



Research Repository UCD

Title	Performance of soil nails in Dublin glacial till
Authors(s)	Menkiti, Christopher O., Long, Michael (Michael M.)
Publication date	2008-12
Publication information	Menkiti, Christopher O., and Michael (Michael M.) Long. "Performance of Soil Nails in Dublin Glacial Till." NRC Research Press, December 2008. https://doi.org/10.1139/T08-084 .
Publisher	NRC Research Press
Item record/more information	http://hdl.handle.net/10197/3074
Publisher's version (DOI)	10.1139/T08-084

Downloaded 2025-12-04 23:02:32

The UCD community has made this article openly available. Please share how this access benefits you. Your story matters! (@ucd_oa)



© Some rights reserved. For more information

Journal: Canadian Geotechnical Journal

Title of paper: Performance of soil nails in Dublin glacial till

Names of authors: Christopher O. Menkiti and Michael Long

Position / affiliation of authors: Associate Director, Geotechnical Consulting Group, 1A

Queensberry Place, London SW7 2 DL, UK (C.O.Menkiti@gcg.co.uk)

Senior Lecturer, School of Architecture, Landscape and Civil Engineering, University College

Dublin, Newstead Building, Belfield, Dublin 4, Ireland (Mike.Long@ucd.ie)

Corresponding author contact details: Dr. Michael Long, Department of Civil Engineering,

University College Dublin (UCD), Earlsfort Terrace, Dublin 2, Ireland.

Phone: +353-1-7163221

Fax: +353-1-7163297

e-mail: Mike.Long@ucd.ie

Date paper originally written: July 2006

Date paper modified: June 2008

Abstract:

Soil nailing is being used in many projects in glacial tills in Ireland, particularly to provide temporary support to steep slopes. Little design guidance is available for such materials and it is known that application of design procedures developed for other material is conservative. Detailed nail instrumentation and field monitoring during large scale soil nailing works for the Dublin Port tunnel project has been undertaken. It was found that the short-term behaviour of nails was the reverse of that assumed in current design methods. Most load was induced due to drilling and nailing the lift immediately below the nail being monitored, rather than due to excavation induced stress relief. The highest forces were developed in the upper nails, where the largest ground movements occur. This is the reverse of most current design methods where the highest soil-nail bond is assigned to the deepest nails. It would seem that the observed short-term, pre-failure behaviour of nailed slope is governed more by the deformation pattern of the slope rather than by large scale development of failed wedges. Current design procedures should be reviewed. Despite this the trial confirmed that the currently used procedures are highly conservative for Dublin glacial till.

Key words: soil nailing, steep slopes, glacial till, suction, design procedures

Introduction

The Dublin Port Tunnel (DPT) provides a link from the motorway system north of Dublin to the port area. The central section comprises twin bored tunnels, driven by tunnel boring machines. The shallower sections of the tunnels at either end, where the invert level is less than about 25 m below ground level, were constructed using cut-and-cover methods. The sides of the cut were generally supported by propped diaphragm walls or bored pile walls. However, in the northern section, where the excavation depth was 12m deep or less, ground conditions were such as to suggest cut slopes supported by soil nailing.

For the soil nailed section, there were very significant constraints on the design. Construction was confined to a narrow corridor along the line of the existing M1 Motorway. The motorway carriageways had to be reduced to one lane for each direction and diverted to either side of the cut. Even with this configuration and using steep sided excavations (up to 80°) the carriageways were only 2m from the slope crest (Figure 1). Additionally, no disruption of the motorway, or damage to adjacent properties, could be countenanced and there were tight cost and programme constraints. However the temporary works design life was relatively short at about 6 months.

At the time of design (2000 – 2002), soil nailing design was still in its infancy. In particular little experience existed on the design of soil nails in Dublin glacial till (known locally as Dublin Boulder Clay, DBC). A case history had been reported by Pedley (2000) but there was no specific design manual or advice note for Irish conditions or indeed for glacial tills.

Johnson et al. (2002) have reported on the use of soil nails in glacial till for a modest 2 m high slope needed for a highway widening project in the UK. A similar solution was detailed by Nicholson and Tse (1999) for a cut slope in glacial till at Tinny Bank near Moffatt on the M6 motorway between Glasgow and Carlisle. Unwin (2001) also reported on some experience of

soil nailing in glacial till in northeast England. More recently, however, guidance on UK practice was published by Phear et al. (2004)

At the time of the DPT design, local experience in Ireland (e.g. Pedley, 2000) suggested that conservatism existed when foreign codes were adopted. As a large soil nailing project in DBC was ongoing, an opportunity existed for a research project involving instrumentation of a number of the nails. Detailed records of construction, geology, instrumentation and design performance were already being kept by the design team for design verification purposes, and these could be drawn upon for the research work.

The research had the following aims:

- To instrument some of the soil nails in order to evaluate their performance as structural units.
- Using this and other complementary project data to study the performance of the whole slope.
- As there was an increased likelihood of future use of soil nails in DBC, given their flexibility and cost savings over retaining walls, the research results could allow design improvements to provide commercial advantage in future works involving DBC.

Ground conditions

Detailed descriptions of the ground conditions and the properties of DBC are beyond the scope of this paper but are available in other publications. The reader is referred to Skipper et al. (2005) who describe the background geology and to Long and Menkiti (2007) for a discussion on the engineering properties of the soil. Menkiti et al. (2004) describe a full-scale instrumented trial excavation at the site.

The ground conditions encountered are summarised on Table 1 and on Figure 2. Conditions are relatively uniform and are characterised by low moisture content, high density,

low plasticity, low permeability and very high stiffness and strength. Ground water pressure is hydrostatic beneath a water table at about 2m depth. Average values of other pertinent soil properties are summarised on Table 2. It is thought that the coefficient of earth pressure at rest (K_0) is of the order of 1.0. It seems likely that the material exhibits a curved failure surface with high values of ϕ' at low effective stress.

Design philosophy

Temporary support for the cut slopes was only required for a period of 3 to 6 months, during which the reinforced concrete tunnel was built and the excavation backfilled. Excavations would largely be in boulder clays, which were known locally to stand at steep slope angles in natural and man-made slopes for considerable periods without support (Long et al. 2003). Advantage could be taken of this short-term stability to provide a temporary works design solution.

However investigations of the boulder clays had indicated that, while they have relatively high effective friction angles for fine-grained materials, there is negligible effective cohesion which could be relied on in design. A slope 12m high inclined at 70° - 80° , with equilibrium pore pressures consistent with a ground water table close to the ground surface, would require a substantial installation of soil nails to maintain stability under long-term conditions. Clearly the observed stability of steep slopes could only be explained by the presence of significant suctions, created in the slopes during excavation and maintained for long periods by the very low permeability of the boulder clay (Long et al. 2004).

An “observational approach” design solution evolved (Milligan et al. 2006). The default design was applied for lengths of the cut which would be required by the construction programme to remain open for periods in excess of 26 weeks. The default design was also applied where the geology was problematic, usually due to permeable layers in the lower part

of the slope (which would allow suctions to dissipate quickly and thus cause instability). Elsewhere, the lower rows of nails were omitted based on criteria established from results of the trial excavation and finite element analyses. In all cases detailed geological logging of excavation faces was undertaken and the performance of the slope was monitored, so that remedial action could be taken from a pre-determined suite of measures, if necessary. More details are given by Long et al. (2003) and Milligan et al. (2006).

Mostly for safety reasons, it was decided that the top half of the slope would always be nailed, i.e. the top 5 rows of nails would always be installed.

Soil nail design

As a fall-back position for when circumstances required, default designs were developed, based on long-term soil strengths. The basic design was for a 12 m high slope at 80° (for a theoretical slope at 75° with construction tolerance of 5°, see Figure 3. Soil parameters used in the design are summarised on Table 3. Partial factors of 1.1 were applied to give design values; the unit weight was only reduced when calculating vertical stresses for the assessment of nail pull-out resistance. A surcharge of 22.5 kPa was included in the design as required by the client's specification.

Nail design was in general to Advice Note HA 68/94 (UK Department of Transport, 1994 and discussed in detail by Love and Milligan, 1995) with reference to Clouterre (1991) and FHWA (1998). Nails were designed to have an inclination of 10° and to be installed in 100mm diameter holes. Pull-out resistance (bond) was calculated from the overburden pressure using the method given in HA68/94, but with a minimum bond stress of 50 kPa in the upper parts of the slope where the calculated overburden pressure is low. Following extensive testing of nails during construction, the minimum bond strength was raised for later stages of design, first to 85 kPa, and then to 90 kPa for the top two rows of nails and 110 kPa for the remainder. Figure 4 shows this design profile together with measured bond strength

from pullout tests on sacrificial test nails. The test nails were drilled in the same manner as the majority of the working nails using air flush mixed with the minimum amount of foam necessary to control dust generation. These design bond values of 90-110 kPa provided a factor of safety of 2.5 on measured ultimate bond resistance. These design bond values can be seen to be much higher than the bond strength calculated using standard design methods indicating a significant level of conservatism for temporary works applications (Figure 4). The high pullout capacity and favourable monitored slope performance allowed the design nail spacing to be optimised from 1.5 m to 1.7 m and on to 1.9 m. However, for the 1.9m spacing, the shotcrete facing thickness was increased from 75 mm to 110 mm.

Excavation was typically carried out in four stages, to depths of 3 m, 6 m, 9 m and 12 m (Figure 3). Slope stability at each excavation stage was considered, using limit equilibrium methods and checked for 2-part wedge mechanisms using Geosolve SLOPE program (Version 9R) as well as for circular failure surfaces using the SLOPE/W program (Version 4.24). In addition to the partial factors on soil parameter values, a partial factor of 1.3 was applied to surcharge loads and nail pull-out resistance. With these partial factors, an overall factor of safety of 1.2 was sought for circular failure surfaces and 1.0 from the bi-planar wedge failure mechanism. This is because the two-part wedge analysis is known to be conservative. With this approach, both methods were generally found to give consistent results. Some typical analysis output is given in Figure 5 to demonstrate the failure mechanism assumed for comparison with the actual nail behaviour described later.

Layout of instrumented soil nails

Chainage 980E – Good ground conditions

At Chainage 980E “good” ground conditions were present (i.e. UBrBC and UBkBC as expected with no significant granular lenses) and therefore the lower 3 rows of nails were omitted (Figure 6). Instruments were installed on Nails 2, 4 and 6. The upper nail comprised

an 8 m long 20 mm diameter high tensile steel bar, the lower two were 10 m long, 25 mm diameter bars. Pairs of strain gauges were installed on opposite sides of the bar. Installation locations were at 0.3m from the excavated slope face, 2m from the excavated slope face and thereafter at 2m intervals along the nail length. The instrumented nails were orientated so that the strain gauges were at 12 and 6 o'clock positions, in order to measure vertical bending. The strain gauges were equipped with thermistors which allowed measurement of temperature at each instrument location.

Geokon VK-4100/4150 strain gauges were used with a gauge length of 51mm (2in.), a range of 3000 microstrain and a sensitivity of 1 microstrain. The gauges are designed for use on steel structures or reinforcement bars. They have an accuracy of $\pm 0.1\%$ of full scale, a non-linearity less than 0.5% and are designed to operate in a temperature range -20°C to $+80^{\circ}\text{C}$.

Additional instrumentation at Ch 980E comprised piezometers at 4 m, 8 m and 12 m depth as well as an inclinometer, surface mounted prisms for detecting slope face movement and settlement markers at the slope crest.

Chainage 940E – poor ground conditions

At this location, as can be seen on Figure 7, a granular layer was proven at just below 8 m and therefore the default design was applied and all nails installed. Based on the experience at Ch. 980E, as will be described below, and due to cost constraints, only Nail 2 was instrumented. The installed instrumentation was similar to that for Chainage 980E Nail 2. The additional instrumentation at Ch. 940E was also the same as at Ch. 980E.

Preparation and installation of instrumented nails

Nails at Chainage 980E

Details of the nail instrumentation procedure are shown in Figure 8 and the following procedure was adopted for attachment of the strain gauges to the nails:

- Log face and mark out nail location.

- Select nail tendon, mark out strain gauge locations and grind down a smooth bed at each location.
- Spot weld vibrating wire gauges to top and bottom of nail, Figure 8a.
- Waterproof gauges using “Skotchkote” electrical sealant.
- Install pick-ups and protection, Figure 8b.
- Take zero readings of strain gauges with nail in a flat unstressed configuration.
- Attach post inflatable packer, Figure 8c. (The packer was formed from an inner tube of a motorcycle tyre and its purpose was: (i) to protect the gauge and (ii) to allow the grout column to be broken out post installation so as not to influence the gauge readings.)
- Check gauges at each stage.
- Protect packers with geotextile (to avoid puncture during tendon insertion).
- Drill hole using rotary techniques with rock roller or blade bits (note borehole needs no casing as soil is self supporting).
- Grout hole.
- Install nail tendon with the correct orientation and with the strain gauges at the correct depth.
- Fully inflate packer and displace fluid grout at instrumented locations.
- Protect excavated face as shown in Figure 8d with mesh and shotcrete.
- Install survey prism to nail head.
- Hook up to data logger.

Initial soil nailing for the project was with a compressor delivering 12 bar air pressure. It was quickly observed, from the detailed monitoring conducted, that the drilling process induced large lateral movements of the slope by a process of pneumatic fracturing. Lateral movements of 10 mm to 15 mm per lift were observed due to creation of new sub-vertical

discontinuities or opening up of pre-existing but tightly closed fissures. This was confirmed by observed interconnectivity between nail holes, with grout being blown out of nail holes up to 8m apart. This drilling process was unsatisfactory as it damaged the soil fabric, increased the mass permeability of the ground and possibly destroyed the excavation induced soil suctions in the ground. However, drilling with air flush was preferred in order to maximise skin friction. Trials were therefore carried out early on in the project and this problem was avoided by using a 7.5 bar compressor or the 12 bar compressor at a reduced air flow setting, and routine cleaning out the drill hole during drilling. (This latter point is important as it prevented blockage around the drill string, a situation that would deliver the full air pressure to the soil mass.) This revised drilling procedure was used for most of the soil nailing (including at the instrumented nail sections). The revised drilling method reduced drilling-induced movements to a maximum of 3 mm to 4 mm of horizontal movement for the first lift and even smaller movements for deeper lifts.

Nail at Chainage 940E

At Chainage 940E a similar procedure to that used at CH 980E was adopted for installing the instrumented nail, except that some changes were made to the packer system. A disadvantage of the post installation packers was the physical difficulty in dealing with and protecting the many wires and tubes that were necessary. At Chainage 940E a pre inflated packer was used. This was actually a child's water safety arm band. The packer was installed with non-return valve, inflated to just below the borehole diameter and was then subsequently protected by a geotextile as before. This system proved much easier to install because of the reduced amount of tubing.

Results of monitoring

Chainage 980E / Row 2

Average tensile and bending strains induced in Nail 2 at Chainage 980E are shown on Figure 9 for a period of some eight months. Note gauge pair A is towards the end of the nail and pair D towards the front (Figure 6). It can be seen that the tensile strains are significant but that the bending strains are relatively small. This confirms that the nails are acting in tension as designed. Following the final soil excavation at this location on the 6 June 2003 both the axial strains and the bending strains remained relatively constant.

Average tensile loads induced in Nail 2 are shown on Figure 10 with an enlarged x axis to focus on load changes during the construction process. Immediately following nail installation, very small compressive loads are induced at the head of the nail due to the weight of the shotcrete facing which sits on the nail. Thereafter, the effect of the next soil excavation (Lift 2) can be observed to induce relatively small tensions. However the effect of installing the nails in this lift on 11 April 2003 was considerable. It seems that the effects of drilling, using compressed air, even with the revised drilling method has caused ground movement and loading of already installed nails in the lift above. (The situation on site on this day is shown on Figure 11.) From Figure 10, it can be observed that Lift 3 has some effect on Nail 2 but Lift 4 and Lift 5 have relatively minor influence. The revised drilling procedure was observed to induce tensions in the outermost 5 m to 6 m of the row 2 nails during drilling for Lift 2 (Locations D, C & B). Induced tensions were nominally about 30 kN. The nominal tension was computed by applying the measured strain to the nominal cross section of the soil nail. However, at the strain gauge locations, the bar diameter is slightly reduced as the strain gauges were bonded onto flat surf on the bars to promote good bonding (see Figure 8a). Correction for the reduced bar cross-section would imply that the actual bar tension is 12.5%

lower for the 20mm diameter row 2 nails and 6.1% lower for the 25mm Row 4 and Row 6 nails. Only nominal tensile forces are quoted in this paper.

The development of shear stress along Nail 2 is shown on Figure 12. Again it can be seen that most of the skin friction is developed during nailing for Lift 2 and the changes in skin friction are much smaller for the other construction stages. Most of the skin friction is developed towards the distal end of the nail, where the tension in the bar is changing most rapidly with distance along the bar. Maximum mobilised shear stress is of the order of 60 kPa, which is significantly lower than the minimum design bond stress of 90 kPa and measured bond values of 250 to 350 kPa at this level (Figure 4).

Load in all nails

Average nail load for all instrumented nails, at the strain gauge pair closest to the shotcrete face, is shown on Figure 13. A similar pattern to that developed above for Chainage 980E, Nail 2 can be observed, i.e.:

- excavation of next lift has only minor effect,
- much of the load is developed at an early stage during nailing works for the subsequent lift,
- Developed load generally remains constant during the monitoring period.
- The developed load generally decreases with increasing depth from Nail 2 to Nail 6.

The load induced in Nail 2 at Chainage 940E is slightly lower than that at Chainage 980E. A maximum load of 40 kN was measured in 980E R2, compared to 35kN at 940E R2. This may reflect variability in the ground and the construction process, but may also be due to more rows of nails being present at Ch 940 and the subsequent greater available resistance capacity.

Lateral slope displacement

Horizontal slope movement at Chainage 980E and 940E is shown on Figures 14a and 14b respectively. For Chainage 980E it can be seen that a maximum movement of about 8 mm occurred due to excavation induced stress relief following excavation to about 5.5 m for lifts 1 and 2 and nailing Lift 1. During nailing of Lift 2 further significant movements of about 3 mm occur. This is consistent with the period during which most load was induced in the nails. Subsequently during lifts 3, 4 and 5 some further movement, up to a maximum of 3 mm occurred. Following the installation of nails in Lift 1 the slope behaves like a “propped cantilever” retaining wall with the nails restraining the movements towards the top of the slope. No movement occurs below final excavation level.

A similar pattern is again evident for Chainage 940E. Here the slope behaviour resembles a “cantilever” retaining wall until the nails in Lift 3 are installed. Maximum movement is again about 16 mm and no movement occurs below excavation level.

Loads on the shotcrete facing

Loading on the shotcrete facing has been inferred from the nail head loads assuming that the shotcrete is permeable relative to the clay. (This assumption is consistent with observations on this site, the presence of numerous construction joints and shotcrete permeability measurements at other sites.) The loading on the shotcrete facing is presented as equivalent earth pressure, based on the as-built nail spacing. The earth pressure has been calculated from an effective area around each nail as shown in Table 4. It is assumed that effective area around each nail extends from that nail to a point midway between it and the neighbouring nails. No nails exist below Nail 6 at Ch 980E (Figure 6). Therefore as a first approximation, the equivalent area for this nail has taken to extend down to the location where Nail 7 would have been installed. The earth pressures presented in Figure 15 are

similar to the active pressures in DBC (for which a K_0 value of is thought to be about 1.0 or higher, e.g. Long and Menkiti (2007)

It appears that slope movements of only 16mm or 0.13% of the total slope height are sufficient to reduce the short-term pressures on the shotcrete facing towards active values.

Temperature

The thermistors incorporated in the strain gauges showed some changes change in temperature during the monitoring period. The largest temperature changes occurred at the strain gauges nearest the nail head, which recorded temperature ranging from 6 to 20°C, primarily reflecting small daily changes superimposed on a more gradual seasonal change. The deepest strain gauges experienced the most muted temperature changes recording temperatures of 10 to 14°C which reflected only a seasonal change (daily changes being fully muted at this depth). Temperature changes only have a second order effect on the measured tendon strains because the gauges are temperature compensated for steel. Close scrutiny of the daily temperature changes of the shallowest gauges indicates a temperature effect of about 3.5 microstrains per °C, compared to a coefficient of thermal expansion of steel of 12.2 microstrains per °C.

Crest settlement

Crest settlements, as measured by conventional geodetic surveying, were small and generally less than 3 mm.

Pore pressure

Pore pressure development in the slopes was similar to that described by Long et al. (2004). On excavation, pore pressures reduced from the in situ hydrostatic values by small amounts due to stress relief. For example at Chainage 940E, pore pressure at 4 m, 8 m and 12 m reduced by 25 kPa to 35 kPa (r_u reduced by 0.3 to 0.06), i.e. pore pressures remained

slightly positive. The situation is similar at Ch 980E, where pore pressures reduced by 18kPa to 45kPa (i.e. r_u reduced by 0.28 to 0.13).

Comparison with design

A comparison between ultimate available nail tendon capacity and maximum mobilised force is shown on Figure 16. It can be seen that the mobilised capacity of the nails is some 25% to 30% of the available ultimate capacity. This suggests that, although the overall nail / slope behaviour is not as assumed in the calculations, the actual design is conservative. The same conclusion is true for nail bond capacity as shown in Figures 4 and 12. The measured short term bond stress in DBC is very high, allowing design optimisation for temporary works problems.

Link between short term and long term behaviour

Little research has been done on the link between short term and long term behaviour of soil nailed slopes in clays. As reported by Phear et al. (2004), the UK Transport Research Laboratory attempted to confirm the relationship between short term tests and longer term values by carrying out long-term pull-out tests to failure (Johnson et al., 2002). In these tests the load was raised incrementally every three to four weeks. The total test periods at their two sites were eight and twenty months respectively. These sites were in low to medium plasticity glacial till and high plasticity Oxford Clay respectively. The results were inconclusive as twenty months may not have been long enough for the pore pressures to reach equilibrium.

Long term behaviour in DBC is also uncertain and requires further research to see how much of the short term strength can be relied upon in the long term. Consideration would have to be given to issues such as pore pressure dissipation, durability and creep. Figure 17 illustrates the behaviour of a cut slope with a uniform distribution of soil nails, which progresses to failure. The pattern of slope deformation at a horizon I-I' close to the slope crest is shown along with the pattern of nail forces. In the pre-failure phase, the maximum

deformations and the maximum nail forces occur at the top of the slope. A similar pattern of behaviour was observed in full scale model tests reported in Clouterre (1991) and in this paper. As the hypothetical slope progresses towards failure, deformations accumulate and localise and the maximum deformation migrates down to a point at some depth. The distribution of nail forces also changes in a similar pattern. If the hypothetical slope is adequately reinforced, then it will not progress to failure and the pattern of slope displacements and nail forces will only evolve to a state part way between the initial pre-failure conditions and the distribution at failure.

The pattern of nail pull-out capacity assumed in most design methods is similar to that at failure, and may be dissimilar to the short term or pre-failure pattern of nail forces. The forces and deformation patterns in many engineering structures are different under working conditions from those assumed at failure. There is nothing wrong with this provided behaviour is ductile and failure does not occur due to brittle behaviour, instability or progressive failure.

The results reported here confirm that in the short term, the construction process dominates with large loads being developed in the upper rows of nails due to drilling / excavation. In the long term the slope will tend towards a drained condition, such that the conventional design assumptions apply, with higher loads developing in the lower nails. The data presented here contributes to an understanding of the short term serviceability performance of temporary soil nailed slopes in relatively impermeable ground and also confirms that both short term and long term should be considered in design.

Conclusions

- 1) The instrumented nail project worked well, with all instruments responding and with no failures. This success was due to close attention to detail.

- 2) Following the final excavation little change occurred in the instrument readings over the monitoring period of about 10 months. This is due to the low permeability of the DBC and the corresponding very slow equalisation of pore pressures.
- 3) Nails act mostly in tension with only limited bending induced.
- 4) The behaviour of nails is the reverse of that assumed in current design methods, i.e.:
 - Most load was induced due to drilling and nailing the lift immediately below the nail rather than due to excavation induced stress relief.
 - The highest forces were developed in the upper nails, whereas the maximum bond capacity is assumed to be available at depth in most design methods.
 - Mobilised earth pressure on the shotcrete facing approaches active values after slope movements of only 0.13% of the slope height. This is consistent with expected behaviour for dense soils which typically achieve active conditions at about 0.1% of slope height.
- 5) Maximum shear stress is developed towards the distal end of the nail.
- 6) Load development in the nails mirror the deformation pattern of the slope.
- 7) It would seem that current design procedures do not reflect these aspects of behaviour. A more sophisticated approach, e.g. by finite element methods, may be considered.
- 8) Despite this, current design methods are still very conservative for DBC. Further design optimisation is possible as experience with using this method of construction is gained.

Acknowledgements

The authors are very grateful to the sponsors of the project namely:

- ◆ Geotechnical Consulting Group, UK
- ◆ University College Dublin
- ◆ Applied Ground Engineering Consultants Limited
- ◆ ARUP Consulting Engineers

- ◆ PJ Edwards Ltd
- ◆ John Barnett & Associates Limited (part of CSA Group)
- ◆ Byrne Looby & Partners

In addition Mowlem-Irishenco JV (main contractor for the Dublin Port Tunnel project) provided men and materials free of charge and ITM - Soils supplied the instrumentation at discounted research rates. The contribution and assistance of the authors, colleagues including Ben Follett, George Milligan and Sam Patterson.

References

- Clouterre (1991). Soil nailing recommendations for designing, calculating, constructing and inspecting earth support systems using soil nailing. Ecole Nationale des Ponts et Chaussées, Paris. English translation, July 1993, US Department of Transportation, FHA, FHWA-SA-93-026.
- FHWA. 1998. Manual for design and construction monitoring of soil nail walls. US Department of Transportation, FHWA, SA-96-069R.
- HA 68/94. 1994. Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques. Design Manual for Roads and Bridges, **4**, 1, 4, HMSO London.
- Jonhson, P. E., Card, G B and Darley, P. 2002. Soil nailing for slopes. Transport Research Laboratory, Report 537.
- Long, M., Menkiti, C.O., Kovacevic, N., Milligan, G.W.E., Coulet, D. and Potts, D.M. 2003. An observational approach to the design of steep sided excavations in Dublin glacial till". *In Proceedings Underground Construction 2003, UC2003*, London, Published by Hemming-Group Ltd., pp. 443 – 454.

- Long, M., Menkiti, C.O. and Follet, B. 2004. Some experience in measuring pore water suctions in Dublin glacial till. *Geotechnical News / Geotechnical Instrumentation News*, **22** (3): 21 – 27.
- Long, M. and Menkiti, C.O. 2007. Geotechnical properties of Dublin boulder clay. *Géotechnique*, **57** (7): 595 - 611.
- Love, J. and Milligan, G. 1995. Reinforced soil and soil nailing for slopes. Advice Note HA 68/94. *Ground Engineering*, **28**, (4): 32 – 36.
- Menkiti, C.O., Long, M., Kovacevic, N., Edmonds, H.E., Milligan, G.W.E. and Potts, D.M. 2004. Trial excavation for cut and cover tunnel construction in glacial till - a case study from Dublin. *In Proceedings Skempton Memorial Conference, Advances in Geotechnical Engineering, Edited by Jardine et al., March, Thomas Telford*, **2**, pp 1090 – 1104.
- Milligan, G.W.E., Menkiti, C.O., Long, M., Kovacevic, N., Paterson, S. R., Coulet, D., and Potts, D.M. 2006. Design and performance of fully and partly nailed slopes in Dublin Boulder Clay. Part 1. Design and analysis. Submitted for publication in *Geotechnique*.
- Nicholson, D.P. and Tse, C-M. 1999. The observational method in ground engineering: principles and applications. CIRIA Report 185, Construction Industry Research and Information Association, London.
- Pedley, M. J. Examples of the use of temporary works soil nailing in the UK and Ireland. 2000. *In Proceedings 25th Annual Meeting and 8th International Conference of the Deep Foundations Institute*, 5th – 7th October, New York, USA, pp. 529-538.
- Phear, A., Drew, C., Ozsoy, B., Wharmby, N. J., Judge, J. and Barley, A. D. 2004. Soil nailing: best practice guidance. CIRIA Funders Report CP105.
- Skipper, J., Follett, B., Menkiti, C., Long, M, Clarke – Hughes, J. 2005. The engineering geology and characterisation of Dublin Boulder Clay. *Quarterly Journal of Engineering Geology and Hydrogeology (QJEGH)*, **38**: 171 – 187.

Unwin, H. 2001. Carbon fibre soil nailing for railway embankments. *In*: Proceedings of the International Conference on Railway Engineering, London.

Tables

Table 1. Summary of ground conditions

Stratum	Thickness (m)	Nature
Topsoil/loess/ made ground	Typically 1.0, maximum 3.0	Variable, predominantly highly permeable coarse gravel with other materials.
Upper Brown Boulder Clay (UBrBC)	Typically 2.0.	Firm to stiff friable slightly sandy clay with occasional cobbles and fine to coarse gravel. Rare thin lenses of grey sandy silt with some medium gravel zones.
Upper Black Boulder Clay (UBkBC)	4 to 10	Very stiff to hard dark grey slightly sandy clay with some gravel and cobbles. Rare fissures, sub-vertical, rough and tightly closed, spacing 0.5-0.75m. Occasional small gravel lenses, mostly forming hydraulically isolated pockets.
Lower Brown Boulder Clay (LBrBC)	5.0 to 9.0	Hard brown silty clay with gravel, cobbles and boulders. More frequent, larger and more complex silt/gravel lenses than UBkBC. Continuous 2m thick layer of silty sand/fine gravel at 10 to 16m depth over a 350m long stretch of the works.
Lower Black Boulder Clay (LBkBC)	Patchy, up to 4.0m thick	Hard gravelly silty clay with an abundance of boulders.
Limestone bedrock	Encountered at 16m depth or below.	Dark grey fine grained limestone interbedded with black shale. Upper few metres often highly weathered

Table 2. Summary of pertinent soil parameters (Long and Menkiti, 2007)

	UBrBC	UBkBC	LBrBC	LBkBC
Clay content (%)	12	15	18	18
s_u (UU) (kPa)	80	220	350	> 250
Permeability, k (m/s)	$\approx 10^{-8}$	$\approx 10^{-10}$	10^{-8} - 10^{-10}	$\approx 10^{-10}$
G_{\max} (MPa)	250 - 1000	1000 - 1200	1000 - 1200	1000 - 1500
ϕ'	36°	38°	38°	38°
c' (kPa)	0	0	0	0

Table 3. Summary of soil nail design parameters

Stratum	γ (kN/m ³)	ϕ'	c' (kN/m ²)	r_u
UBrBC	20	31.5°	0	≤ 0.4
UBkBC	20.5	32.5	0	≤ 0.2

Table 4. As-built nail spacing at the locations of instrumented nails.

Instrumented Nail	As-built spacing at nail location		Assumed
	Vertical Spacing	Horizontal Spacing	effective area
980E R2	1.25m	1.5m	$1.2 \times 1.5 = 1.875 \text{m}^2$
980E R4	0.9m	1.5m	$0.9 \times 1.5 = 1.35 \text{m}^2$
980E R6	1.55* m	$(1.9^{**} \text{m} + 1.5^{**} \text{m})/2$	$1.55 \times 1.7 = 2.635 \text{m}^2$
940E R2	1.35m	1.7m	$1.35 \times 1.7 = 2.295 \text{m}^2$

* There are no nails below nail 980E R6 (see Figure 6). The vertical spacing is taken to extent from the theoretical location of Row 7 to the mid point between Rows 5 and 6.

** A change in horizontal spacing occurred at nail 980E R6. To the right of this nail, a nail spacing of 1.5m was used. To the left of the nail, design optimisation allowed a wider spacing of 1.9m to be used.

Notation

G_{\max} small strain shear modulus

c' effective cohesion

k coefficient of permeability

r_u pore pressure ratio = u / σ_v

u pore pressure

s_u undrained shear strength

K_0 co-efficient of earth pressure at rest = $\sigma'_{h0} / \sigma'_{v0}$

ϕ' effective friction angle

γ bulk unit weight

σ_v total vertical stress

$s_u(\text{UU})$ Undrained strength from unconsolidated undrained triaxial tests

Figure Captions

Fig. No.	Caption	File / Ref.
1	Typical cross section	GWEM
2	Summary of ground conditions	D(Q)/DPT/BasicsII.grf
3	Default design for soil nailed slope	
4	Measured ultimate skin friction versus depth for Dublin Boulder Clay	
5	Typical output from slope stability analyses	
6	Layout of instrumented soil nails at Ch 980E - good ground conditions	
7	Layout of instrumented soil nails at Ch 940E - poor ground conditions	
8	Soil nail installation pictures	D(W)/DPT/nailwithtwogauges.jpg, geokonpickup.jpg, postinflatable.jpg, faceshotcreteetc.jpg
9	Average tensile and bending strains in Nail 980E/Row 2	D(W)/DPT/Row2strains.grf
10	Average tensile loads in Nail 980E/Row 2	D(W)/DPT/Row2loads.grf
11	Situation on site 11 April 03 during nailing of Lift 2 at Chainage 980E	
12	Development of shear stress along Nail 2 Chainage 980E	D(W)/DPT/Row2skinfriction.grf
13	Development of load – all nails	D(W)/DPT/Row24and6loads.grf
14	Lateral slope movement from inclinometers	D(W)/DPT/Inclos980.grf
15	Earth pressures on shotcrete facing inferred from the nail head forces	
16	Comparison of ultimate and mobilised nail tendon capacity	D(W)/DPT/Nailcapacity.grf
17	Deformations and nail forces in a typical cut slope with a uniform distribution of nails	

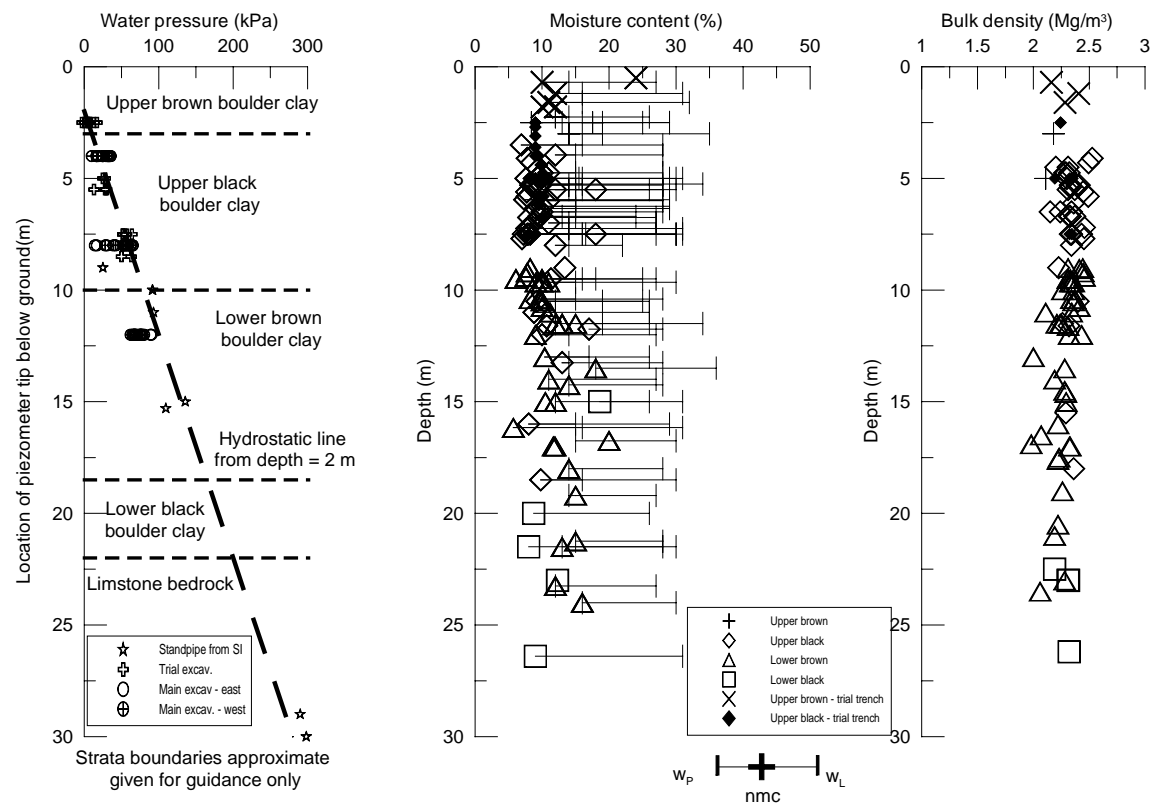


Fig. 3. Default design for soil nailed slope
(nails in rows 6 to 10 or 8 to 10 omitted where allowed by observational approach)

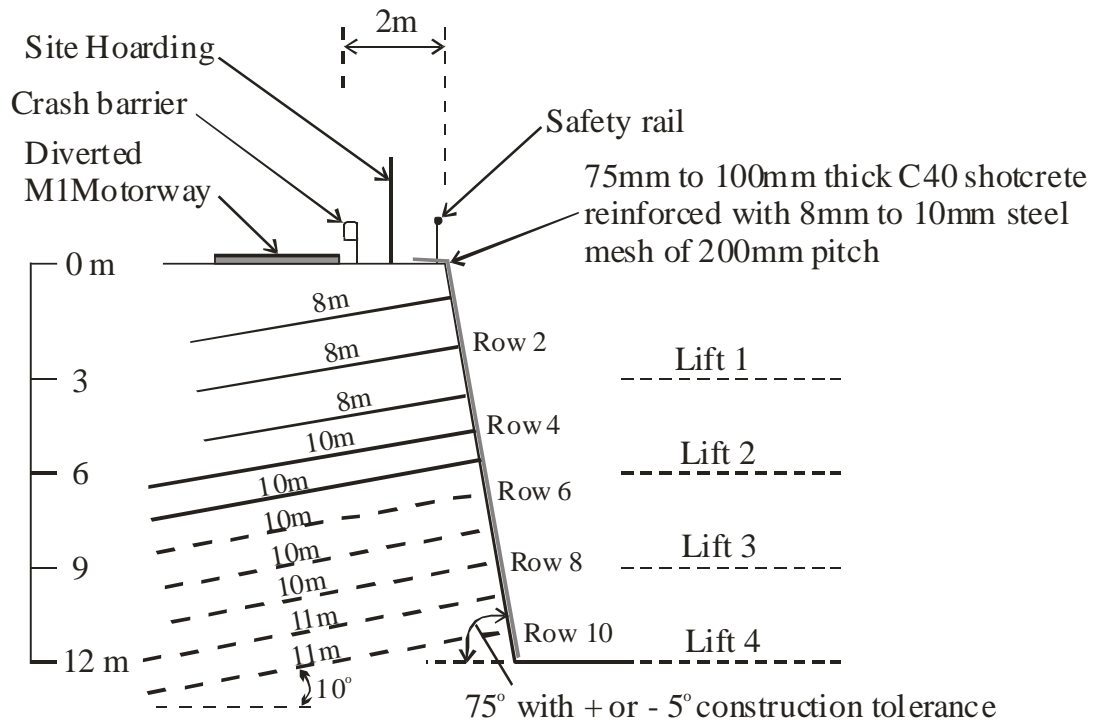


Fig 4. Measured ultimate skin friction versus depth for Dublin Boulder Clay

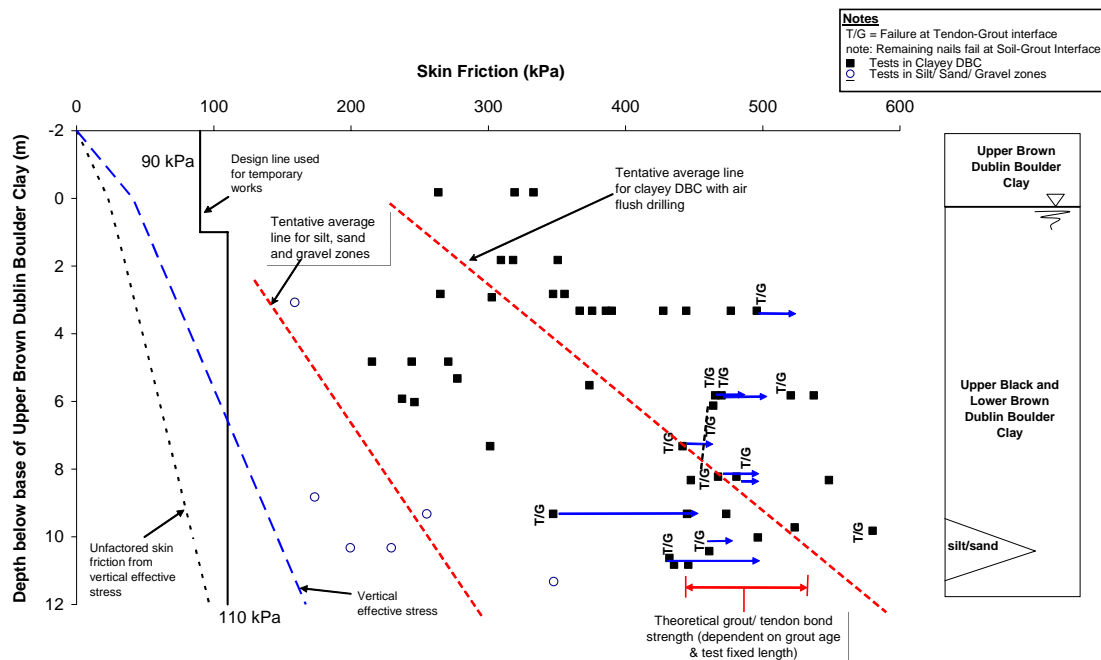


Fig. 5. Typical output from slope stability analyses

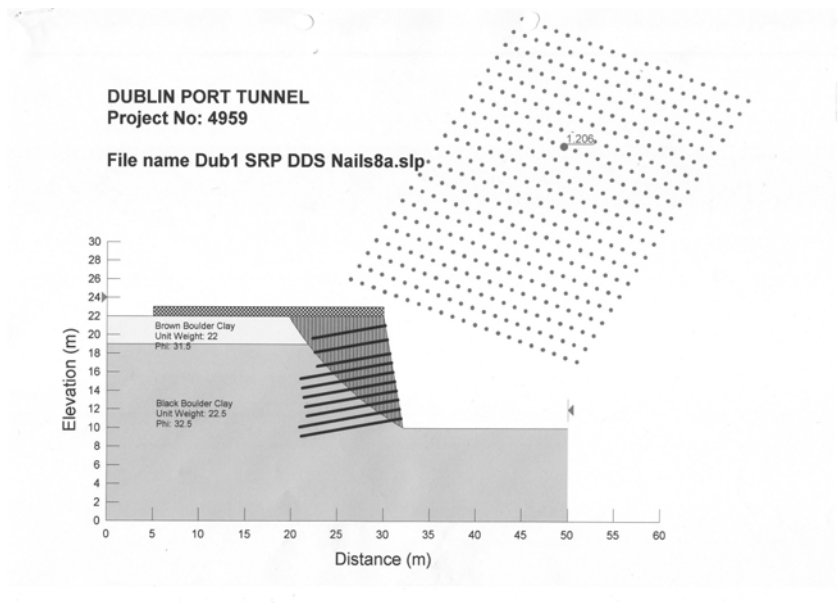


Fig. 6. Layout of instrumented soil nails at Ch 980E - good ground conditions

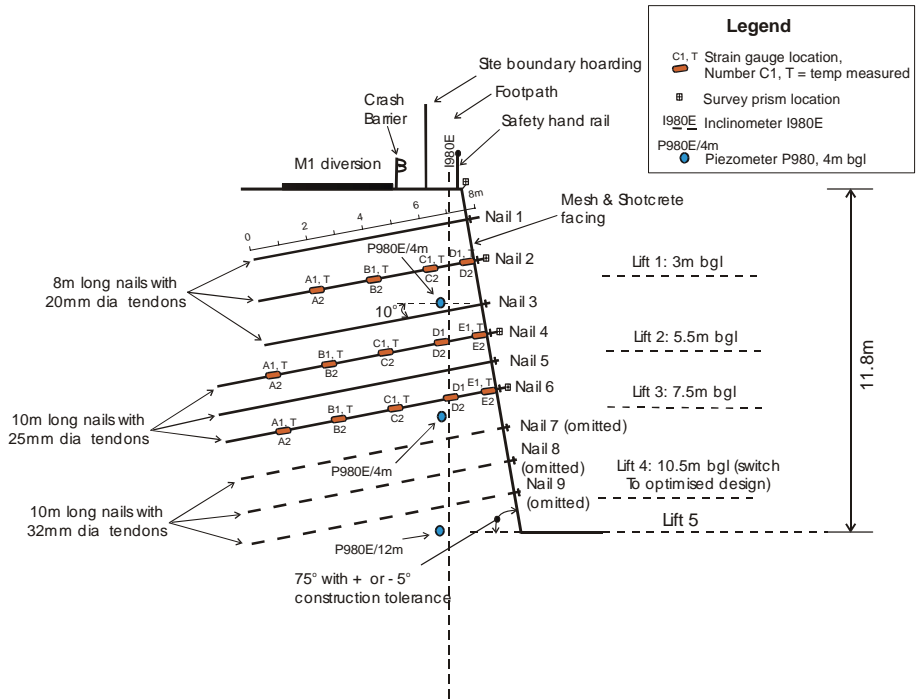


Fig. 7. Layout of instrumented soil nails at Ch 940E - poor ground conditions

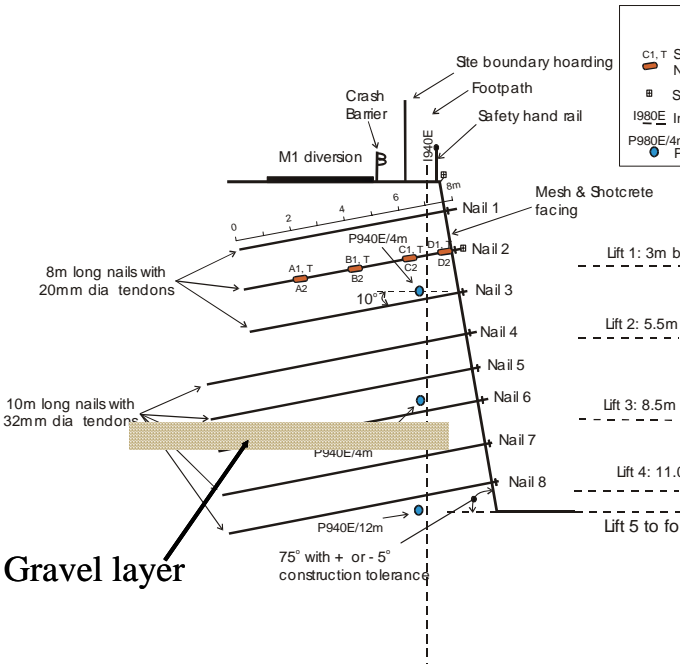


Fig. 8. (a) Nail with strain gauges, (b) Signal pick up device, (c) post inflatable packer, (d) face showing protected nail head, shotcreting, previously installed nails and connection to data logger.

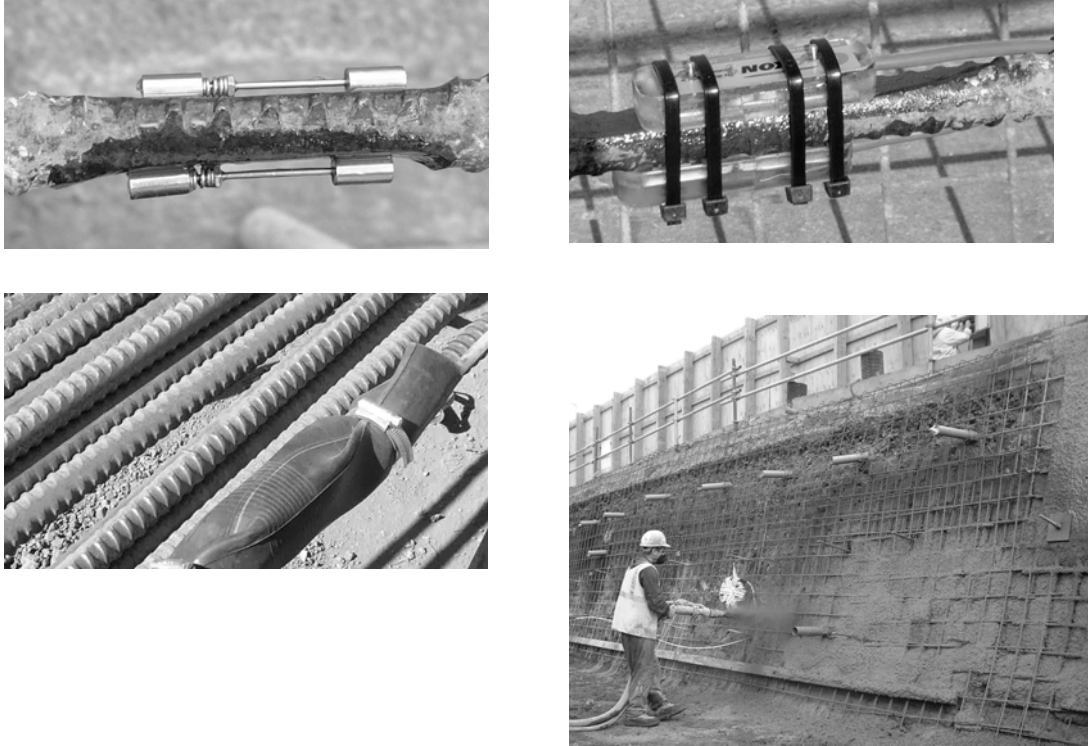


Fig. 9. Average tensile and bending strains in Nail 980E/Row 2

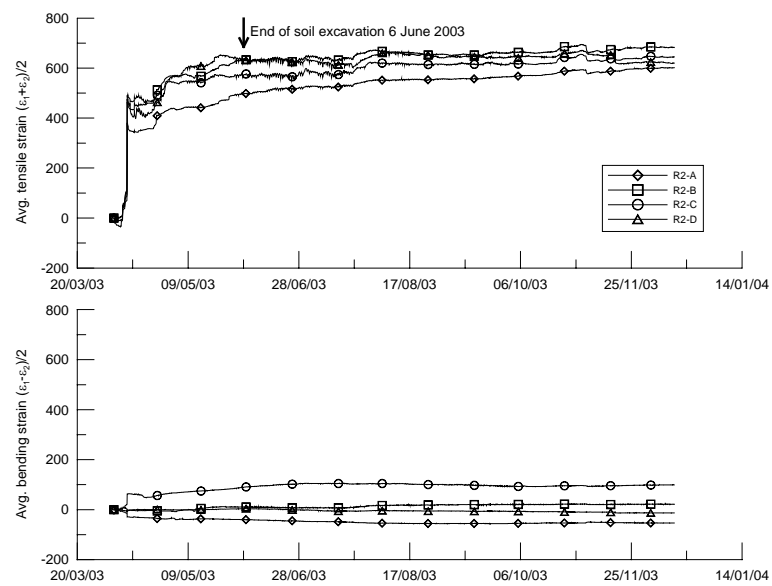


Fig. 10. Average tensile loads in Nail 980E/Row 2

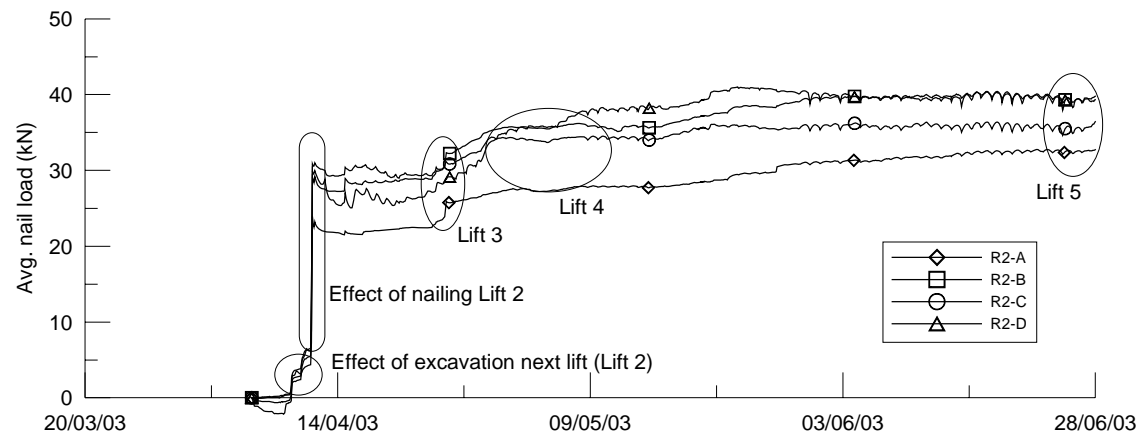


Fig. 11. Situation on site 11 April 03 during nailing of Lift 2 at Chainage 980E



Fig. 12. Development of shear stress along Nail 2 Chainage 980E

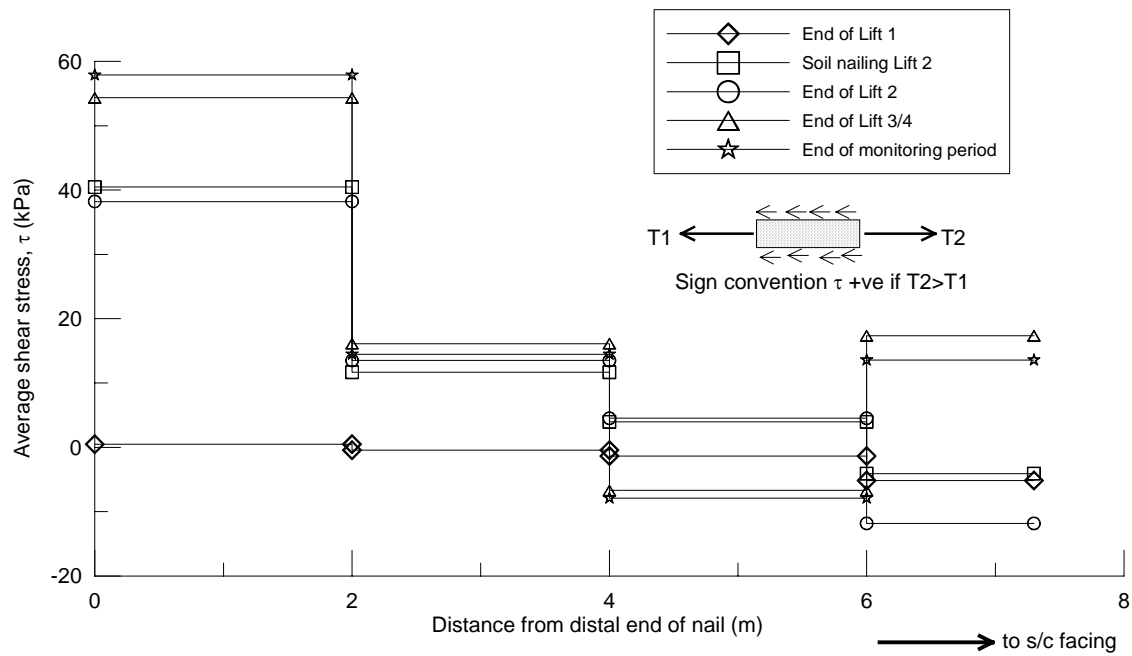


Fig. 13. Development of load – all nails

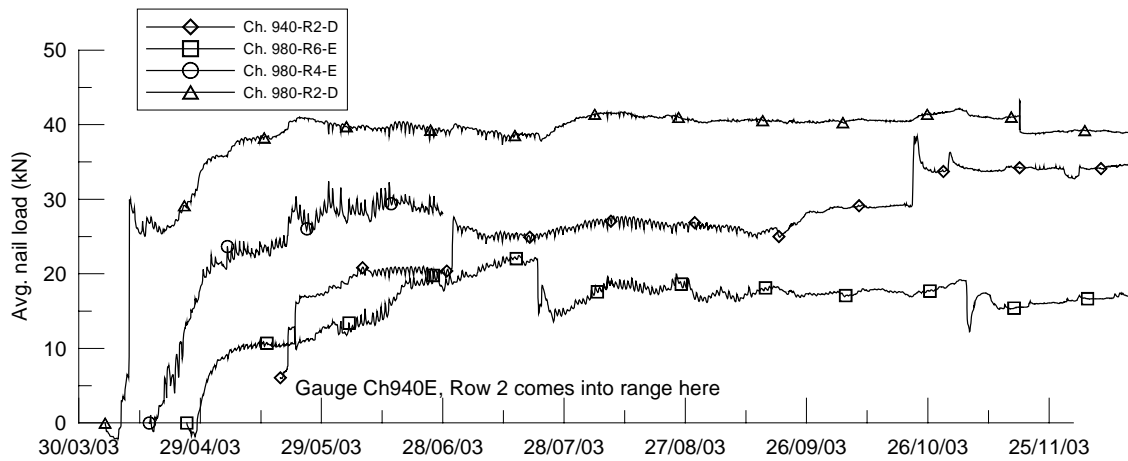


Fig. 14. Lateral slope movement from inclinometers

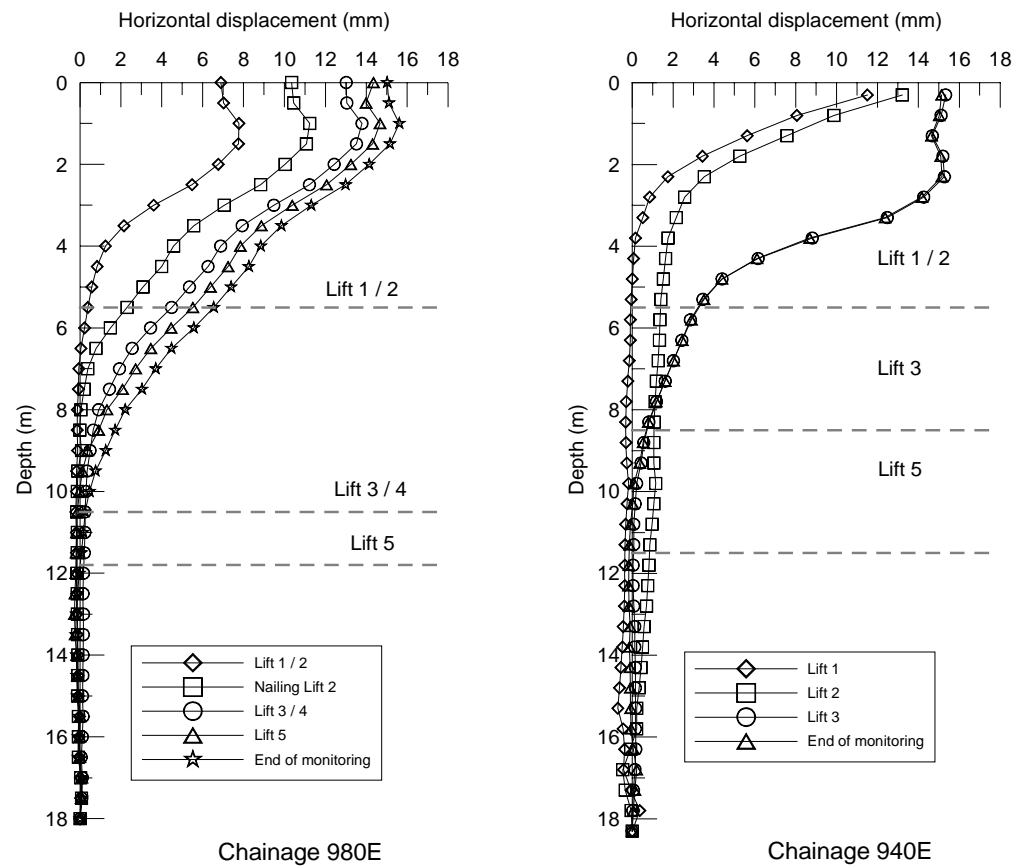


Fig 15. Earth pressures on shotcrete facing inferred from the nail head forces

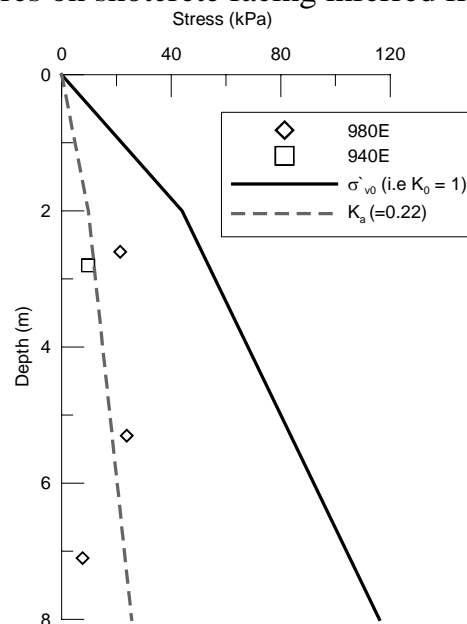


Fig 16. Comparison of ultimate and mobilised nail tendon capacity

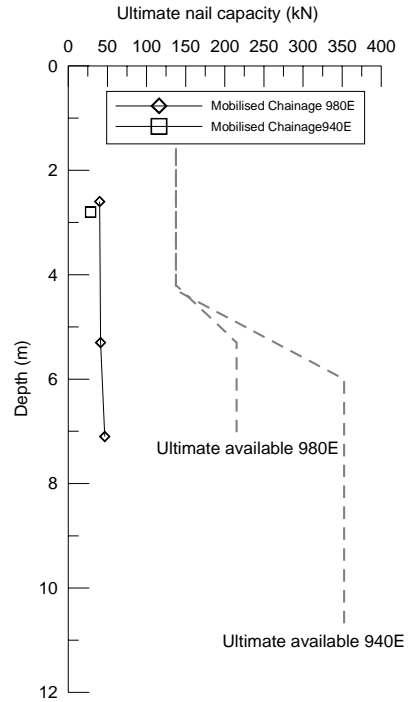
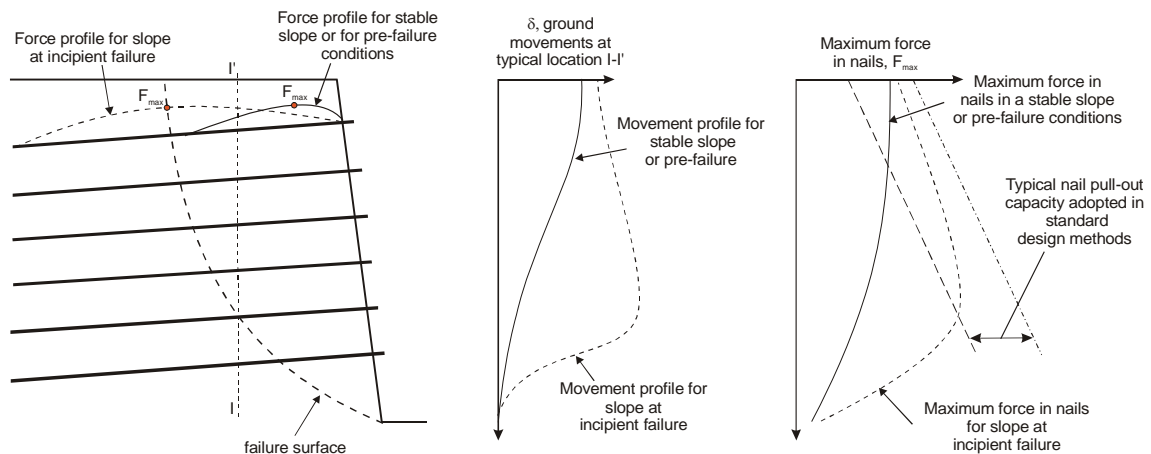


Fig 17. Deformations and nail forces in a typical cut slope with a uniform distribution of nails



End