STABILISATION OF AN EXISTING RAILWAY EMBANKMENT USING SMALL DIAMETER MINIPILES AND LARGE DIAMETER PIN PILES AT KITSON WOOD, TODMORDEN

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KEYWORDS: Embankment Stabilisation, Pin Piles, Minipiles, Horizontal Ties, Long Term Monitoring.

ABSTRACT
This paper describes the design and installation of an embankment stabilisation scheme carried out for Railtrack plc during 1998. The running rails over this section of embankment were showing signs of significant movement during 1997 and a track speed restriction (TSR) of 5 mph had been imposed. A combined solution consisting of large diameter pin piles to arrest the deep seated movements and tied minipile walls to stabilise the shoulders was chosen. Long term monitoring of the minipile wall and tie forces have been carried out and are discussed.

Figure (i) - Original Embankment Prior To Any Remedial Works (Late 1997)

INTRODUCTION

This £850,000 design and build contract involved the stabilisation of approximately 80 m of embankment adjacent to Underbridge No. 5 at Kitson Wood in Todmorden (see fig [i]). The works were undertaken by Kvaerner Construction (KC) and Kvaerner Cementation Foundations (KCF), operating under a partnership agreement for Railtrack Project Delivery (North West). Underbridge No. 5 is at 29miles 58 chains on the Burnley Branch Line between Hall Royd Junction and Gannow Junction. The line is not electrified and has two tracks which are used for both freight and passenger services.

The embankment and bridge were constructed on a relatively steep slope in an area of complex geomorphology during the mid to late nineteenth century. They form part of over 5,000 bridges, tunnels and earth retaining structures which Railtrack are responsible for in the North West of England.

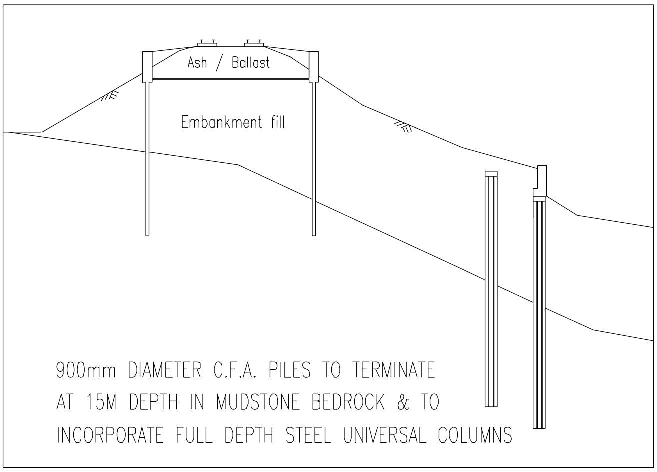

Figure (ii) -Typical Section Detailing Remedial Works

Historical records indicated long term problems with this section of track and several houses in the vicinity have also experienced foundation movement. There was evidence of a rotational slip along the southern critical section of embankment where ground movements have been greatest. A desk study revealed a large berm in this area around the early 1900's, which is a common remedial measure to mitigate rotational slips. Unfortunately, recent development work in the mid 1990's to construct new houses cut into the toe of the slope and triggered off the slope movements again.

Inclinometer and reference peg readings taken during 1997 indicated surface movements of up to 150mm over a 6 month period. A combination of deep seated rotational movement and ash/ballast ravelling over and down the steep embankment shoulders as the trains passed were deemed to be the most probable failure mechanisms. This significant movement resulted in a monthly re-ballasting requirement of approximately 40 tonnes to keep the railway lines within the required tolerances. Furthermore, a 5 mph TSR was imposed to minimise dynamic loading and consequent settlement. The combination of re-ballasting (which only exaggerated the movement by applying additional surcharge) and TSR were creating serious commercial and safety issues within Railtrack and a permanent long term solution had to be sought.

The final solution consisted of the following two separate elements and these remedial works were carried out between January and April 1998 (see figures [ii] & [iii]).

1. Large diameter pin piles with full depth heavy universal columns were installed lower down the slope to stabilise the deep seated rotational slip.

2. Small diameter minipile walls were installed along both sides of the embankment shoulder to prevent the ash/ballast movement. These minipile walls were connected via horizontal ties through the embankment and reinforced concrete (r/c) capping beams constructed along either side.

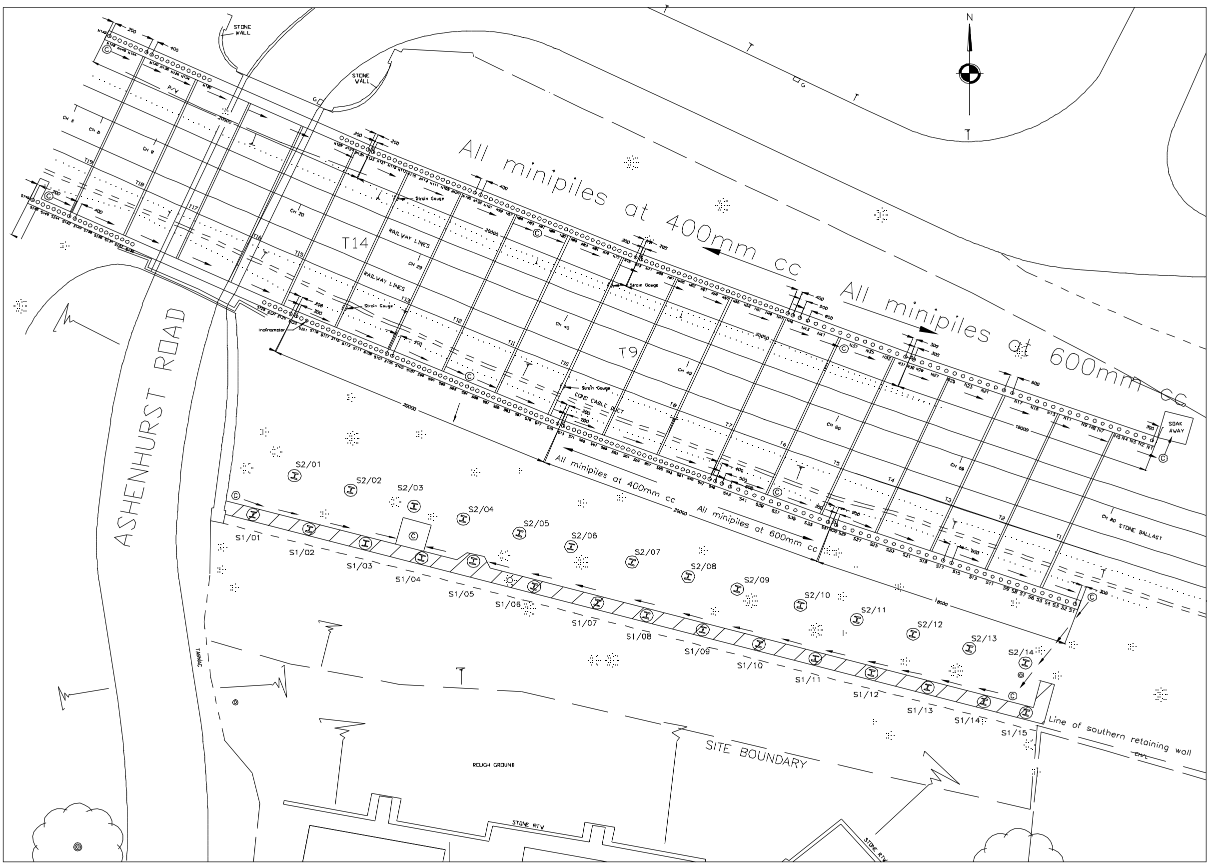

Figure (iii)- Plan View of Site and Proposed Remedial Works

A comprehensive landscaping and re-grading exercise was finally carried out which included providing ‘level’ rear gardens to the new properties.

An interesting aspect of this paper is the long term monitoring of the horizontal tie forces and inclinometer readings within the minipile wall. The build-up of forces within horizontal ties passing under railway embankments has been the subject of considerable discussion over recent years. In particular, whether the dynamic loadings from the passing trains create a ‘ratchet’ effect thus causing excessively high tie forces to be generated. The effect can be further exaggerated by seasonal vegetation/ moisture content variations. This paper attempts to dispel some of these concerns and presents readings taken over the first three years which show the tie forces levelling off at well below the predicted design values. The final set of readings are due to be taken in July 2003.

REGIONAL GEOLOGY

The site is situated just above the flood plain alluvium of the River Calder. The Todmorden and Kinderscout Grits outcrop on the valley sides above the site, which is overlain by a few metres of hill wash (clay derived from weathered mudrocks). The solid geology comprises Todmorden Grit over Shales of Millstone Grit age. Faulting is present near the site and stratal dips are variable. The north eastern slopes and most of the valley side are affected by major landslips for up to 100 metres above the bridge and track, where the Kinderscout Grip outcrops.

SITE INVESTIGATION and GROUND CONDITIONS

The first site investigation was carried out by Strata Surveys Ltd during December 1996 and comprised three shell and auger boreholes. One of the boreholes was located on the crest of the embankment and was advanced through the bedrock to 22 metres depth using rotary coring techniques. The remaining two boreholes were located on the problematical southern embankment and reached refusal at approximately 12 metres depth. A series of eight dynamic probes were also undertaken across the area to provide additional information. Inclinometers, piezometers and permanent ground markers were then installed for subsequent monitoring over the next 12 months.

Made ground was encountered in all boreholes and extended to between 1.5 metres and 4.6 metres depth. A 0.7 metre layer of ballast overlying ash fill was present along the top of the embankment. The made ground was generally found to comprise a soft to firm brown silty clay, possibly reworked/completely weathered mudstone. The underlying mudstone bedrock was highly weathered with no fresh rock to 12 metres depth.

Prior to commencement of the remedial works, an additional two rotary cored boreholes to 15 metres depth were undertaken by Geotechnical Engineering Ltd along the line of the proposed large diameter pin piles. This was carried out to allow close inspection of the ground conditions over the full depth of the pin piles and confirm its ‘drillability’ for the proposed continuous flight auger (CFA) piling techniques. These additional site investigation works formed part of KCFs contract and allowed the design proposals to be finalised.

MONITORING WORKS - PRE-REMEDIAL WORKS

One month after the December 1996 site investigation, the first inclinometer readings indicated movement at around 6 metres below ground level along the southern side. Subsequent readings a month later could not be completed beyond 6.5 metres depth because the inclinometer tube had become too bent. This period of intense movement coincided with the placement of 160 tonnes of ballast along the top of the embankment. Within the monitoring period available it was not possible to clearly identify the groundwater regime in the vicinity, nevertheless standing water levels appeared to be around 4 to 6 metres below the embankment. Twenty permanent ground markers (PGMs) were installed, primarily on the southern embankment to create two distinct cross-sections. The PGMs placed along the northern embankment did not show any appreciable movement. The PGMs on the southern embankment showed significant movement on both level and position (150 to 200 mm) and were consistent with a rotational type failure mechanism.

PIN PILES - DESIGN

A considerable amount of slope stability analysis work was undertaken by Bullen Consultants Ltd during late 1997. Due to the particular nature of the geology, two stability models had been used:

Method 1: Bishop's Rigorous Method with parallel inclined interslice forces.
Method 2: Janbu's Method with horizontal interslice forces.

Various combinations of loadings and water table regimes were considered within these sensitivity analyses. Mobilised & residual angles of friction of 21^0 and 13^0 respectively were obtained which are within the range anticipated for pre-sheared medium plasticity soils or clay filled joints. The overall geometry of the slip surfaces are similar for both methods under the same loading conditions and the slipped mass weighed around 3,400 kN/m to 3,520 kN/m per metre of slope.

The detailed pin pile design is outwith the scope of this paper, suffice to say that it followed the recommendations of Viggiani (1981), whereby the out-of-balance force resulting from conventional stability analysis is taken by pin piles to provide an adequate factor of safety. Final checks on the design were carried out using the procedure reported more recently by Poulos (1995).

The final design comprised the installation of 29 No. 900 mm diameter CFA piles to 15 metres depth positioned in two rows, 3.5 m apart and staggered at 4.0m centres longitudinally. Each pile was reinforced full depth with a grade 50 steel 356mm x 406mm x 551 kg/m universal column (UC) encased within 35 MPa concrete. It was important to ensure that the column was installed so that its maximum bending resistance was mobilised in the direction of the slip.

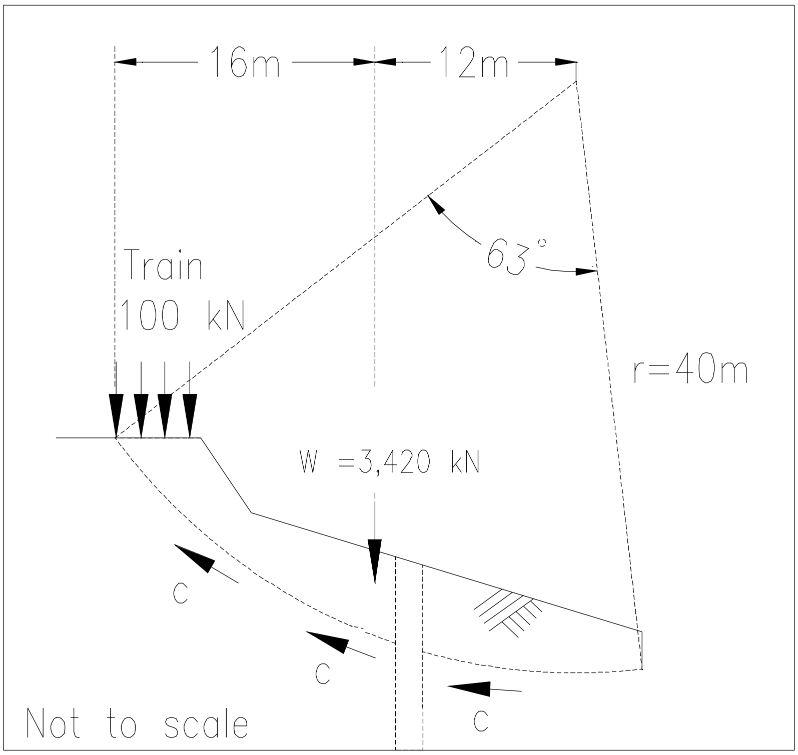

Figure (iv) - Conjectured Slip Circle

KCF undertook a simplistic analysis in order to verify the adequacy of the proposed scheme prior to its adoption. The basis of this check was to draw a conjectured slip surface from between the tracks, through a point 6.5 metres below the embankment mid-point and finally exiting at the base of the recently constructed retaining wall at the toe of the slope (see figure [iv]). A unit width of slope was then considered and, making the assumption that the factor of safety (FOS) was currently unity, the shear resistance along the slip line could be back calculated using the Swedish Method.

Therefore, taking moments about the centre of the conjectured slip circle,

$$\text{F.O.S.} = 1.0 = c \cdot r^2 \cdot \text{Ø}$$

$$\backslash \quad c = 24.5 \text{ kN/m}^2$$

This is a reasonable 'ultimate' cohesion along the soft to firm clay failure interface.

Therefore in order to increase the minimum overall factor of safety of the slope from 1.0 to 1.5 an additional stabilising force of 539 kN/m needed to be provided.

The proposed pin piles were at 2.0 metre centres and therefore each pile needed to be able to safely withstand a tangential force of 1,078 kN.

A Broms analysis for a free-headed pile gave a maximum generated bending moment of 1,882 kNm for this shear force.

The grade 50 356 x 406 x 551 kg/m UC has a M_r of 3,239 kNm which gave a load factor of 1.72 and KCF were prepared to adopt the proposed design on the basis of these checks.

In addition to the design of the permanent works solution the stability of the embankment in the temporary situation also needed to be considered. This was achieved by again considering moments about the centre of the conjectured slip circle from the train and piling rig/temporary platform loadings. The pin pile works were programmed to take place during a four week line closure (for the reconstruction of the nearby Horsfall Tunnel) and therefore the train loadings would be temporarily removed.

Moment from trains per metre = 2,800 kNm (temporarily removed)

Moment from piling rig / temporary platform per metre = 2,760 kNm

In summary, the additional moments from the piling rig were approximately equal to the moments removed from the railway due to the train live loading. Furthermore, the pin piles would be installed from Ashenhurst Road, i.e. the embankment stability would be gradually increased as the piling rig worked its way along the line of piles. Finally, continued monitoring of the embankment pegs would also be carried out throughout the works. In view of the foregoing comments the temporary stability of the embankment was deemed to be satisfactory.

PIN PILES - CONSTRUCTION

The 900mm diameter x 15 metre deep CFA piles were constructed using a Soilmec R622 piling rig. This is one of Cementation's largest rigs weighing 70 tonnes and is capable of generating 220 kNm of torque. The 900mm diameter continuous auger string was bored to 15m depth and then concreted through the hollow stem as the auger was extracted. A total volume of approximately $10m^3$ to $12m^3$ of concrete was required for each pile. The whole process was carefully monitored via pressure and flow measurements to ensure a homogenous 900mm diameter pile was formed over the whole length (see figures [v] and [vi]). A C35 blended concrete pump mix was delivered to site in $6m^3$ wagons and temporarily stored in an agitator for onward distribution to the pile bore as required. The CFA piling technique proved eminently suitable for these ground conditions and the piles were formed in an efficient manner with minimal vibration and relatively little noise.

Figure (v) - Construction of 900 mm Diameter CFA piles

Figure (vi) - Lifting of Universal Column (note the steepness of the overall slope)

Immediately following completion of the concreting of the bore the heavy section UC was carefully lowered in to the full depth, ensuring that the correct orientation was maintained. Each 15m long UC weighed 8.3 tonnes and was formed by welding together a 10 metre and a 5 metre section. A 45^0 point was formed at the bottom end and a lifting eye at the top. All of the fabrication works were carried out at Cementation's plant and workshops depot in Bentley near Doncaster, using coded welders. Centralisers were added to the UC at regular intervals during installation. All of the concreted bores were formed and UCs inserted to the full depth without any problems and production rates of 6 to 8 piles per day were achieved (see figures [vii] and [viii]).

Figure (vii) - Installation of Universal Column

Figure (viii) - View looking South-East

Following completion of pin pile installation, a 2.0m high reinforced concrete wall/capping beam was constructed which connected together the front row of pin piles (see figures [ix] and [x]). The main reasons for this wall were to reduce the slope over the top critical section of the embankment and also to provide relatively level rear gardens to the new properties. Communication and liaison between Railtrack and the local residents was extremely important and they were kept fully informed of all proposals and developments. Inevitable disruption and inconvenience was caused during the works, especially during the closure of the adjacent Ashenhurst Road for approximately 4 months. The overall scheme for the garden re-grading/retaining wall/landscaping was jointly discussed, developed and agreed with the local residents.

Figure(ix) - Trimmed Pin Piles

Figure(x) - Retaining Wall Reinforcement

A drainage system was also installed over the whole embankment and connected into the main surface water drainage beneath Ashenhurst Road. This was a relatively simple and cost effective method of further enhancing the embankment stability.

MINIPILES AND TIES - DESIGN

The minipile design was based upon a worst case scenario whereby the embankment material on the downside failed completely and slumped away to its residual angle of friction of approximately 13^0 (see figure [xi]). This leaves the minipile wall supporting the "equivalent" retained height of embankment. KCF have used this design method successfully on a number of other Railtrack / London Underground Ltd. embankment stabilisation projects in recent years. Two design cross-sections were analysed and the critical 6.0 m equivalent retained height case will be used to explain the process. The water table regime within the embankment was potentially complicated and a 1.0m variation between the active and passive sides at dredge level was assumed.

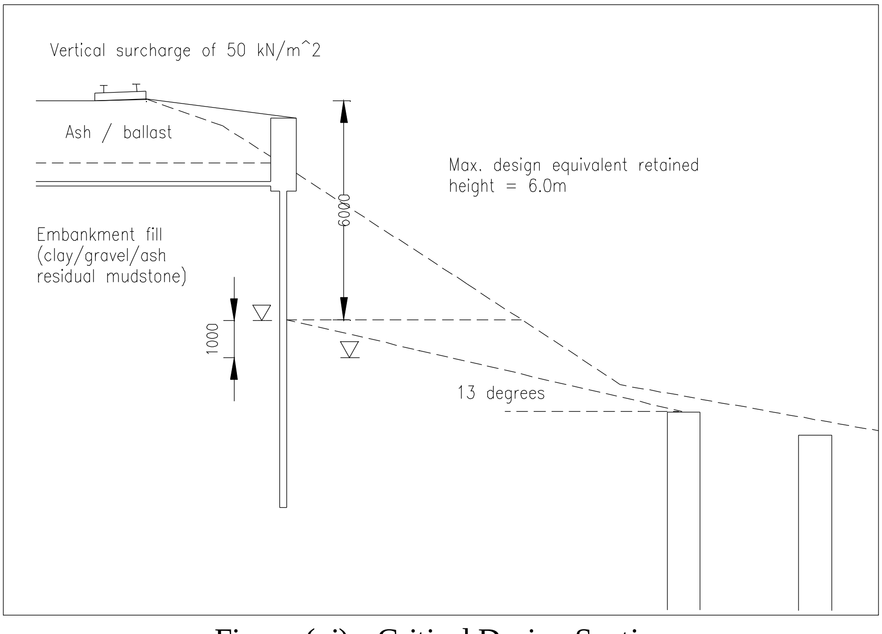

Figure (xi) - Critical Design Section

The earth pressure coefficients were obtained from BS 8002:1994 and the KCF ‘in house’ computer programme ‘Cemwall’ gave a maximum generated bending moment of 77.2 kNm/m.

A nominal 250mm diameter grouted minipile containing 7 x T20 within a 170mm diameter R6 helical was proposed of length 10.0m.

Therefore, considering these minipiles, of M_r 47.0 kNm at 400mm centres a load factor of 1.52 was proven, which is acceptable.

The helical shear reinforcement calculation was carried out in accordance with BS 5400:Pt.4:1990.

For the second design case of 5.0m equivalent retained height the minipile spacing could be increased to 600mm (also containing 7 x T20) and the pile length reduced to 9.0m.

The maximum horizontal propping force was 84.0 kN/m from the critical 6.0m equivalent height design case, but was adopted for the second design case because the tie bar materials cost was relatively small.

A maximum tie bar spacing of 4.0m was used:

\ Maximum Design Tie Force = 336.0 kN

For a 50mm diameter grade 500 rebar,

Ultimate Tensile Capacity = 981.7 kN

\ Structural Load Factor = 2.92

This is a suitably conservative value for a horizontal tie compared to the 1.5 to 2.0 generally used.

A 1.4m deep x 0.7m wide r/c “capping” beam was used with nominal reinforcement to connect the ties to the minipile walls. The design and construction of this beam followed the guidelines of BS 8110 and is not covered within this paper.

250mm square x 30mm thick mild steel plates were proposed for each end of the tie.

BS 8110 clause 5.2.3.4 states that for a bedded bearing,

Ultimate Bearing Stress = 0.6 . f_{conc} = 21 MPa.

\ Ultimate Plate Capacity = 1,271.26 kN

\ Structural Load Factor = 3.78

The ties were incorporated within a plastic duct which extended into the r/c beam along each side and the plates were cast into this beam. A fluid cement grout was pumped around the ties to ensure that there were no voids. These measures ensured that there would be full corrosion protection throughout without the need for any maintenance.

MINIPILES AND TIES - CONSTRUCTION

The minipiles were installed using two twin tech TD308 drill rigs with 2.2m masts. These are the smallest rigs in Cementation's fleet and consist of a 1.5 tonne drill unit and a 1.5 tonne powerpack, which may be up to 30 metres away. Most of the drilling was undertaken from scaffold platforms along either side of the embankment. Prior to commencement of any operations, a green zone fence line was erected 3.0m from the nearest running rail to allow the works to take place whilst the line was in operation. The fence lines and scaffold platforms were vitally important with regard to safety and great care was taken with their construction. In particular, the transom spacing was matched with the minipile spacing to ensure ease of installation. Temporary 273mm OD diameter casing was used through unstable strata and 240 mm diameter open hold boring thereafter. A specially purchased boring head was used to penetrate the mudstone bedrock using augering techniques. Following drilling to the required depth, the augers were carefully removed and a 1:1 colloidally mixed sand:cement grout with a 0.42 water:cement ratio was pumped into the base until full. The centralised reinforcement cage was then immediately installed to the full depth. Production rates of between 5 and 8 minipiles per day per rig were achieved (see figures [xii] and [xiii]). These minipiles were installed to within 1.0m of the rear of the bridge abutments.

Figure (xii) - Installing Minipile Reinforcement

Figure (xiii) - Trimmed Minipiles

Upon completion of the minipiles, the installation of the ties commenced. These were installed using a small 'Grundomat' pneumatic hammer.

The advantages of installing horizontal ties should not be under estimated when compared with the commonly used alternative system of installing inclined raking ties from each side:

i) A small working platform is required along one side only.
ii) Installation is from one side only (i.e. 1 tie may be the equivalent of 3 to 6 rakers along each side).
iii) They are normally short compared to raking minipiles.
iv) They provide a very efficient horizontal propping system.
v) It is faster when compared to scaffolding both sides and installing probably many more rakers.
vi) It is more cost effective because only 2 men and a small amount of plant are required.
vii) It has the ability to install ties over bridges unlike raking minipiles.

A small amount of structural repair works were also undertaken on Underbridge 5 comprising stitch drilling and minor grouting/pointing works, however these are not covered within this paper.

Minor problems were encountered during the installation of the initial ties due to the grundomat hammer meeting obstructions. The most suitable ground conditions for the grundomat hammer would be a firm consistent cohesive fill/natural material. The solution was to resource a hammer which was able to install its own plastic liner as it drilled, and then retract back along it if an obstruction was encountered. Upon reaching the far side the hammer was removed leaving the plastic liner insitu through the embankment into which the 50mm diameter rebar could be installed in 4 metre sections, connected together using full strength couplers. Alignment was generally good when the grundomat had been set up/off correctly and no major obstructions were encountered, on average an installation tolerance of 1 in 50 was achieved. Production rates of 3 ties / day were achieved after the initial problems (see figures [xiv], [xv] and [xvi]).

Figure (xiv) - Grundomat Hammer

Figure (xv) - Tie Installation Complete

Figure (xvi) - Tie Bearing Plate

Finally, the r/c beam with tie plates, drainage and compacted free draining shoulder support were completed by Kvaerner Construction with the line open throughout (see figures [xvii] and [xviii]).

Figure (xvii) - Beam Reinforcement

Figure (xviii) - Completed R/C Beam

MONITORING WORKS - POST REMEDIAL WORKS

Tie bars T9 and T14 passing through the railway embankment have been instrumented with vibrating wire strain gauges. Each tie bar has been fitted with two pairs of gauges, one pair at each end, approximately two metres from the r/c capping beam.

One inclinometer tube has also been installed down the centre of minipile S121 on the South side of the embankment, adjacent to tie bar T14.

Datum readings on the tie bars and inclinometer were taken on 14 April 1998 and monitoring is to be carried out after the following periods have elapsed (the final reading will be in July 2003):

1 month	(May 1998)	-	Cumulative 1 month
2 months	(July 1998)	-	Cumulative 3 months
4 months	(November 1998)	-	Cumulative 7 months
8 months	(July 1999)	-	Cumulative 1 year 3 months
9 months	(April 2000)	-	Cumulative 2 years
32 months	(July 2003)	-	Cumulative 4 years 8 months

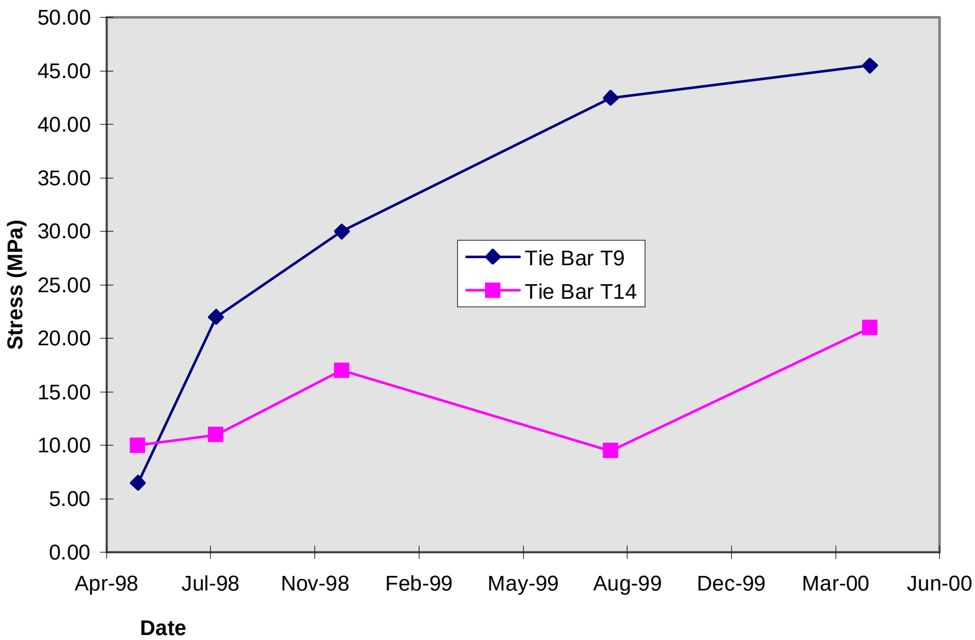

Figure (xix) Average Tie Bar Stresses

The average tie bar stresses are given in figure (xix) to date which shows the maximum stress to be levelling off at around 50 MPa in T9.

A stress of 50 MPa in Tie T9 is equivalent to a force of 98.1 kN, which is 29% of the design force of 336.0 kN.

Tie bar T9 is in the middle of the problematical section of embankment where the movements have been the greatest. Tie bar T14 appears to have a stress level of around 50% of T9. This may be due to the proximity of the bridge abutment which is approximately 10m away providing additional support.

The inclinometer readings gave deflections of approximately 7 mm at the top of the r/c capping beam, reducing to zero at 4.0m depth. There were 4 No. full strength coupled joints per tie, which inevitably would have generated a few mm of movement as the forces increased. These readings have levelled off and are showing no signs of movement over recent periods.

The generated force of 98.1 kN in T9 would cause an extension of approximately 4mm which is reasonable when compared with the inclinometer readings.

CONCLUSIONS

This interesting project has clearly demonstrated how large diameter pin piles can be used to stabilise deep seated slope stability problems. Furthermore, how a minipile structure connected together via horizontal ties can also be incorporated to provide shoulder support to the adjacent railway lines. Finally, that 'long term' tie bar readings are around 30% of the predicted design values for these ground conditions, loading regime and analysis method.

Figure (xx) - Completed Remedial Works

ACKNOWLEDGEMENTS

The author would like to thank Railtrack Project Delivery (North West), Kvaerner Construction Ltd and Kvaerner Cementation Foundations Ltd for their kind permission to publish this paper.

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