Temporary Soil Nail Wall In Granular Fill Embankment

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SUMMARY

An existing road system was upgraded to improve traffic flow in south Auckland. The work comprised realignment of the road embankment and construction of a new bridge to replace the existing bridge. Temporary lateral support was required for the existing road embankment to maintain access over the bridge. The paper describes the design of the temporary lateral support comprising a soil nail wall and the use of self drilling anchors. A pre-construction load test on a soil nail, with and without grout, and the proof loading of the contract soil nails is also discussed.

INTRODUCTION

The Great South Road bridge over the North Island main trunk railroad was rebuilt together with realignment of the road embankments to improve traffic flow as shown in Figure 1. The upgrade included the construction of a new six lane reinforced concrete bridge over the railway line and the demolition of the existing bridge originally built in about 1911. The new bridge is wider and 0.75m higher than the original bridge. Reinforced Earth retaining walls were constructed to retain the realigned approach embankments as well as the fill behind the columns supporting the bridge deck.

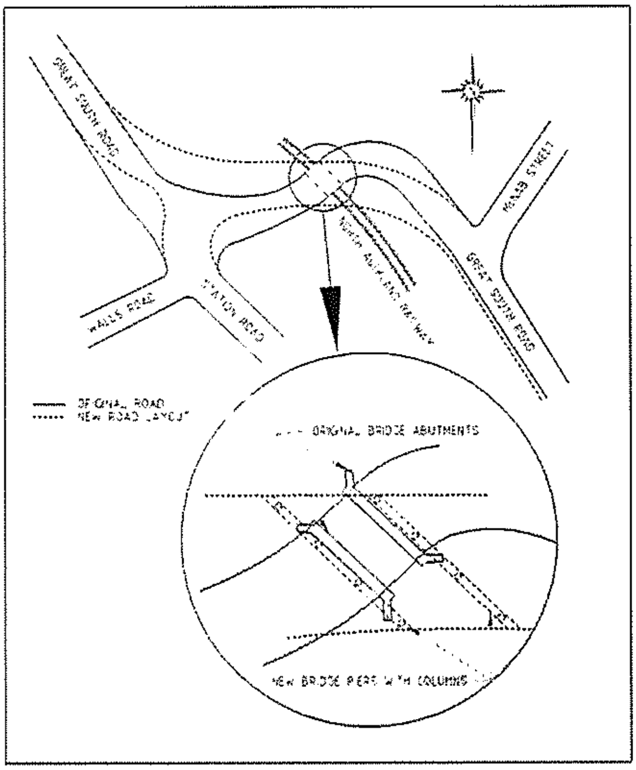

Figure 1 – New Road Alignment and Bridge

The traffic flow over the bridge had to be maintained in both directions during construction. The contractor's construction sequence therefore required the diversion of traffic over the northern half of original bridge while the southern portion was demolished and that section of the new bridge constructed. A photograph of this stage of the construction is shown in Figure 2. When this was completed the traffic was then diverted to the new section of completed bridge and the northern portion of the new bridge constructed.

Space and access was limited for the construction of the new bridge, Reinforced Earth retaining walls and approach embankments. The contractors design brief was for a near vertical temporary lateral support system without temporary propping to retain the existing fill embankment.

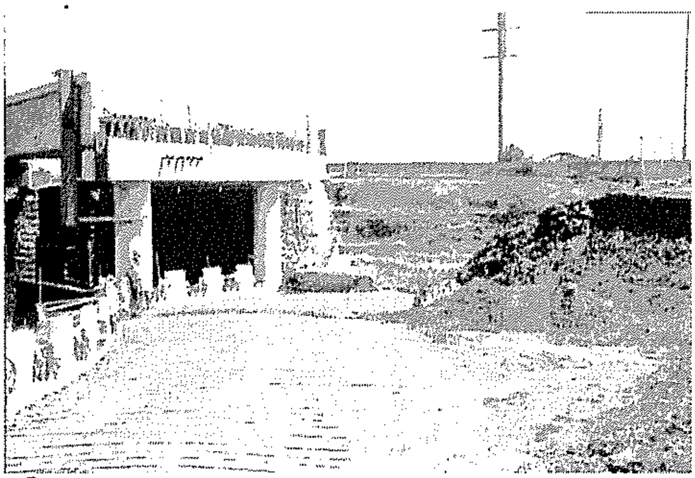

Figure 2 –Completed Soil Nail Wall (Far Side) . Bridge Pier and Reinforced Earth Retaining Wall Under Construction

GROUND CONDITIONS

The geotechnical information at the time of the initial temporary lateral support design indicated that the site was underlain by hard basalt rock at between about 0.5 to 1.5m depth. Overlying the basalt was fill generally comprising sandy silt and basalt gravel, cobbles and boulders. It was anticipated that the original approach embankment material would comprise predominantly lightly compacted granular fill.

At the commencement of site works preliminary excavation was carried out within the existing road embankments to investigate the fill. This showed that the fill within the approach embankments generally comprised scoria and basalt varying from gravel to cobble size overlying a thin layer of tuff at ground level. Basalt rock was confirmed underlying the tuff.

TEMPORARY LATERAL SUPPORT

Temporary lateral support was required for two near vertical cut sections about 6.0m high either side of the existing bridge abutments. To allow a 2.0m set back behind the top of the wall to the edge of the diverted road traffic the temporary cut slopes could be inclined at 75 degrees to the vertical. The two temporary lateral support walls were 17.5 and 21.5m long with concrete traffic barriers along the road edge.

Various temporary lateral support systems were considered and the two most viable options for the site constraints were considered to be :-

a) Anchored king post and waling wall.
b) Soil nail wall.

Both options had practical installation problems. The main construction difficulties with the king post and waling wall was the embedment of the steel king post sections in the basalt rock to obtain sufficient fixity and the installation of deadman type anchors, or similar, beneath the diverted road without significant disruption to the traffic. For the soil nail wall the main difficulties were considered to be the installation of the soil nails through the granular fill and the requirement for the excavation benches to stand unsupported until the soil nails and shotcrete could be completed in stages with depth.

After reviewing the installation difficulties and economics it was decided that the soil nail wall was the preferred option. Initially consideration was given to forming a vertical grout curtain to help support the staged cut excavations. However after some early trial excavations in the road embankment it was apparent that the shallow cuts in the granular fill should stand near vertical for sufficient time to install the soil nails and apply the shotcrete. The grout curtain was therefore not installed. If local collapse of the staged cut face occurred during excavation a flash coat of shotcrete was to be applied immediately to help support the face.

Installation of soil nails in the "traditional" manner of drilling, grouting then inserting the soil nail would typically require temporary casing to support the drilled hole. However the drilling subcontractor put forward an alternative of using self drilling anchors. This option was reviewed and considered feasible in the ground conditions on site. Self drilling anchors comprise hollow drill rods joined together with threaded couplings and sacrificial drill bit. Typically the anchors are installed using a top driven hammer and cement grout is pumped through the drill rod during installation acting as the flush and permanent grout surround to the soil nail. On reaching the required embedment depth the "drill rod" is then left in the hole as the reinforcing bar to the soil nail. The main advantage of this system is that temporary casing is not required and the nails are relatively quick to install. The rate of drilling can be varied as required to allow increased flushing and/or penetration of the grout into the surrounding soil. Secondary grouting can also be carried out at the end of the anchor.

It was acknowledged that there were risks associated with constructing soil nails in the granular fill, and in particular the loss of grout around the reinforcing bar. It was therefore agreed with the contractor that a pre-construction soil nail would be installed and load tested to confirm the practicality of the installation procedure and the load carrying capacity of the soil nail. Furthermore it was agreed that every soil nail would be proof loaded to 1.2 times the design load.

DESIGN OF SOIL NAIL WALL

The soil nail wall was designed using an in-house developed spreadsheet with single wedge stability analysis. The analysis considers wedges at various inclinations to determine the required soil nail resistance for stability. An iterative process is then carried out for different soil nail configurations, lengths and potential failure wedges to determine the minimum required soil nail length for a specified soil nail resistance per metre length.

The stability analysis was carried out carried out using the Strength Factor Method given in Padfield and Mair (1984) to determine the required total soil nail resistance. For the temporary works design a factor of safety (Fs) of 1.25 was adopted together with a worst credible friction angle (Φ) of 33 degrees and zero cohesion for the granular fill. Padfield and Mair (1984) recommend an Fs of 1.1 to 1.2 for temporary works using moderately conservative parameters and 1.0 for worst credible. A higher Fs was used for design because of the traffic behind the wall and consequences of failure. The mobilised friction angle (Φ_{mob}) using the Strength Factor Method is determined as follows.

$$\Phi_{mob} = \mathrm{Tan}^{-1}((\mathrm{Tan}\,\Phi)/Fs)$$

The required total soil nail resistance was calculated using Φ_{mob} determined as above with a factor of safety of 2.0 on the soil nail pull out resistance.

The design pull out resistance of the soil nail was determined by reference to literature and load tests carried out by the drilling subcontractor in similar ground conditions. The subcontractor proposed the use of an HSD R25 self drilling anchor and for this soil nail a pull out resistance of 16kN/m length of nail was adopted for design. The pull out resistance equates to a frictional resistance of 100kN/m^2 for a soil nail of 50mm diameter. USDT (1998) quote typical friction values of 80 to 100kN/m^2 and 120 to 240kN/m^2 for dense and very dense silty sand and gravel respectively. Furthermore Barley (1993) reports a friction value of 110 kN/m^2, without failure, in a mixture of wet sands, gravels and clays. The design friction is therefore within the range of values quoted in the literature.

The HSD R25 anchor details are as follows and the components are shown in Figure 3.

Bar diameter	25mm
Stock bar lengths	2.5m
End drill bit diameter	42mm
Yield load	170kN

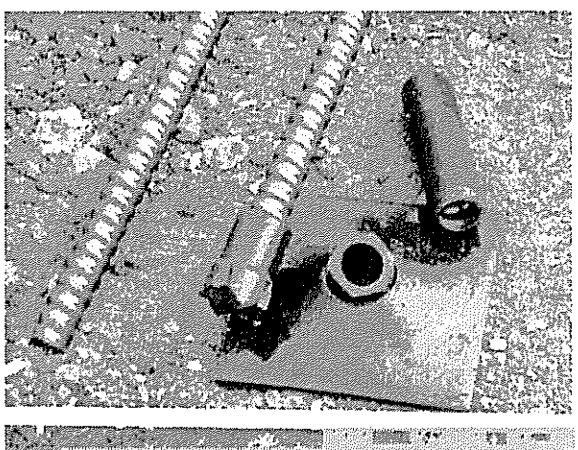

Figure 3 – Pho[illegible] Anchor R[illegible] l.
Drill Bit, Coupling and Anchor Plate (150mm square) with Spherical Nut

The design soil nail layout is shown in Figure 4 that was similar for both walls. A typical section through the soil nail and shotcrete is shown in Figure 5. Initially it was proposed to include strip drains behind the shotcrete but the granular fill was considered to be sufficiently free draining not to require additional drainage. Weepholes were however formed through the shotcrete.

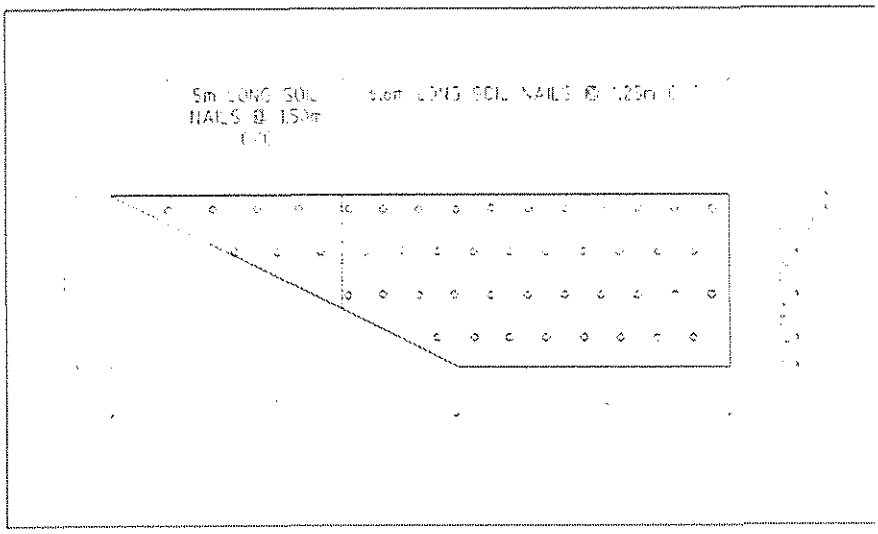

Figure 4 – Western Soil Nail Wall Details

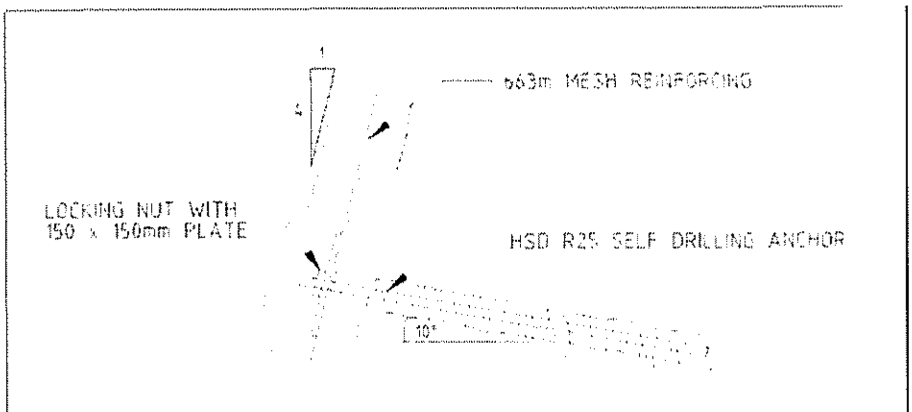

Figure 5 – Typical Section Through Soil Nail

For the temporary soil nail wall the ratio of soil nail length (6.6m) to height (6.0m) is 1.1. However the soil nail wall was designed for traffic loading as specified by Transit New Zealand 'Bridge Manual" that for retaining structures is assumed to be equal to a surcharge pressure caused by 0.6m of fill. If 0.6m is added to the retained height then the equivalent length to height ratio is 1.0. The spacing of the soil nails is 1.25 by 1.5m. These dimensions are consistent with those quoted by USDT (1998) of 0.7 to 1.0 for typical length to height ratios (all soil types) and spacing of 1.0 to 2.0m. Barley (1993) refers to a typical spacing of 0.5 to 0.8m in granular soils, It may however be anticipated that the length to height ratio for the temporary soil nail wall would be higher than generally quoted due to the relatively small diameter of the HSD R25 anchors (about 50mm) and hence lower resistance per metre length of soil nail. In comparison Barley (1999) refers to typical soil nail diameters in the range 100 to 150mm in weak rock to granular materials and 150 to 200mm in cohesive strata that are significantly greater than the HSD R25 anchors.

LOAD TESTS ON SOIL NAILS

The pre-construction soil nails discussed below were installed at about 1.0m depth below the top of the road embankment level and inclined at the design angle of 10 degrees.

Prior to the installation and load test on the pre-construction soil nail an HSD R25 anchor was installed to a 5.0m embedment length using air flush only. No grout was pumped through the drill rod during or after the installation process. The anchor was then load tested to assess the frictional resistance of the bar alone as a comparison to the grouted soil nail. The failure pull out load on the bar was 15kN that equates to a load capacity of 3kN/m length of installed drill rod or friction of about 38kN/m^2 (based on an anchor diameter of 25mm). This compares to the design friction of 100kN/m^2.

After the above test a pre-construction soil nail was installed to a 6.0m embedment length. Approximately 50 litres of grout was used during drilling and, on achieving the required embedment depth, a further about 25 litres of grout was pumped through the end of the soil nail with only a low build of pressure of between about 15 to 20kPa. No grout return was observed at the excavation face during the installation. The grout included a thixotropic additive to reduce grout loss into the granular fill as well as an expansion compensating additive.

The design failure load for the soil nail was 96kN (6m length x 16kN/m). The load was applied to the test nail in lOkN increments and held for at least 10 minutes or until no measurable creep occurred. At 96kN applied load no measurable deflection of the soil nail was observed and the load was then increased to 120kN and held for 1.5 hours with still no measurable deflection. Note that the soil nail deflection was measured from the end of the soil nail rod to an independent reference point using a steel rule. It is estimated that the measured deflection accuracy was about +/- 0.5mm.

The above load test confirmed that the grouted soil nail carried a load of at least 1.25 times the design failure load of the anchor with no measurable deflection and at least 1.1 times the design failure load of a 6.6m soil nail (design length). The latter provided confidence for the 6.6m contract soil nails in the event that the soil nail resistance is not uniform over the length of the soil nail. It is possible that the volume of grout pumped through the end of the anchor may have a significant affect on the soil nail load capacity and therefore the failure load may not be directly proportional to the embedded length. If the maximum test load on the soil nail load was carried uniformly over the embedment length then the friction exceeds 127kN/m^2 (based on a 50mm soil nail diameter).

CONSTRUCTION OF TEMPORARY SOIL NAIL WALLS

The construction sequence for the soil nail wall was as follows:-

(i) Excavate bench to just below the top row of soil nails.
(ii) Install soil nails.
(iii) Place mesh reinforcement and apply layer of shotcrete sufficient to cover mesh reinforcement.
(iv) Fix anchor plates and locking nuts to the soil nails and tighten up against the shotcrete.
(v) Apply final layer of shotcrete covering soil nail anchor plates and locking nuts.
(vi) Form weepholes in the shotcrete face.
(vii) Allow grout to gain sufficient strength then proof load the soil nails to 1.2 times the design load.
(viii) Provided (vii) meets the acceptance criteria repeat (i) to (vii) for subsequent rows of soil nails.

The acceptance criteria for the proof load to 1.2 times the design working load was a maximum deflection of 1.0mm.

The excavation face in the granular fill stood up relatively well and although there was occasional spalling of the fill a flash coat of shotcrete was not required. It was however noted that a relatively thick layer of shotcrete had to be applied in areas to cover the mesh reinforcement mainly due to the uneven excavation face in the granular fill. The soil nails were installed using the same construction sequence as the test nail.

All the contract nails were proof loaded and met the acceptance criteria at 1.2 times the design load.

USDT (1998) quote typical lateral displacements at the top of soil nail walls relative to the overall height of about 0.1% for weathered rock and competent soils, 0.2% for granular soils and 0.4% for fine grained clay soils. It was therefore anticipated that between about 5 to 15mm of displacement could potentially occur during and after construction of the temporary soil nail wall. No physical monitoring of the wall displacement was implemented but daily inspections were carried out to visually inspect the road surface immediately behind the top of the wall. For the duration that the temporary soil nail walls retained the embankment fill no significant deflection was observed, such as cracks within the asphalt pavement, or similar. The walls performed very well and allowed the permanent works to be constructed unimpeded by any obstruction within the working area.

EXHUMED SOIL NAILS

After completion of the southern portion of the bridge and approach embankments the traffic was diverted over the new portion of completed bridge. The northern section of the original embankment was then excavated and the temporary soil nails exhumed. The author was unfortunately not present when this was carried out but visited the site several weeks later and inspected three exhumed soil nails that were still on site. The soil nails were significantly bent during, and also possibly after the excavation and did not show any significant grout along the length of the soil nail reinforcing bar. Each nail did however have a "bulb" of grout at the end of the bar. On questioning the main contractor it is apparent that they did not pay particular attention to the soil nails during excavation but their recollection was that there was grout along its length but that it was not always continuous.

The above observations and discussions indicate that it is likely that the soil nails had a discontinuous anulus of grout along the length of the reinforcing bar with a "bulb" of grout at the end. The continuity of the grout around the bar was probably a function of the nature of the fill that the soil nail was installed through and in particular the voids within the granular fill. It is therefore considered that the soil nail most likely behaved partly as an undereamed anchor with frictional resistance mobilised along its length together with some additional resistance at the end of the soil nail due to the "bulb' of grout.

CONCLUSIONS

1) The soil nail wall provided an economical solution for the temporary lateral support of the embankment with no disruption to traffic during construction and no space restrictions within the contractor's working area once the soil nail wall was completed.
2) For the particular circumstances on site the self drilling soil nail had significant advantages over the 'traditional' soil nail installation procedure requiring the use of temporary casing.
3) The soil nails installed on this site are considered suitable for temporary works but as installed would not be suitable for permanent works because of the discontinuous grout cover along the reinforcing bar. For permanent works additional precautions and grouting would be required for corrosion protection.
4) For the temporary retaining wall it is considered crucial that a pre-construction load test was carried out together with proof loading of each contract soil nail to ensure that the soil nails could safely carry the design load.
5) Because of the number of soil nails installed within the retaining wall there is a certain amount of redundancy in the system should an individual soil nail fail or not carry the full design load. This further reduces the risk of failure of a soil nail wall and, in the author's opinion, was a major advantage over alternate temporary lateral support systems considered for the contract where more reliance would have to be placed on highly loaded individual anchors or tie back systems.

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