Horsfall Tunnel, Todmorden: The design, construction and performance of a temporary reticulated mini-pile retaining wall.

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Introduction

Railtrack maintain over five thousand bridges, tunnels, and earth retaining structures in the North West of England, most of which were designed and built during the mid to late nineteenth century. Horsfall Tunnel is one such structure located between Todmorden and Hebden Bridge on the Manchester to Normanton Trans-Pennine express route. The tunnel was built between 1837 and 1840 and has a long history of movement confirmed by recent monitoring showing deflections of up to 3mm per annum. A 40 mph speed restriction had been in place for some time and visual inspections near to the west portal indicated the tunnel to be increasingly intolerant of further movement and in danger of structural failure (Figure 1).

Figure 1: The west portal before the start of site works

Following a feasibility study procured by Railtrack the most cost effective engineering solution was considered to be to demolish approximately 35m of the distorted west portal. The adjacent hillside would be retained by a reticulated mini-piled retaining wall constructed from above the existing tunnel in advance of the demolition. Railtrack had negotiated a four-week blockade with the train operating companies for completion of the works.

Concerns highlighted during the detailed design, regarding the long term stability and performance of the reticulated mini-piled wall, led to a decision to re-instate the tunnel. The mini-piled wall was used to temporarily retain up to 14m of adjacent slope during demolition and re-construction of the new tunnel. Significantly, the increased scope of works had still to be completed within the four week blockade.

This paper presents the design, construction and performance of the temporary reticulated mini-pile wall designed and constructed by Kvaerner Cementation Foundations. Ove Arup & Partners were appointed as independent checker.

Contract Procurement

Kvaerner Construction won the contract for the works, in joint venture with Kvaerner Cementation Foundations, during October 1997. The contract was tendered and awarded within the space of a few weeks and an extremely tight construction programme was necessary in order to meet the pre-determined start date of the four week blockade. The blockade start date had been agreed with the train operators eighteen months in advance and could not be changed. Site work commenced on 15 October 1997 with the piling works being completed by late December allowing sufficient time for concreting and curing of the pile caps before the start of the blockade on 19 January 1998. The contract was let under the I.Chem.E (Green Book) form of contract with an agreed target cost. The reticulated minipile wall formed approximately thirty percent of the £1.4m overall cost.

Design

Reticulated mini-piled structures have been used in a variety of contexts ranging from slope stabilisation to retaining walls. Many examples of each exist in the technical literature taken from Europe and the United States.[1, 2, 3, 4] It is, however, Dr F. Lizzi who is considered to have introduced the "Reticolo di Pali Radice" (Reticulated Root Piles) technique in the 1950's. As the name suggests, the system comprises a network of minipiles constructed in the ground to act together as a unified mass of soil and structure. An excellent summary of the design principles for this and related techniques is set out by Bruce & Jewell, although the most specific design guidance for reticulated mini-piled walls is provided by Lizzi.[5, 6, 7]

The conforming "Engineer's" scheme comprised a contractor designed reticulated mini-piled wall, approximately 35m in length with retained heights ranging from 9m at the exit portal up to 14m at the interface with the existing tunnel. The proposed wall was a permanent structure with pre-cast reinforced concrete facia units, faced with local stone, tied at pile cap level and founded upon strip footings. The uphill slope behind the wall ranged from sub-horizontal at the exit portal up to 35 degrees at the tunnel interface.

The approach adopted for design was that set out by Lizzi.[7] The network of mini-piles and surrounding soil are treated as a gravity structure, with the usual external stability checks, then the ability of the pile elements to accommodate the internal forces is checked. The true distribution of load amongst the piles is very difficult to assess accurately and so it is important to include a reasonable degree of redundancy in the design.

Ground Conditions

The published geology maps showed the tunnel to be underlain by sandstones and mudstones of the Millstone Grit Series, probably the Todmorden Grit. The bedrocks are indicated to be sub-horizontally bedded and a vertical fault, trending NW - SE, is shown to pass close to the western portal of the tunnel.

This information was supplemented by a site investigation undertaken through the difficult terrain immediately upslope from the tunnel. The investigation confirmed the sequence of mudstone, siltstone and fine to medium grained sandstone and showed it to be overlain by a mixture of superficial debris, probably scree, and made ground. The borehole logs showed the mudstone, siltstone and sandstone to be fractured with a variety of subhorizontal joints and some more steeply dipping joints. The joints were often weathered and slickensided and sometimes, in zones of heavy fracturing, completely weathered to clay. The superficial debris

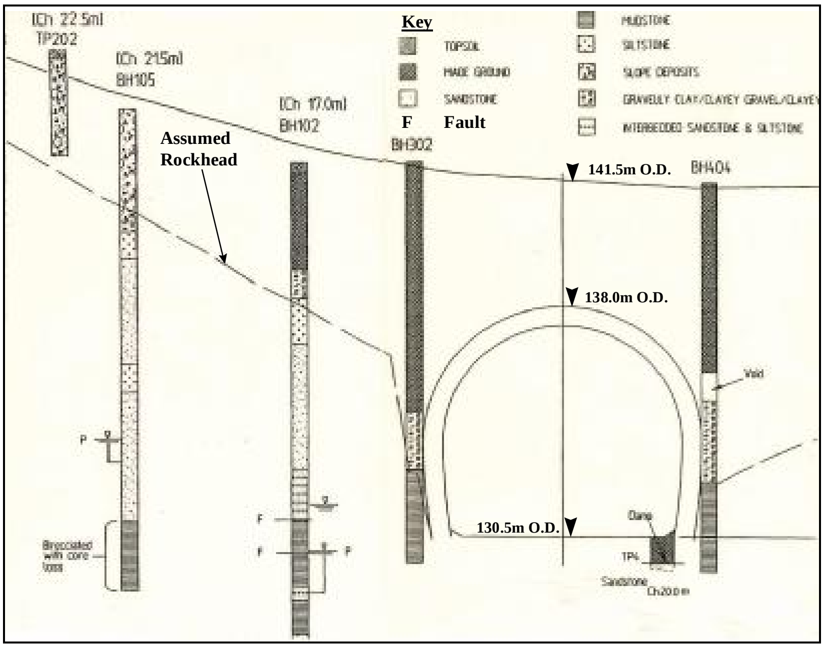

Figure 2 : Ground Profile at Ch 10m (Courtesy of Hyder Consulting Ltd)

comprised sand, gravel, cobbles and boulders of fine to coarse grained sandstone and was difficult to distinguish from the made ground which was probably derived from it. The made ground / scree deposits were up to 9m thick immediately behind the proposed wall. However, the bedrock, weathered to varying degrees, was usually encountered 2m to 3m above track level (Figure 2).

The tunnel is sited at mid slope on the northern side of the Calder Valley, in an area and geological setting renowned for slope failures (Figure 3). In order to place the published geology and site investigation records into full context, a geomorphological assessment was undertaken of the area surrounding the site using available aerial photographs.

The area of ground above the tunnel, to the north of the site, was shown to comprise a generally flat plateau area, until the ground slopes away at a steep angle toward the relatively flat flood plain on the valley floor. The slope above the tunnel appeared to comprise a scree slope of material weathered from the rock outcrops. The scree may be a more recent feature concealing a much deeper failure surface generated during the periglacial period when the valley would have been over deepened and higher porewater pressures would have been operating along with processes of stress relief, cambering and valley bulging. Further evidence of these processes showed themselves as a prominent semi-circular feature below a wooded scarp on the opposite side of the valley confirming that the valley had experienced landslip in the past.

The subhorizontal jointing found in the boreholes is likely to represent bedding and the more steeply inclined jointing is likely to have been generated by movements related to faulting and folding of the strata. The heavily fractured and weathered zones of rock could have been generated by a variety of events including faulting and / or mass movement in the form of landslip, cambering and valley bulging.

Figure 3 : A panoramic view of the site during construction

Movement due to any of these events will have substantially reduced the shear strength of the rock mass and over sixty percent of the recorded discontinuities were in an unfavourable orientation. Of these, over fifty percent were dipping at an angle greater than 30 degrees.

Detailed Design

The network of mini-piles was designed as a gravity structure and provided with an appropriate degree of redundancy to accommodate any uncertainties in the system. External stability was checked, adopting conventional factors of safety, and analysis of the internal structure was undertaken to establish the design loadings for the various structural elements.

It was during development of the detailed design that some concerns were raised regarding the long term performance of the wall. The concerns were twofold; firstly that the durability of the piles may not meet the 120 year design life requirement unless an alternative structural facing was developed. The conforming scheme showed a clear gap between the front row of piles and the pre-cast facia panels, leaving the front row piles exposed in the long term. There could be no guarantee that some of the piles would not be damaged during demolition of the tunnel, without any quick and simple method for repair, especially if the damage was at high level. Programme constraints prevented alternative structural facings from being investigated further. The second concern was for the overall stability of the structure following the removal of the tunnel given the adverse dip of the sandstone / siltstone bedding beneath the toe of the wall.

As the tunnel and the hillside behind had remained broadly stable for the 150 years or so since its construction, and given the concerns detailed above, the difficult decision was taken to re-construct a new tunnel and so confer temporary status on the reticulated mini-pile wall.

External Stability

A practical pile layout was determined in order to establish the dimensions of the gravity structure for use in analysis. The position of the front row piles was pre-determined by their proximity to the running rail, in combination with an allowance for drilling tolerances. The position of the back row of piles was constrained by the need to minimise the possibility of undermining the existing slope during construction of a working platform, required for installation of the piles and construction of the pile cap. Given these constraints, a layout comprising three rows of nominal 200mm diameter piles at 500mm centres was established, with alternate piles on the second row inclined at 10 degrees and alternate piles on the back row inclined at 15 degrees (Figure 4).

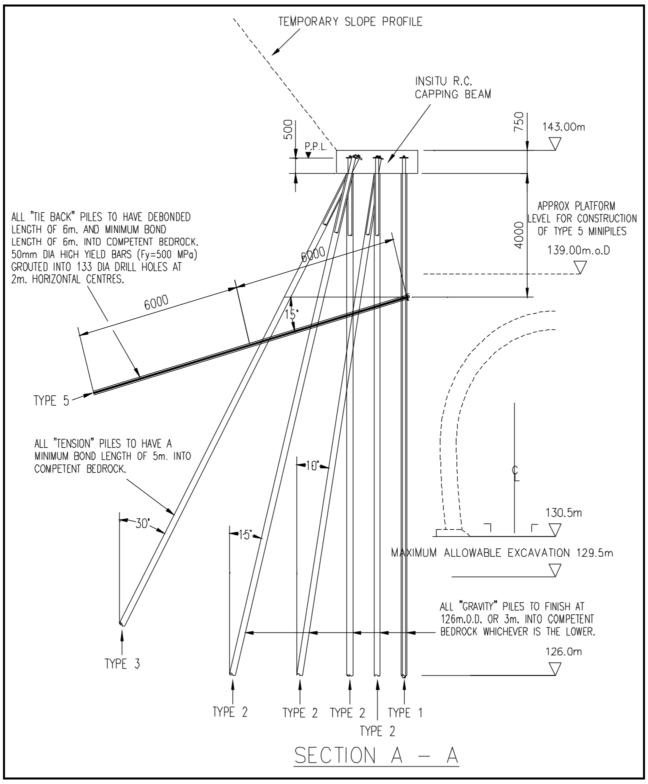

Figure 4: A typical cross-section through the retaining wall

In addition, the front row piles were supplemented by intermediate piles, toed into bedrock, to form a contiguous front face.

The structure, now defined by the front and back row of piles, was analysed as a conventional gravity retaining structure against sliding, overturning, bearing capacity, and overall stability. As a result of these checks, additional piles were added to the back row of piles inclined at 30 degrees in order to improve the overturning stability. Furthermore, concerns highlighted regarding the global stability of the whole structure prompted a detailed investigation by the independent checker, Ove Arup and Partners.

The strata encountered at the site are generally flatbedded or dipping at a shallow angle and there was evidence of strata dipping both into and out of the slope. There were also steeply dipping discontinuities inclined out of the slope in an unfavourable orientation some of which were considered to have experienced movement and thought likely to have a reduced shear strength. Global stability was checked using the OASYS computer program SLOPE. A variety of circular and non-circular (wedge) failure surfaces were analysed, resulting in a minimum factor of safety of 1.2. This was considered acceptable for a temporary structure but not for a permanent structure with a 120 year design life given the significant consequences in the event of failure. These conclusions contributed significantly to the decision to re-construct the tunnel.

Internal Stability

The internal stability of the structure was checked using a variety of techniques by both the designer, Kvaerner Cementation Foundations, and the independent checker, Ove Arup & Partners.

Hand Calculations

Initially, the overall pile / soil system was considered and then the specific issues relating to the piles at the front face of the wall were addressed.

The overall pile / soil system was treated as a propped cantilever, with the piles assumed fixed into the bedrock below track level, in order to calculate the maximum bending moment resulting from the applied soil and water pressures. Considering the front and back rows of piles as compression and tension members tied at the pile cap, the forces in the individual piles elements were calculated. Both the central row of piles and the soil mass strength were ignored for analysis purposes, but clearly they are an integral part of the system. Having assessed the forces in the pile elements, a check was undertaken to ensure that there was adequate skin friction available to transfer the load from the piles into the ground. Finally, the horizontal shear resistance through the pile / soil structure was checked at various levels up the wall.

The two key issues that needed to be addressed at the front face of the wall were the potential loss of ground between the front row piles and their potential for buckling. Loss of ground was considered only to be a problem through the zones of made ground and was addressed by supplementing the main structural piles with intermediate piles toeing a metre or so into bedrock, effectively creating a contiguous wall along the front face. The problem of potential bucking was an altogether more difficult problem.

The back and central row piles are entirely surrounded by soil and are consequently afforded a good degree of lateral restraint. The front row piles, in contrast, are restrained only by the pile cap and the foundation bedrock. The forces calculated in the individual piles were compared to the calculated Euler buckling load and the pile section found to be inadequate. Further investigation demonstrated that provision of a tie at about 4m below the maximum retained height would increase the Euler bucking load to an acceptable value. Whilst this minor addition appeared to be a relatively straight-forward solution on paper, the practical difficulties were immense given the need to install the ties in advance of the tunnel demolition.

Clearly the final assessment of any scheme of this nature relies heavily on experience and engineering judgement, supplemented by analysis in a variety of forms. Having prepared the original design by hand it was independently checked using a range of other analytical techniques.

Analytical Computer Models

The OASYS program GSA was used to assess the internal distribution of forces in the piles. For the purpose of this analysis the reticulated piles were assumed to act as a structural frame loaded along the back row of the piles. The analysis gave acceptable and credible values for the axial loads in the piles but large lateral displacements and bending moments were calculated in the piles because it did not model the shear stiffness of the soil enclosed by the piles.

The OASYS finite element computer program SAFE was therefore used to model the wall using a simple linear elastic finite element model to assess the way in which earth pressures applied to the virtual back of the assumed “gravity structure” would be distributed within the structural elements of the wall.

The model comprised a relatively simple rectangular mesh, distorted to represent the geometry of the wall in cross-section. Elements to the rear of the mesh were given an enhanced vertical stiffness to represent the axial stiffness of the piles. A column of elements at the front of the mesh was given a bending stiffness that modelled the more heavily reinforced contiguous piles which form the front face of the wall. The pile cap was modelled by elements with representative stiffness.

Loads were applied as line loads to the rear of the mesh, modelling the distribution of earth pressures assessed during the external stability checks. The finite element mesh was extended beneath the proposed structure, and restrained by fixities at the base, at a distance below the structure which would ensure that there would only be a small effect on the forces calculated within the structure itself. The 30 degree raking anchor piles were not modelled in the analysis, nor were the row of ties which were later added to the structure. The analysis showed deflections which were significantly smaller than those shown by the structural frame analysis and the bending moments were, as a consequence, also reduced. The computed axial forces were comparable to those calculated in the GSA analysis.

Through use of these models, and following the decision to re-construct the tunnel, the independent checker was able to accept the reticulated mini-pile wall as a temporary structure.

Mini-Piled Wall Construction

The commencement of the minipiling works was delayed until early November due to difficulties in preparing the working area and access road. These problems were primarily due to the steeply sloping site and the adverse weather conditions in this exposed part of the Pennines. The ground level rose steeply behind and also along the line of the minipile wall from the tunnel portal and it was necessary to form three separate working platforms with a one metre step between each level. In addition, a sprayed gunite facing was also required to prevent degradation of the slope above the highest platform, where minor cutting into the hillside had been unavoidable.

Figure 5 : Casagrande "C6" rig operating above the tunnel

Originally it had been planned to use four Casagrande "C6" drilling rigs but it quickly became apparent that this would not be practical nor cost effective in such a small restricted site. The area directly above the tunnel had a weight restriction of four tonnes which severely restricted the movements of the "C6" drill rigs weighing around 12 tonnes. It was therefore decided to work two drill rigs for two shifts each day. Consequently the minipile installation works were ongoing from 6 a.m. to midnight throughout Monday to Friday (Figure 5).

The ground conditions were extremely variable, ranging from large boulders to a fine sand and were potentially a very difficult drilling medium. In order to efficiently bore through this material, a new drilling system was resourced from Europe which allowed fully cased full face rotary percussive drilling using an air flush. The "Symmetrix" system consists of a pilot bit on the end of a down the hole hammer which connects into a ring bit on the end of the temporary casing. This allows both the pilot bit and the ring bit to rotate together, thus advancing the bore and the temporary casing simultaneously, to form a nominal 230mm diameter hole. This system proved efficient and reliable. Production rates of up to nine minipiles per shift were achieved and no hammers or bits were lost. Having drilled through the superficial deposits and upon reaching competent bedrock, the "Symmetrix" pilot bit was removed and the remainder of the boring undertaken using a normal down the hole hammer with a 178mm diameter bit.

Throughout the drilling works careful monitoring of the existing tunnel lining was carried out to check for any potentially damaging vibration. The results indicted that the passing trains created significantly more vibration than the localised down the hole hammer drilling. There were also concerns about the verticality of the mini piles within the limited space available for the tunnel reconstruction.

Several of the bores along the critical front row were checked for verticality before grouting, these gave excellent results with verticalities of 1 in 100 or better being achieved.

Following completion of the drilling the hammer and rods were removed and a small diameter tremie pipe inserted to the base. The bore was then completely filled with a 1:1 sand / cement colloidal (high shear) grout mix with a 0.43 water / cement ratio resulting in a high strength fluid grout mix with unconfined compressive strengths in excess of 50 MPa at 28 days. Despite the variable ground, grout takes during the works were not excessive and no evidence of leakage was apparent in the existing tunnel.

Once the bore was fully grouted the centralised reinforcement was carefully lowered into the fluid grout to the full depth. The reinforcement in all rows except the front row consisted of fully threaded 40mm and 50mm diameter "MAC500" (Fy = 500 MPa) bars installed in three metre lengths, connected over the mini pile bore, using full strength threaded couplers. Along the front row a 133mm O.D. tube with a 14.3mm thick wall (Fy = 600MPa) was used to enhance the bending moment capacity. Again, due to access and handling restrictions these tubes had to be cut and threaded into 3.0m lengths for reconnection above the minipile bore. In order to ensure full strength at each joint an external coupling and parallel thread detail was designed and prepared at the Kvaerner Cementation Foundations fabrication workshop in Doncaster.

Programming and sequencing of the works was carried out on a daily basis in order to maximise productivity. It was also necessary to provide a staged hand over to Kvaerner Construction to allow early construction of the pile caps. The mini pile installation was completed a few days before Christmas, providing just sufficient time for curing of the final pile caps prior to the blockade in mid January.

Figure 6: A view of the wall shortly after demolition of the tunnel

During the first two weeks of 1998 the ground above the tunnel was locally excavated adjacent to the pile caps to within 1.0m of the tunnel crown. This was required to allow the lighter Casagrande "M3" drill rig to install a series of inclined ties at about four metres below the top of the wall to remove the risk of potential buckling. Despite a restriction being imposed on the tunnel for additional weight, the removal of weight was equally risky because of the potential of a pin forming in the arch. Nevertheless, it was essential for the design that the ties were installed and operational before the demolition as the loading onto the wall was expected to be immediate and rapid. All of the ties were successfully installed in advance of the demolition which commenced on 19 Jan 1998 (Fig 6).

Performance

From the early stages in the design it was apparent from its size and nature that the wall was quite unusual and it was clear that there would be a keen interest in its performance. The wall was made more unusual in that it would be brought into operation in such dramatic fashion following demolition of the tunnel and it was agreed with the construction team that a range of measurements should be made before, during, and immediately following the demolition.

Instrumentation

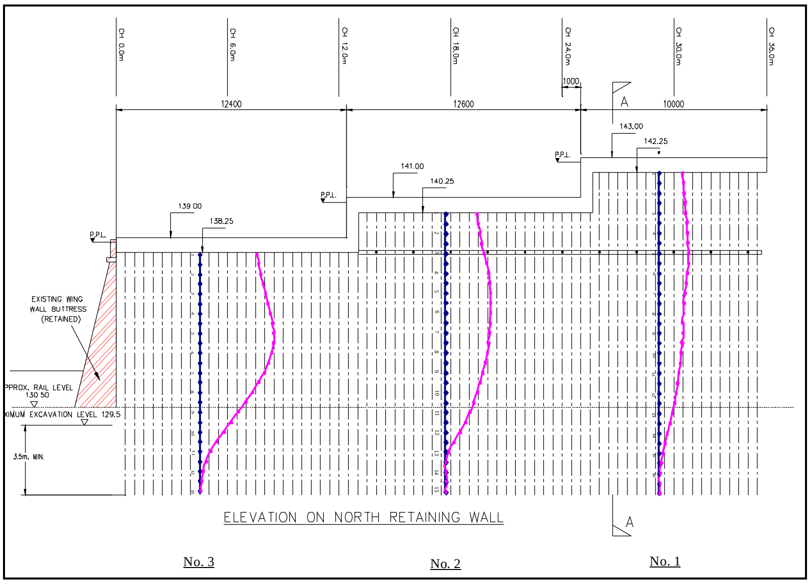

Figure 7 : Results from inclinometers installed in the front row of piles

A total of nine 3D survey targets were established on the pile caps, three at each step level. In addition, three inclinometer tubes were installed within a front row pile at each retained height. Base readings were established for both the survey targets and the inclinometers following completion of the pile caps. The instruments were read again immediately following demolition of the tunnel and subsequently until it was clear that the movements had ceased. The inclinometers showed the wall to deflect a maximum of 60mm at the lowest section and only 25mm at the highest section, with between sixty and seventy percent of the deflection occurring instantaneously and the remainder creeping over the subsequent week (Figure 7).

Interpretation

The largest deflections occured at the section of lowest retained height, for reasons that become clear upon inspection. The lowest section is restrained laterally only by the pile cap whilst the intermediate section is restrained also by the additional ties. Finally the highest section, yielded the least deflection but was additionally tied into the adjacent hillside beyond the interface between the old and new tunnels. Clearly there must have been some influence from the variation in level of made ground behind the wall along its length, but the results appear sufficiently unambiguous to confirm the influence of the additional ties.

Figure 8 : The re-instated tunnel in operation

Conclusions

As the result of these works, Railtrack have been able to restore the full line speed of 70 mph with the associated benefits to the travelling public (Figure 8). The project has also provided the opportunity to solve a technically challenging problem using the latest drilling technology and an established, although uncommon technique, to retain substantial wall heights where the more conventional alternatives have been ruled out. The overall project success was acknowledged when the project won the 1999 ICE Yorkshire Association award for excellence in concept, design and execution of Civil Engineering works. A plaque was unveiled at the site on 26 March 1999 by the President of the Institution of Civil Engineers.

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