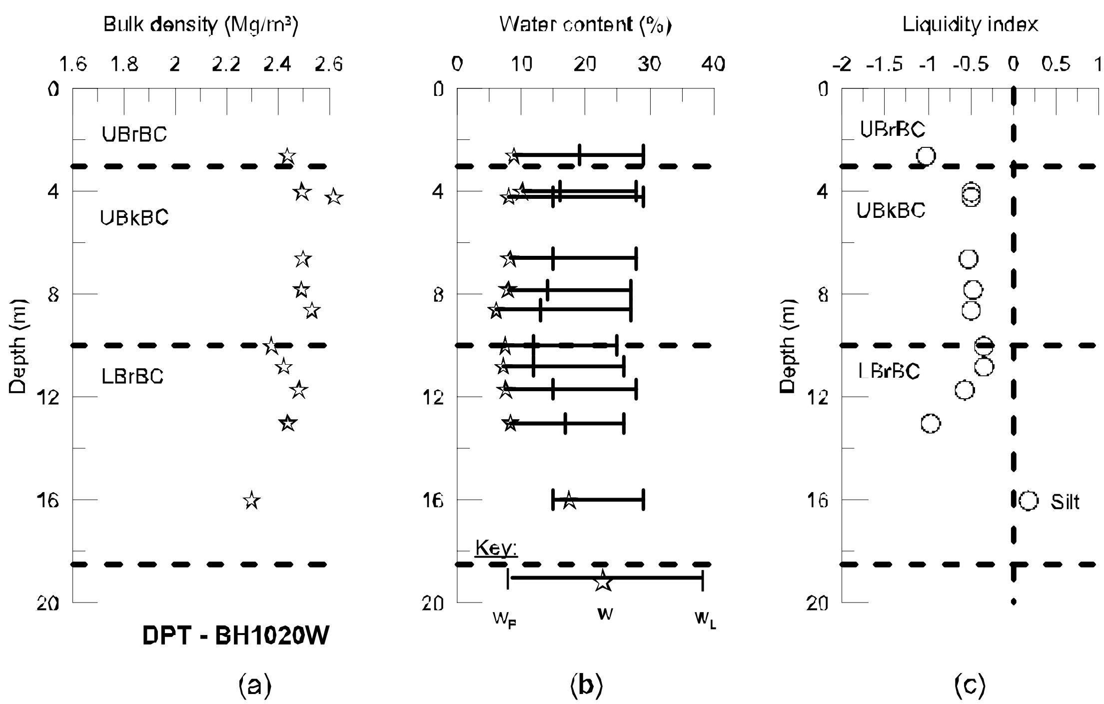

Figure 2. (a) bulk density, (b) water content and (c) liquidity index for DPT – BH1020W

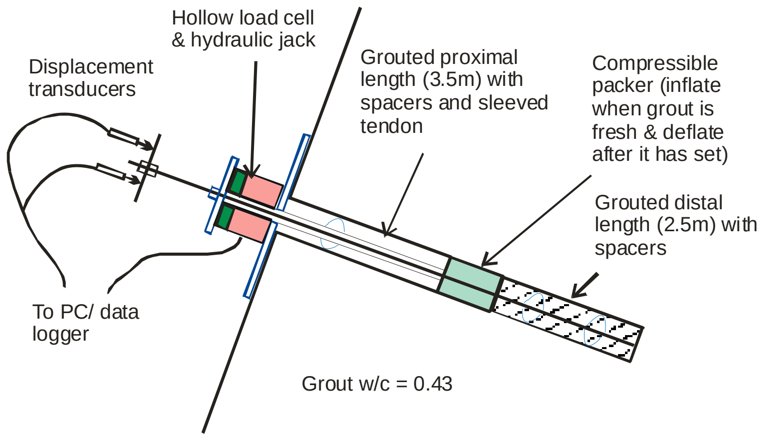

Figure 3. Set-up for sacrificial soil nail tests.

Figure 4. Borehole (diameter 100 mm) before nail installation.

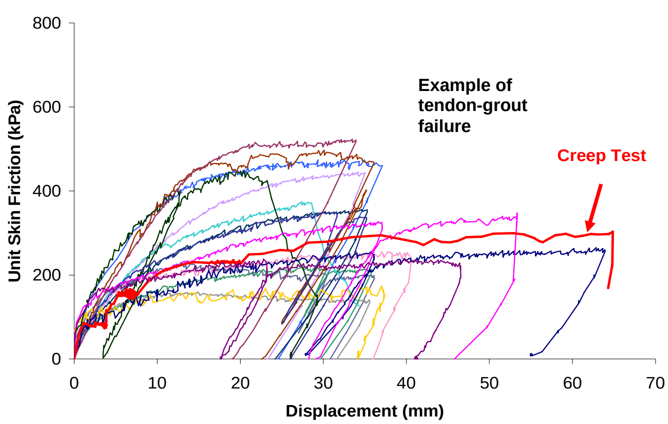

Figure 5. Measured unit skin friction (soil-grout interface).

Figure 6. Photograph of exhumed test nail.

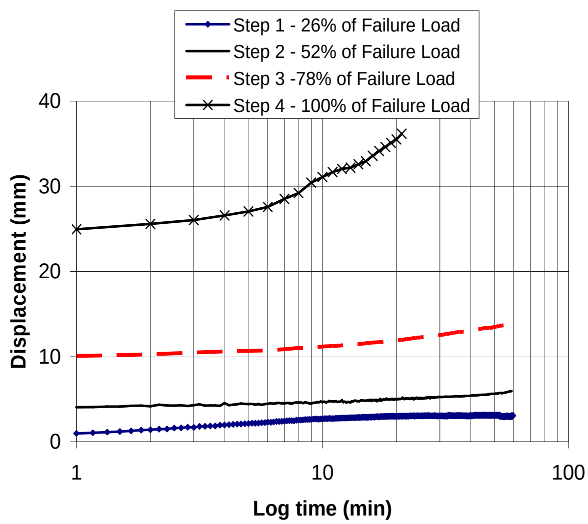

Figure 7. Typical creep test data – displacement versus time for four constant load stages (Nail 37, Lift 1).

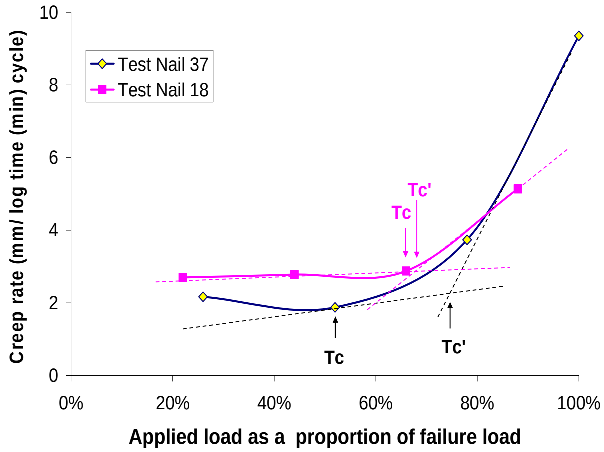

Figure 8. Example creep tests: creep rate versus applied load (as percentage of failure load) Nail 37, Lift 1 & Nail 18, Lift 4.

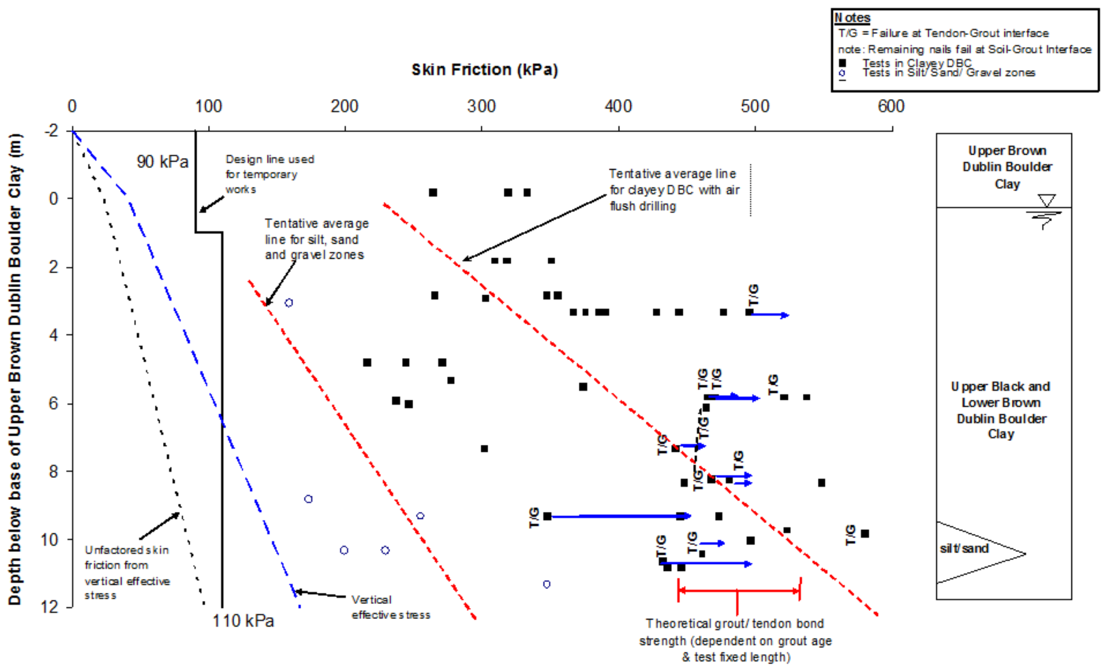

Figure 9. Results of pull-out tests: measured bond strength, design assumptions & ground conditions. Tendon/grout failure denoted as “T/G”; associated arrows show soil-grout bond strength derived by extrapolation.

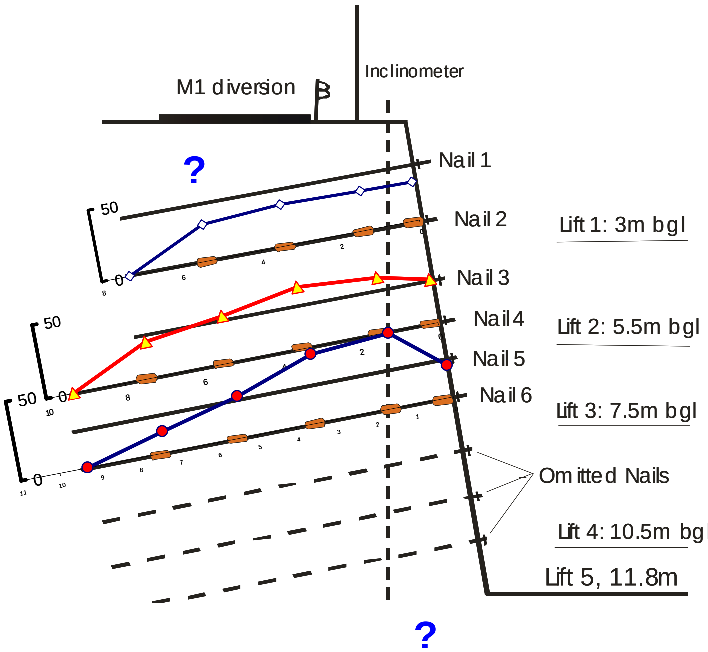

Figure 10. Soil nail tension (kN) at strain gauge locations and potential slip surface that was restrained

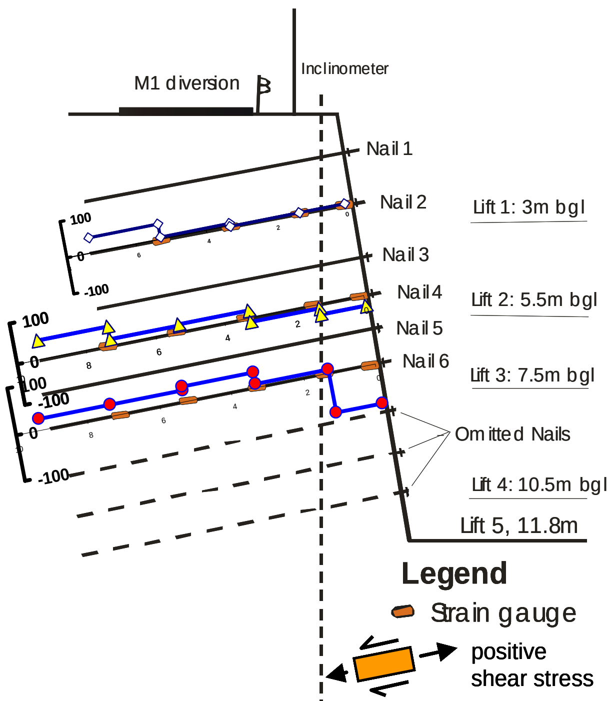

Figure 11. Soil-grout shear stress (kPa, averaged between strain gauge locations).

Figure 12. Soil nailed slope under construction at Chainage 1300E, (a) Lift 1 and (b) Lift 2

Figure 13. Blocky and highly fissured Dublin Boulder Clay found in the top 3m at Chainage 1250E to Chainage 1265E. This led to local overbreak in cut slope

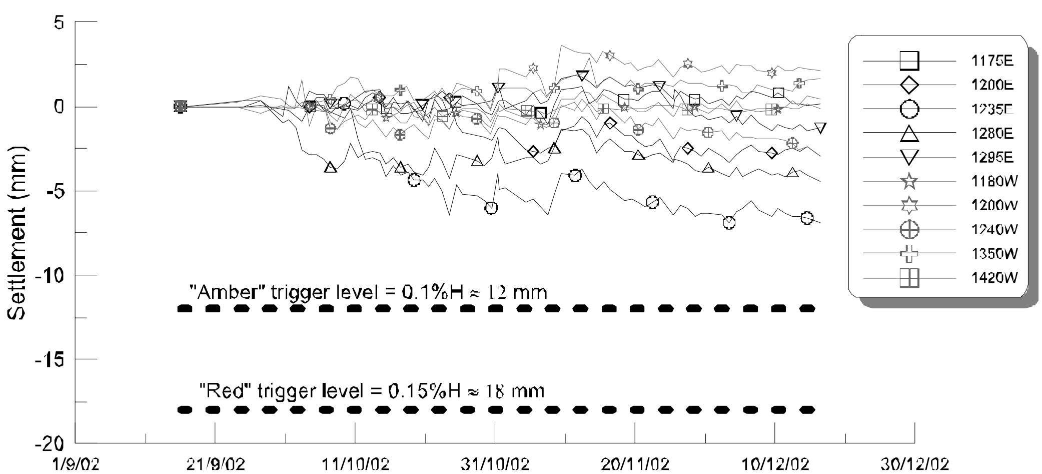

Figure 14. Surface settlements behind soil nailed slopes

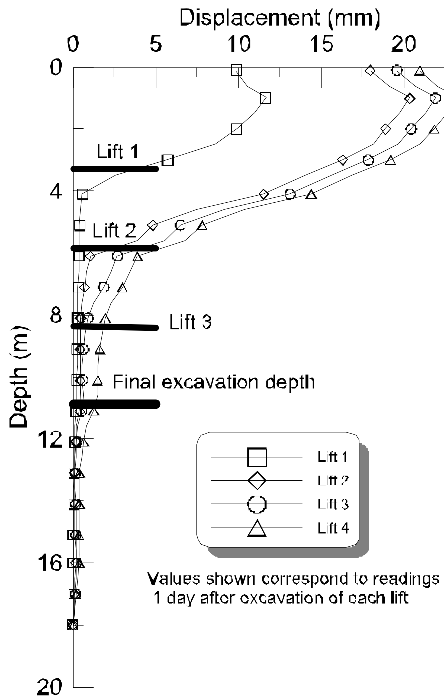

Figure 15. Lateral movement – Chainage 1295E

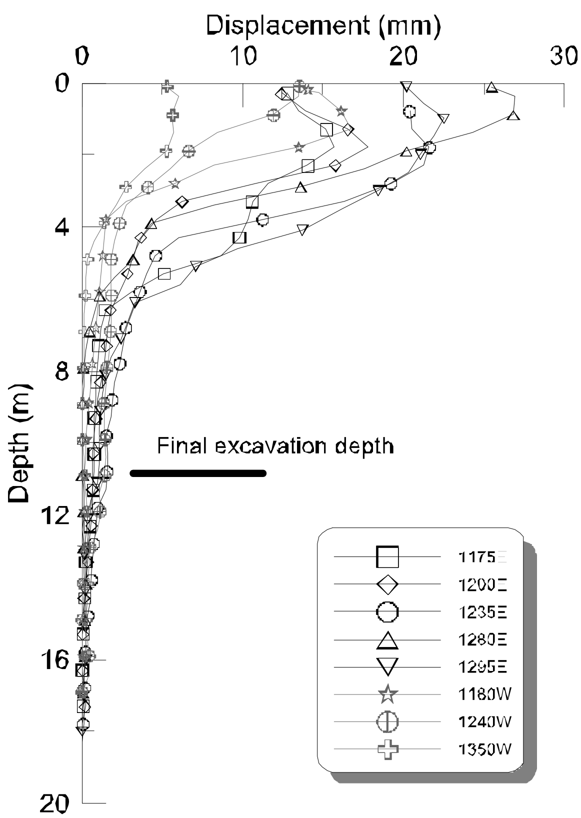

Figure 16. Selected maximum lateral movements

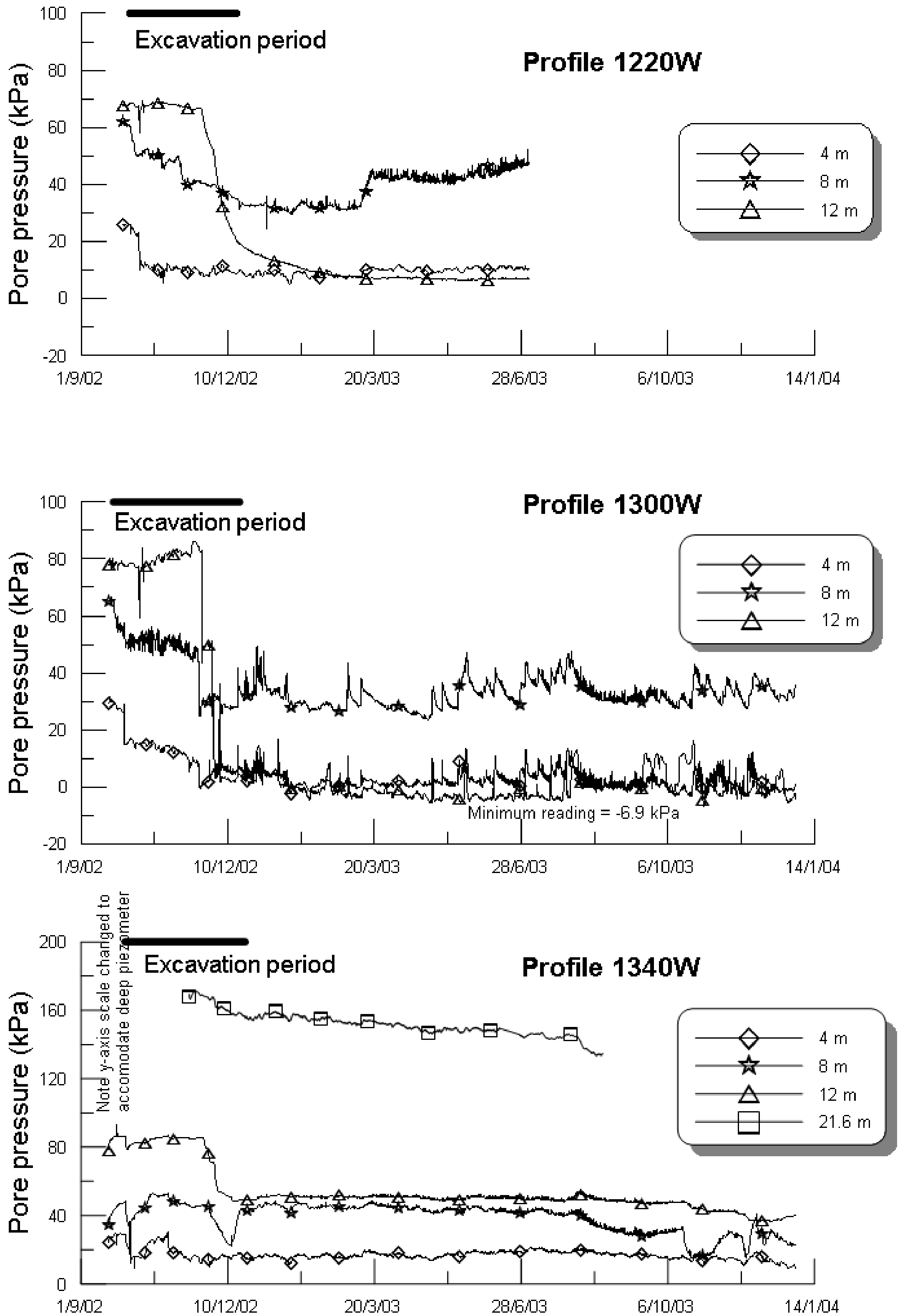

Figure 17. Selected piezometer readings

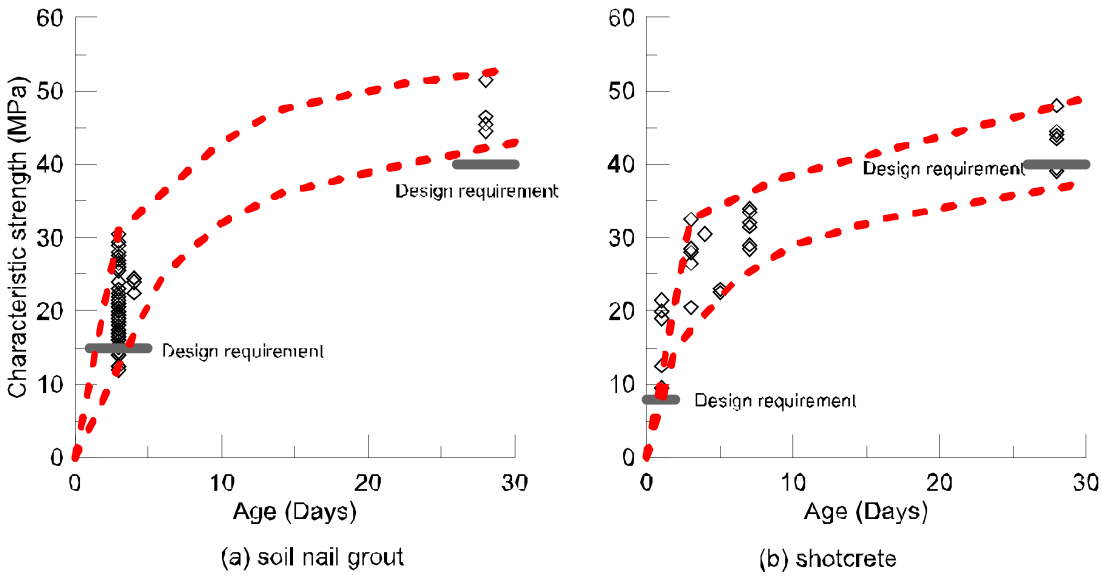

Figure 18. Measured characteristic strength of (a) soil nail grout and (b) shotcrete

Figure 19. Scheme nearing completion

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