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ABSTRACT

The landslip is located in the Giant's Seat Woods area of Kearsley, Bolton in Greater Manchester, England. The ground conditions at the site comprise made ground material over mudstones of the Manchester Marls. An area of land crossed by Red Rock Lane had, since the late 1700s, experienced land slip resulting in the ground generally moving south towards the adjacent river. By the late 1990's the movement of the slip had resulted in fractures and fissures under the road and damage to several other services. A project was developed to stabilise the land along approximately 130m of Red Rock Lane.

Options to provide a solution included service diversion, improved drainage, toe weighting, dig out and replace, soil nailing, piled wall and a rock anchored pile wall. The ground conditions and compact nature of the site suggested that a drilled minipile and anchored wall would offer the optimum performance.

Inclinometer results confirmed clearly that the slip surface lay predominantly within the mudstones, moving at up to 40mm a month, at depths ranging between 3 and 10m below road level. Continued monitoring of the inclinometers and piezometers through construction of the wall provided interesting and useful results regarding the slip behaviour and was finally able to confirm the restoration of the groundwater table and the curtailment of the slip movement.

INTRODUCTION

This paper presents a history of a failing slope and the development and implementation of a geotechnical solution to arrest the movement. From desk study work the early history of local slope failure is tracked, changes of land use are detailed along with review of monitoring, investigation and testing of the slip. The case history then follows the decision making process to determine a stabilising solution and finally details the design, construction and monitoring of the mini-piled wall used to prevent further failure. The post-constructional movements of the slope and wall are contrasted with those measured prior to the stabilising measures.

The relatively modest scale of the project is in contrast with the depth of historic desk study information available. In addition, the completeness of the records and the clarity of results of the slip monitoring allow a ready appreciation of the geotechnical problem and the response of the slip to treatment.

The case history runs from about 1790 through to present day. The slip is located along Red Rock Lane, Kearsley, Bolton in the UK. Figure 1 indicates the approximate location within the UK. The road cut by the slip links Ringley Fold Wastewater Treatment Works and Rhodes sewage farm, it also provides the sole access to Giants Seat Horticultural Nursery. To the north of the road is the previous course of the Manchester Bolton & Bury Canal, now in filled, and a steeply sloping

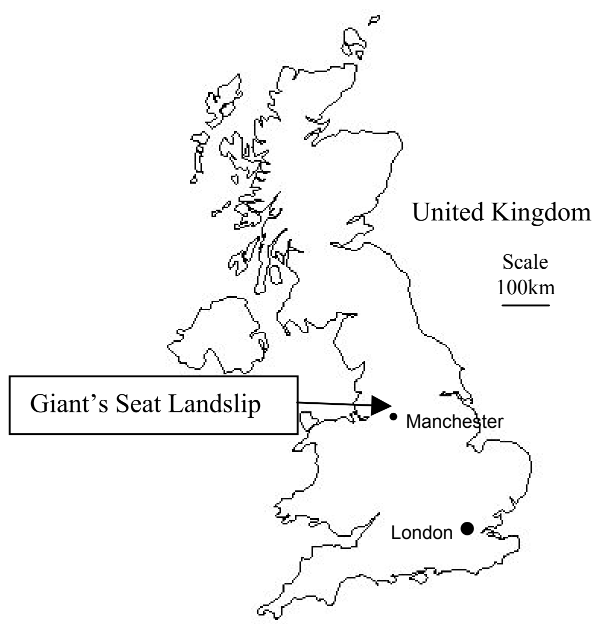

Fig. 1. Location of Giant's Seat Landslip

wooded area of land Giant's Seat Wood. The steep sloping banks to the River Irwell lie immediately to the south of the road.

SUMMARY OF THE PROBLEM

In the UK there are likely to be many natural slopes with a factor of safety (FoS) close to unity. Most of these are not in urban areas, but where slips occur which compromise human activities consideration is required as to how to geotechnicaly manage the risk of failure, and to develop strategies and solutions to do so.

The Giant's Seat Landslip, although in a semi-rural area, affects a number of infrastructure and service assets. The slip has been managed through a combination of monitoring, investigation and finally stabilisation measures. At the point in time that the decision was taken to stabilise the slip the failure zone potentially affected:

- Telephone lines
- Electricity Lines
- A Road – used by general public and sludge tankers
- A Sludge Main
- A Sewer Pipeline.

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Figure 2. shows the location of the site and the area that had exhibited ground movement, problems with services and cracking and settlement of the road.

In the past the movement of the slip had necessitated repairs to a number of these services. The following sections examine the history of the slip, it's change of use and the build up of data and information required to properly manage the risk and develop sufficient knowledge of the failure mechanism and geometry in order to provide a solution to the problem. A

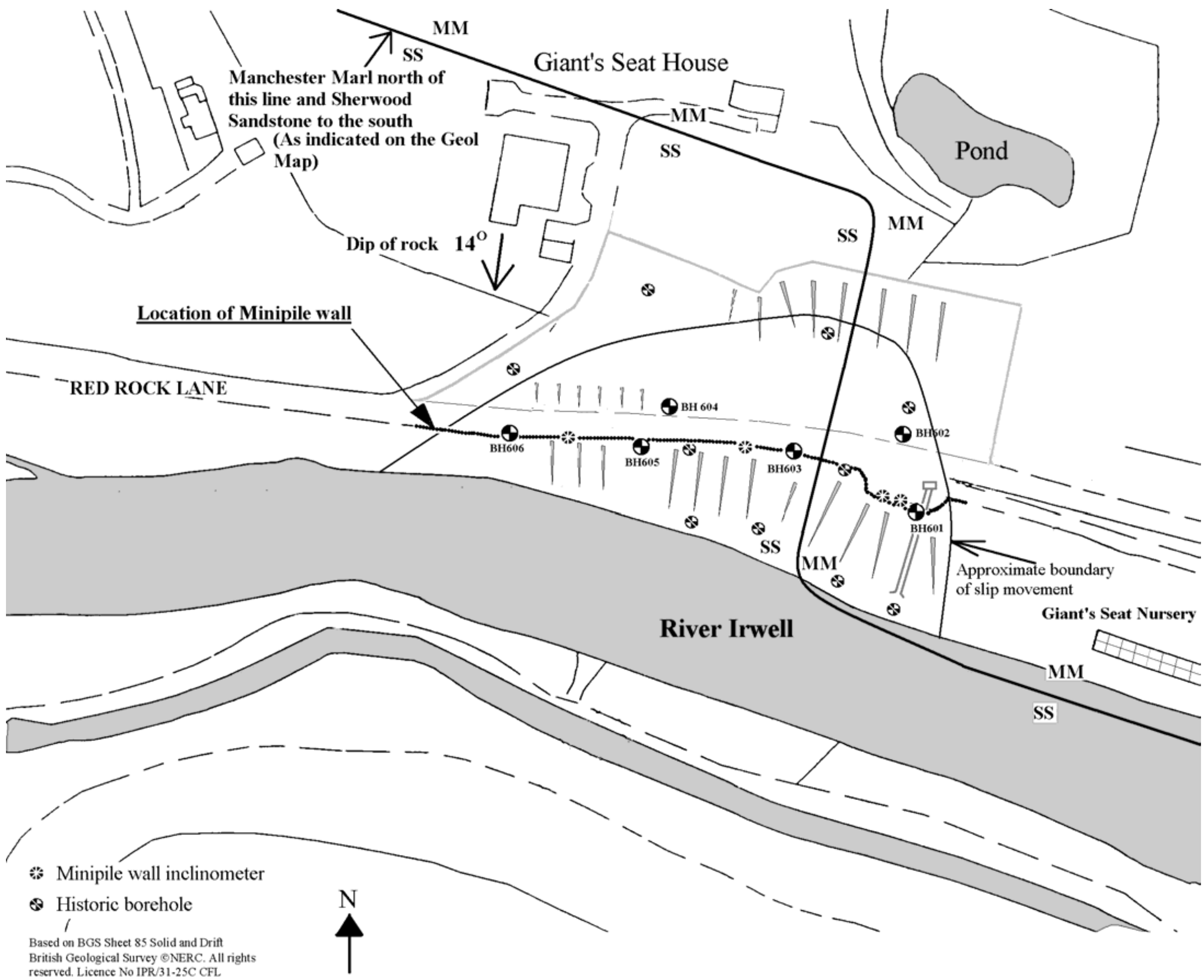

Fig.2. Site Location indicating the area of slip movement, the location of boreholes and the line of the stabilising wall.

crucial early phase in geotechnical work is the desk study. This can provide essential base information with regard to the site topography, geology and history of land use.

Site Geology

The British Geological Survey maps covering the site, Sheet 85 solid and drift (scale 1:63,360) and a 'geological diagram' produced for the Bolton Regional Sewerage Scheme (dated 1949), indicate the following geological succession:

Alluvial Deposits	- located to the south side of the river
Glacial Clay	- located to the north of the site area
Sherwood Sandstone	- south of the river underlying Alluvium
Manchester Marl	- noted to be underlying part of the site and identified as 'formerly exposed red Marl'.

Earlier maps from 1826 and 1929 have slightly differing boundaries to the extent of the Sandstone that overlies the Manchester Marl. It maybe that river erosion and progressive movement of the slip down slope had made mapping and interpretation from outcrop more difficult.

There is reasonable agreement from all three maps that the Marl and Sandstone to the north east of the river dip at about 12-15^{o} in the direction of the river.

The term marl has been used to describe a variety of materials although it is the common description for rock with 35-65% carbonate and a complementary proportion of clay. However, in the UK material referred to as marls often have a substantially lower proportion of carbonate material (commonly less than 20%) and may be more aptly described as marly clays or mudstones; in this paper this is the type of material described as marl.

Evidence of past mining in the area is apparent with former clay pits shown to the north of the site area and a colliery to the south side of the River Irwell.

History of Land Use

Prior to the Industrial Revolution the site area was rural. The late eighteenth century was a period of rapid development of the canal system in order to serve the newly establishing industries. The location of the site was one of the key areas for such development and along the Irwell Valley two Canals were constructed. One on the south bank was called Fletcher's Canal that provided a link from a colliery to the larger Manchester Bolton & Bury (MBB) Canal that ran on the north side of the Irwell valley and through the site. Both these are seen on Fig. 3. based upon the 1907 map that also shows the location of several structures to the south of the MBB canal not seen on later maps.

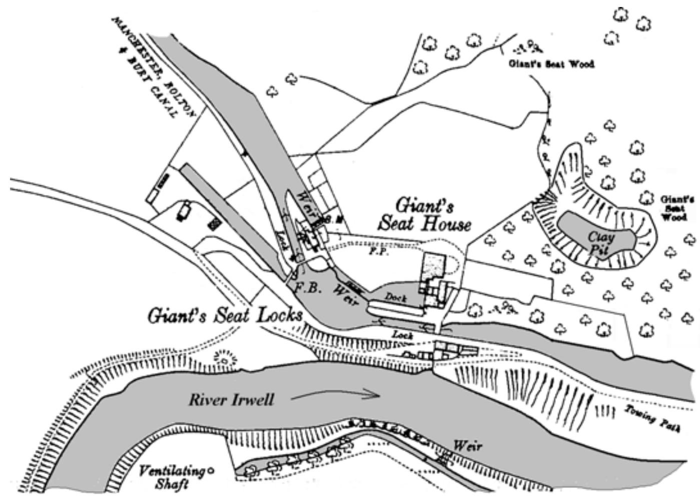

Fig.3. Site map in 1907 showing the MBB canal (site within circle)

Also seen on the 1907 map is a Clay Pit. Such pits can commonly be found close to the canal system associated with puddle clay sealing of the canals. However, in this case, based upon the chronology of maps the clay pit significantly postdates the canal. A pipe, perhaps overflow, ran from the water filled former clay pit to the north side of the canal. On later maps the clay pit is identified as a pond.

The canal remained operational from the 1790s until the 1960s when the British Transport Commission classed it as "a waterway having insufficient commercial prospects to justify their retention for navigation". In reality the canal had a history of significant bursts and two in the 1930s were significant enough to cause a dramatic drop in water traffic. From this time onward various lengths were infilled, some during the second word war and additionally during post war periods. The bursts are of interest with regard to slope stability assessment. The canal was generally built side long to the valley side with cut to the up slope generally forming fill on the downward slope that thus created the channel for the canal. The closest burst to the Giant's Seat failure was about 2km to the southeast. Contemporary reports from the 1930s suggest that seepage of the canal water caused softening at the soil/rock interface leading to failure of the down slope fill side of the canal and causing the breach.

Since backfilling of the canal in the vicinity of Giants Seat a road was constructed between the line of the canal and the river. Further fill was added to make up the road to this level along with construction of some sandstone walls to retain the fill and road. Additional uncontrolled tipping appeared to have taken place on the slope down from the road to the river predominantly at the east end of the site.

Utility providers took advantage of the line of the previous canal and along alignment installed a sewer pipe and sludge main in part cut and part fill construction. With increasing development local supplies of power, water and telephone lines utilised the road corridor for their routings.

DEVELOPMENT OF THE FAILURE MECHANISM 'MODEL'

Formal and informal monitoring of the slip movement, repair of the assets, consideration of desk study information and direct investigation has been ongoing over the past 30 years. It is convenient however, to consider three main phases of work in the 1970's, 1980s and then the late 1990's.

In the 1970's a limited site investigation proved the thickness of the filled ground below the road level and also, significantly, that the underlying rock was substantially Marl. Of the earlier geological maps, that of 1826 appeared to offer the most accurate representation of the conditions encountered. However, the marl passes by 'transition' in to the overlying sandstone and so at the top of the marl increasing frequency of marly sandstone and sandstone bands can be anticipated. The postulated failure mechanism was sliding at the interface of the Fill and the Marl. The potential for a temporarily high water table due to river levels or heavy rain was observed to reduce the factor of safety (FoS), although factual records of groundwater levels did not suggest that such high levels occurred. It was also considered that water could cause softening of the marl. An interesting observation based on 'local knowledge' was that the onset or increase in rate of slip movement occurred after periods of wet weather.

By the early 1980's it was apparent that some additional drainage measures had not had curtailed the slip movement and further repairs to cracking and settlement of the road and damaged services had been required. A further phase of field walkovers, site investigation and reporting was undertaken. At this stage the possibility of a long established, deeper seated, failure surface was suggested. The theoretical and site investigation work concentrated more closely upon the geometry of the marl beds, their description and nature in cores and their material properties.

Additional groundwater level monitoring was undertaken and a review of the available ground movements collated by the utility provider was initiated. The site investigation involved the installation of several piezometers and shear tubes and obtained cored and undisturbed samples of the marl. Detailed description of these samples indicated the presence of 'slicken' or polished surfaces, shear and slip surfaces. These details were used to assist in development of the likely depth and extent of the failure surface along which movement had occurred and to prepare a contour plot of the failure surface. Detailed analysis of the rock cores by the Institution of Geological Sciences indicated the likely presence of a NNE-SSE fault that was likely to limit the eastern boundary of the slip.

Site inspection indicated a potential zone of heaved material within the river channel at the eastern end of the slip. As the slip was on an outward bend of the River Irwell it was considered that removal of toe material and previously higher water table levels had potentially initiated slope failure. It was considered that as this movement had occurred over such a long period of time marl shear strength parameters would have been progressively reduced to near the residual condition at the shear surface. Two ring shear tests were undertaken which provided results of $\Phi'r$ of 13.7 ^{o} and 15.5 ^{o} with respective values for C'_r of 7.5 and 0 kN/m^2. Although the dip of the rock in the slip zone is relatively shallow, about 14^o, it can be seen that quite small changes in the level of the water table could initiate the onset of movement if the marl at the shear surface was at or close to residual strength values. Such a mechanism would fit that described from local knowledge noted in the 1970s.

In both the 1970s work and that in the 1980s the movement of the slope during the site operations was reported. However the degree of movement was not so clearly established, although the surface movement in the 1980s was in the order of 5-6mm/month.

At the completion of the 1980s phase of work some discussion took place with the utility owner over potential risks and actions. Having identified a link between increase in groundwater level and slip movement, the possibility of improved drainage was identified as an option. However, the measured piezometric levels were little above the river water level and indeed much of the slip surface was below the level of the river. This was discounted as likely to provide only marginal benefits. Previous drainage measures had also not had a marked effect on stability. Toe weighting was considered, although the geometry of the site was a constraint and much of the additional load, if it were to provide maximum benefit, would impinge on the river channel and probably influence flows.

It was considered that, due to probable impracticability and inefficiency, conventional slope stabilisation were unlikely to meet the requirements for this problem. Direct enhancement of the shearing resistance along the slip plane was likely to offer the best means of restoring long-term stability to the mass that had slipped. Piles were considered, although potential slip down slope of any shear key piles would leave the possibility of them needing to work in cantilever. This, it was considered, would potentially require a larger pile size than that simply required for increase in shear resistance. It was advised that this approach would entail considerable expenditure and major disruption during the work that would require road closure.

The alternative offered at this stage was to provide a seating for the pipelines that would offer the possibility of jacking and adjusting their position in response to slip movement and to avoid future bursts and leakage.

Neither of these two options was adopted. However, advice to continue monitoring of the slip movement was followed, not least as the assets remained vulnerable to future movement and the possibility remained, although considered unlikely, of a large dramatic failure.

By 1995 continued movement of the slip had caused further problem with the pipelines and significant settlement and cracking of the road. Several phases of repair work had been undertaken.

Further review of the problem involved:

- Design of a final phase of investigation sufficient to confirm specific data relating to the slip such as the depth of shear surface and rate of movement. The extent and nature of the site investigation was to be suited to the anticipated geotechnical process selected for stabilisation.
- Establish instrumentation that could be used to monitor slip movements before, during and after stabilisation works.
- Develop an evaluation report of stabilisation options utilising all available information and data.
- Following these stages the preferred option was to be developed to a full design.

The options for stabilisation were considered in parallel to the design of the additional site investigation. The site investigation included further cored boreholes and installation of five inclinometers. The depth of investigation was intended to intercept the likely failure surface and the instrumentation was to detect the depth and degree of movement of the slip.

Later sections of the paper, indicate that the instrumentation was used to accurately establish the depth of the failure plane within the marl at the selected locations. The groundwater level within the marl was found to be between 0 and 2m above the river level (Approximately 34mAOD).

Summary of Failure Model

From evaluation of the desk study and site investigation information the following summarises the factors that are considered to have potentially contributed to slope instability at Giant's Seat:

1. Removal of slope toe material by the River Irwell causing initial instability
2. Further removal of slope toe material by the river causing continued instability and increased movement.
3. During the period of high river levels, the strata would become saturated with water. Unless able to drain out again quickly, keeping pace with the subsequent falls in river level, the effective weight of the strata overlying the potential failure plane dipping towards the river was temporarily increased.
4. Slipped material and debris would be developed and shear surfaces thus formed within the marl as a result of the large strain movements involved would tend to gradually reduce the soil parameters (cohesion and angle of shearing resistance) to their residual values.
5. Construction of the canal in the 1780-90s provided a potential source of seepage water, potentially percolating through sandstone bands within the marl.
6. Additional weight of fill forming the down slope bank of the canal and towing path.
7. Additional weight of fill from backfilling of the canal and overfilling to the pipelines.
8. Seepage water introduced into the area of canal backfill from the pond overflow pipe.
9. Additional weight of fill from build up for the road construction.
10. Uncontrolled filling adjacent to the road.

Many of these changes in the distribution of loads and pore water or seepage pressures whilst in themselves being relatively small, could nevertheless have been of significance in the state of critical or limiting stability which probably existed at various stages in the development of the slip.

Figure 4. is a section through the site from the river in the south to the pond in the north. The significant depth of the fill materials above the marl can be seen. The base of the fill was initially believed to be the possible failure plane. The location of the pond and overflow pipe is indicated along with the 'design' slip surface in the marl. The approximate zone where various tension cracks had been noticed over the years has also been illustrated.

DECISION TO STABILISE THE SLOPE AND SELECTION OF THE OPTIMUM TECHNIQUE

A number of factors prevailed in arriving at the decision to stabilise the slope. The slip movement was continuing and further repairs to services and the road were likely and a more permanent solution would allow the adoption of a routine maintenance approach. Re-routing of the pipelines was not possible within the current land ownership and would not solve the potential problem of future road failure. The possibility of accelerated or more severe failure, although considered a low risk, could not be ruled out. The consequences of a burst in the pipelines and any disruption at the treatment plants were becoming more severe.

Consideration was given to selection of the optimum form of stabilisation measure. Prior to detailed design three options were developed to a level that allowed comparison of the major factors considered in selection of the design solution. These factors were Certainty of Success, Durability, Constructability, Schedule/Programming, Environmental Impact and Cost.

The three methods considered to this level were:

- Toe weighting with fill and regrading
- Soil Nailing
- A Mini-Piled wall along with raking tension piles/anchors

The toe weighting was to involve construction of a 3m high stone bund retaining additional benches of fill. The second and third options followed the approach of providing direct improvement at the shear surface. The minipile wall was envisaged to provide one long shear key across the failure surface. There is often debate as to the exact load transfer mechanism of soil nails although in this case it is reasonable to consider that they would offer tensile reinforcement across the failure surface and in addition some additional shear resistance would be provided. For the preliminary assessment of the options a 120m long piled wall was considered along the side of the road closest to the river. The concept considered for the soil nails involved installation of about 1000 nails both up and down slope of the existing road.

Whilst it was considered that each of these options could be engineered to provide an improvement, the mini-piled wall was determined as the preferred option. There were considered to be distinct advantages with regard to the degree of existing tree and woodland clearance, the likely achievable improvement in factor of safety and a perceived lower risk strategy. It was also decided that using this option would allow the road to remain operational, at least as single lane, throughout the work.

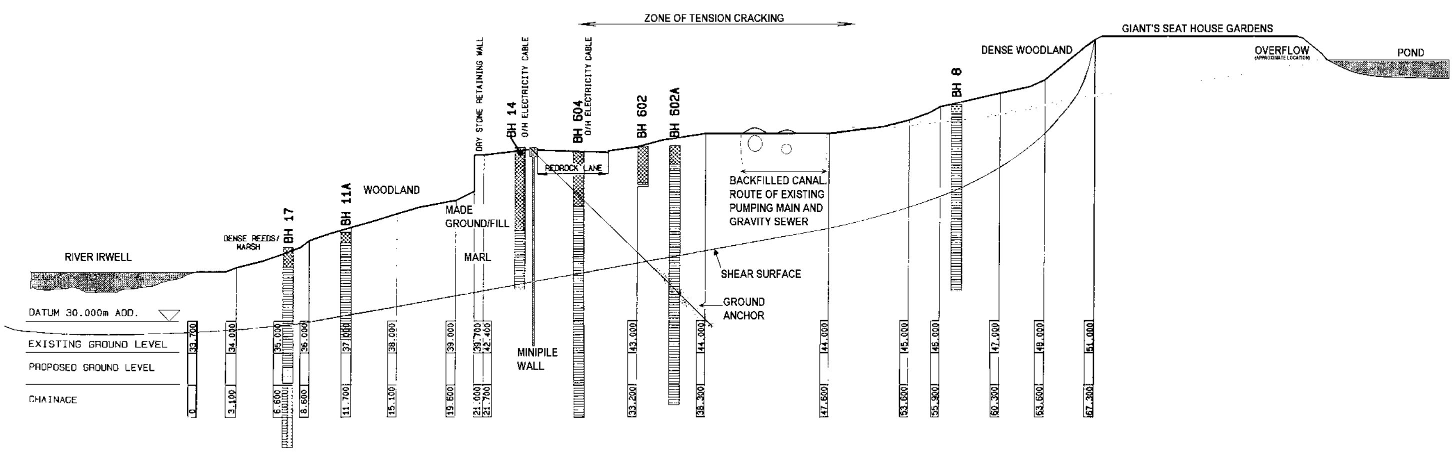

Fig. 4. Section through the Giant's Seat Landslip (the design slip surface is indicated as are the approximate locations of boreholes, mini-pile wall and anchors)

SLOPE STABILITY AND THE DESIGN OF THE MINIPILE WALL

Having identified the Mini-piled wall as the appropriate option, two phases of design were undertaken. The first involved consideration of the overall slope stability and the effect of the wall as a sheer key. The second stage concerned the design of the wall itself.

Overall Stability

It had been identified that the crucial factors likely to be influencing the slope stability were the residual soil shear strength parameters and the ground water level and, in particular, temporary rises in the ground water level.

In preparing the slip model for analysis, information regarding the depth of failure surface was provided by the instrumentation installed in 1996. Figures 5. and 6. illustrate the inclinometer results in two of the boreholes next to the road alignment. The lines on the plot represent regular readings indicating progressive increase in deflections over a period of about 8 to 9 months. A clear displacement of the ground can be seen just above the failure surface.

Along with the depth of the failure surface a design value of the residual shear strength parameters was decided upon. Two ring shear tests in 1995 supplemented the earlier tests and provided results of Φ'r of 12.5 ^{o}, similar to the earlier work, and 31 ^{o}. The samples had been taken to represent the marl, but examination of the last sample of material revealed that there was much fine sand in the sample and this is the probable cause of the single high result. A representative value of Φ'_r of 15 ^{o} was taken for design with C'_r of 0 kN/m^2.

For the groundwater level some further assumption was required. All ground water level monitoring indicated a relatively low piezometric level and the base design case was to set ground water level in the proximity of the river at the river level (34mAOD) rising to about 1.5-2m above this level about 50m away from the river. Highest flood level at the river from historic records was 37.5mAOD. For the worst-case condition, a level of 44mAOD was selected at about 50-60m from the river representing water almost at ground level at the location of the backfilled canal. This model was taken to represent a case following poor weather accompanied by highest discharge from the pond into the area of the old canal. It was after such conditions that past movements had accelerated.

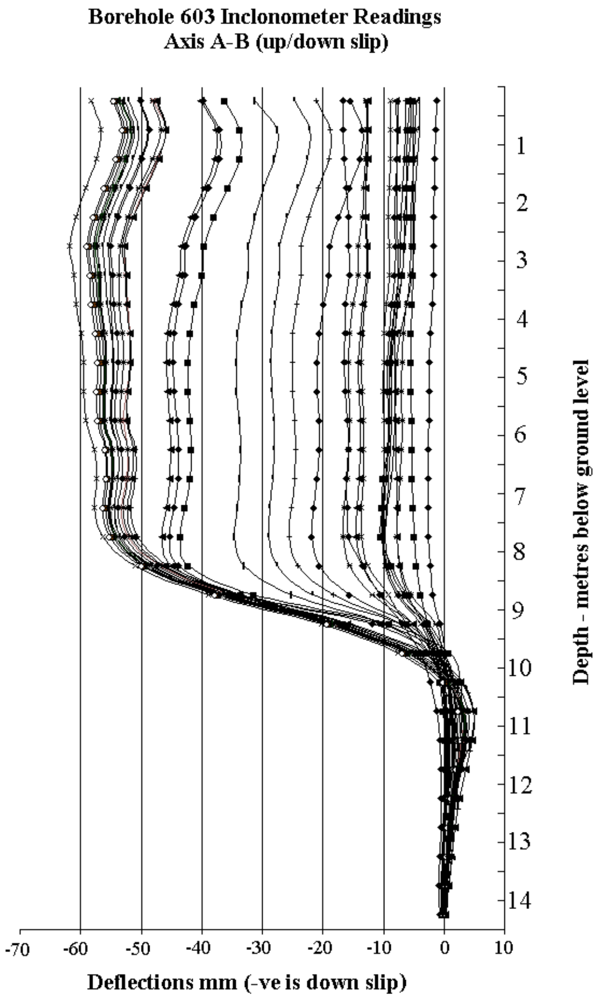

Fig. 5. Inclinometer readings over 9 month period BH603.

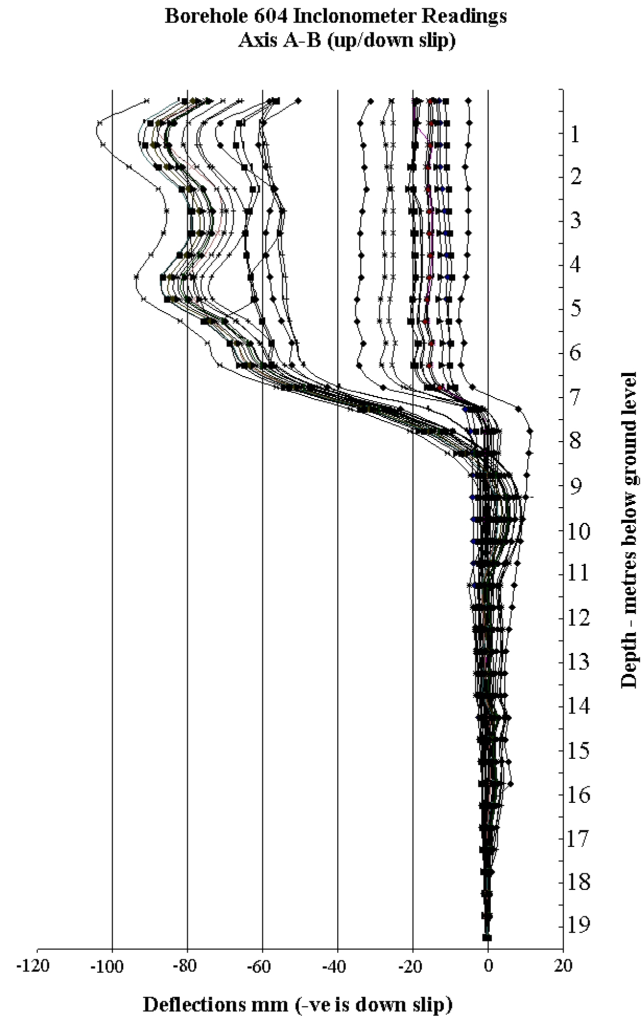

Fig.6. Inclinometer readings over 9 month period BH604.

The slope stability package STABLE by MZ Associates, allowing Bishop circular and later Morgenstern and Price and Sarma non-circular methods, was used for the analyses.

Analysis of the failure plane at various sections through the slip was undertaken and one of the sections is included as Fig. 4. At each of these sections four main design cases were considered:

- The base case groundwater level close to the river level (GW – Low).
- High temporary groundwater level close to ground level at the location of the backfilled canal (GW – High).
- The GW – High condition with the benefit of the wall constructed and providing a shear key.
- The Portion of the slip downhill from the wall analysed with GW – High.

The last design case assumes that the uphill portion of the slip is stabilised and the assets protected although the lower slope receives no direct shear enhancement. Improvement of the lower slope factor of safety is mainly achieved through a change in the balance of driving forces (reduced by the provision of the wall) to the available resisting forces (unchanged). A lower factor of safety was considered acceptable for this portion of the slope than that for the slope as a whole as certain other design conditions were modelled I the detailed wall design phase. The stability results are summarised below:

Table 1. Summary of stability Analysis Results

Average Results of Stability Analysis				
	GW-Low	GW-High	GW-High With wall	GWH & wall. Lower Slope
Factor of Safety	1.38	0.95-1.0	2.05	1.29

It can be seen that in normal 'dry weather' condition of the slip is stable but that an increase in groundwater level causes a significant decrease in the factor of safety to a marginal level of instability. However, in this case it is the relative reduction in FoS that is significant, as is the relative increase that can be seen provided by the construction of the wall in the model. The slip downhill from the wall under high groundwater level conditions was considered to have a satisfactory FoS.

Wall Design

In terms of the slope stability the main purpose of the wall was to provide shear enhancement across the slip plane. From the overall stability analysis a shear force of 650kN per metre of wall was required to provide the factors of safety detailed in the previous section. In order to satisfy the space constraints of the site, and to minimise temporary loading on the slip, one hundred and seventy 220mm diameter mini-piles were selected for construction of the wall at 600mm centre to centre spacing. To resist the shear force, at about 11m down the pile shaft, the selected full-length reinforcement was a 140mm OD steel circular hollow section grouted with a sand and cement grout inside and outside the tube. Embedment beyond the failure plane ranged from 4-10m. The sensitivity of the slip to build up in water level was carefully considered and it was specified that the wall should include 'gaps' between the piles equivalent to 30% of the length of the wall evenly distributed along the wall length. The permeability of the gaps between the piles was to be maintained to at least the prevailing value.

Thus the wall provides the improvement in shear capacity at the slip plane. However, although the FOS for the lower slope below the wall was improved it was recognised that it was lower than the overall FOS. An additional design case was considered assuming that down slope of the minipile wall the ground moved away (either as failure or tension crack) from contact with the wall to a depth of 4-5m. A capping beam with 40 ground anchors was also included in the design although it was recognised that the anchors would not need to work at their full design capacity unless the specific design condition arose in service. All anchors were to be tested to prove their design capacity. The 32mm double corrosion protected ground anchors had a design fixed length of 10m and a free length 15m and were designed at 2.4m centre-to-centre spacing. The location of the wall capping beam and anchors is indicated on the section in Fig. 4.

The wall was modelled using the OASYS programme FREW. This is a programme used to analyse the behaviour of flexible retaining walls and appropriate in the case of the mini-piled wall with passive (not substantially post tensioned) ground anchors requiring the modelling of a number of stages/design cases.

Inclinometers were been installed in the investigation boreholes. The design of the permanent works also incorporated the facility to install inclinometers in the piles, and in the gaps between, in order to monitor the deflections of the completed structure.

CONSTRUCTION AND SLIP MONITORING DURING CONSTRUCTION

The inclinometers installed in the last phase of site investigation had clearly identified the failure surface and confirmed that movement was ongoing. In these circumstances the continued monitoring of the slip during and after completion of the stabilisation work was considered an essential aspect of the project.

In addition to the wall the work required reconstruction of the road over a length of about 130m although to minimise the additional load added to the slip the surface was to remain at the settled level and regraded to smoothly link with the road outside the slip zone. The contractor's programme involved a staggered approach to pile construction to increase in shear capacity over a wide area early in the construction phase.

Rotary drilling and down hole hammer work with air flush were used to form the piles and anchors (Fig. 7.). These techniques can have a detrimental effect on the friction capacity of pile and anchors in marly ground due to smearing and the air flush can be potentially disruptive to shear surfaces. However due to the sensitivity of the slip to groundwater levels it was considered that carefully controlled

Fig. 7. Anchor installation work from temporary staging (Mini-pile wall capping cage visible in foreground)

air flushing techniques were preferred in this case. None of the potential difficulties with this flushing medium in the site soils were encountered.

A particular challenge of the work at this site was to achieve a balance between maintaining adequate grout cover to the pile steel and ensuring that the 'permeability' gap between the piles was maintained and not grouted as a consequence of pile construction.

In addition to the inclinometer checking during the construction phase the ground water levels were also monitored. Construction work commenced in March 1997. Three to four later a significant rise in groundwater level was noted in the piezometer in BH603. This borehole is on the uphill side of the wall. Over the same period it was noted that the rate of slip movement increased from about 15mm/month to about 28-40mm/month. Figure 8. shows the cumulative movement of the inclinometers at a specific depth. The change in slip rate can be seen occurring from about May1997 onwards. Included on the same plot is the relative behaviour of the piezometer in BH603 showing the rise, of about 2m in water level, over the same period of time.

to the steel in the pile. In order to prevent further groundwater level increase, and if possible reduce it, the gaps between the piles in the vicinity of BH603 were drilled and the gaps filled with gravel.

It can not be determined conclusively that the increased failure rate was due to increased groundwater levels, recorded in one of the piezometers. However, the two occurred at the same time and it had been established theoretically, and through historical reports, that the slip was very sensitive to groundwater level fluctuation. Furthermore, from Fig 8. it can be seen that, after the work to improve gap permeability, the groundwater level BH603 rapidly returned to its steady level and this is followed by a slowing of the rate of movement of the slip. Again these events are very likely to be connected although, at this stage of the project, the rate of slip movement was also being slowed by an increasing number of installed piles. Figure 9. shows the piezometer readings from the boreholes monitored during construction

The rate of slip movement began to slow about 3 months into the construction of the wall and, as can be seen form Fig. 8. had effectively been curtailed after 4-5 months of pile

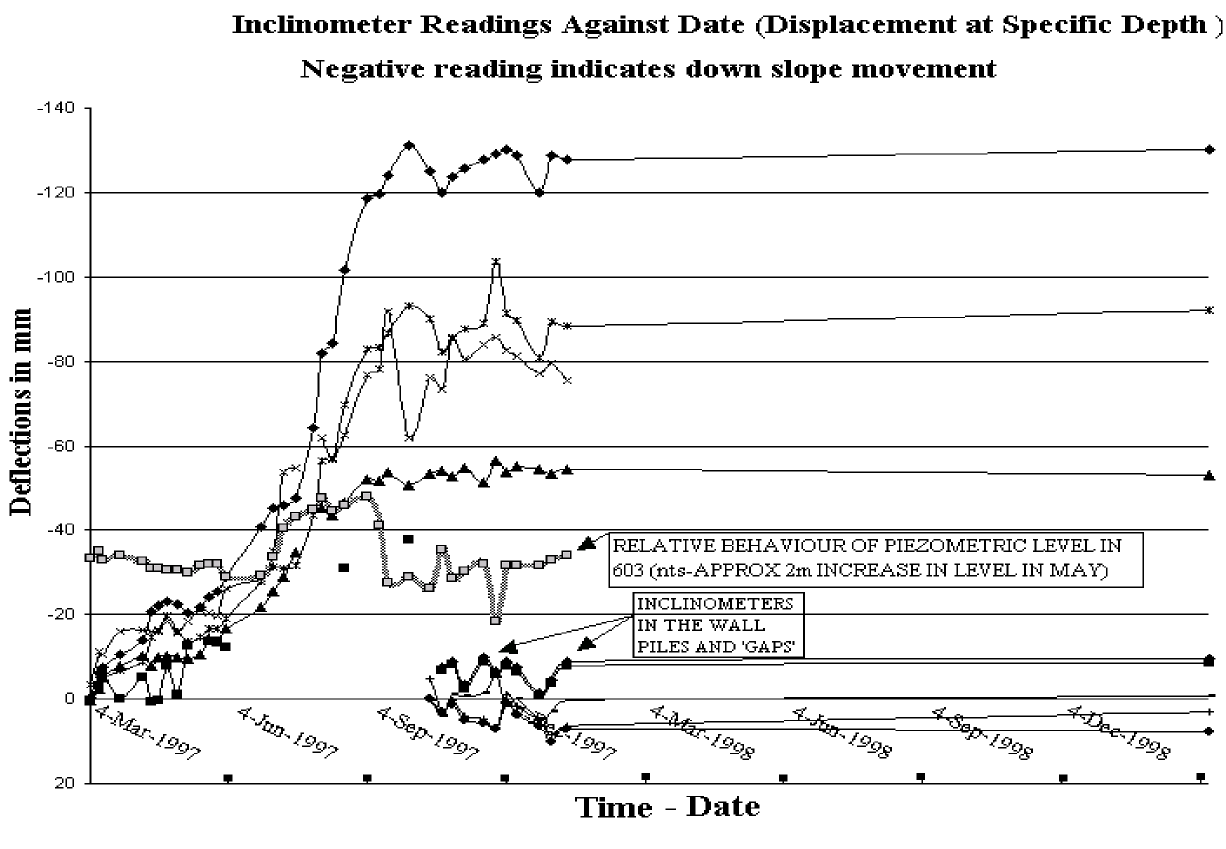

Fig. 8. Slip movement against time indicating change in rate of slip due to groundwater level rise and curtailment of movement at completion of piling in late June early July 1997.

Permanent sleeves were included in the pile shaft through the fill materials to control the grout seepage and maintain cover

installation. This was also the stage that reading could commence in the inclinometers installed in the piles and gaps of the wall. From June to the end of October 1997 no significant movement was recorded within the remaining site investigation borehole inclinometers or those inclinometers installed in the piles and gaps. Groundwater levels also remained steady over the same period. A further reading of the instruments late1998/early1999 confirmed that there had not been movement of the slip mass since the installation of the wall.

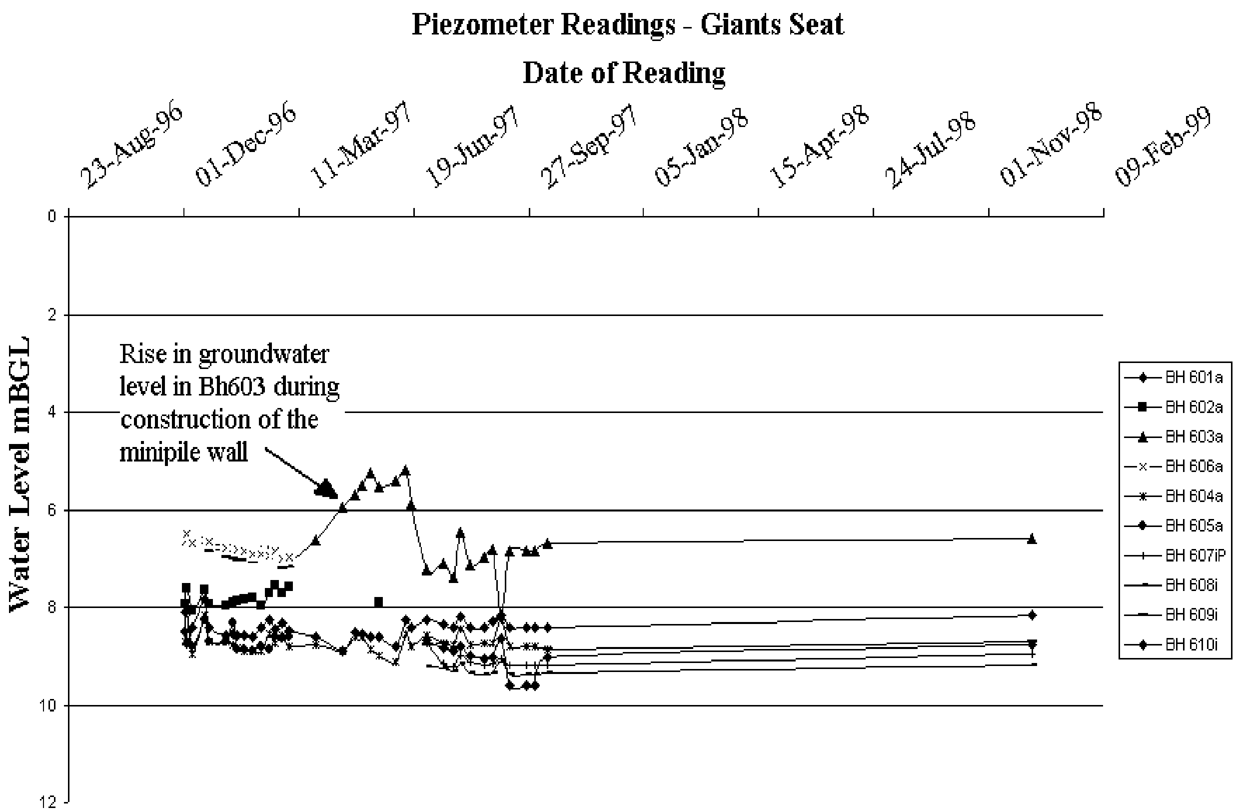

Fig. 9. Groundwater level monitoring from piezometers in the slip area before, during and post construction of the mini-pile wall.

In 2003 a visual inspection of the site was made. In years prior to the wall installation regular repairs had been required to the road and services. In 2003 the inspection revealed no cracking or distress to the road surface and no reports of damaged services had been recorded. Significantly, inspection downhill from the wall, the area not directly supported by the shear enhancement, showed no tension cracking or down slope movement away from the front of the minipile wall. This indicates that the consequential improved FoS had also provided the required improvement in stability in this in this zone.

SUMMARY AND CONCLUSIONS

The case history of the Giant's Seat Landslip shows clearly what the key role that geotechnical engineering can take in the management of ground risk and protection of assets. The desk study information gathered allowed detailed consideration of the slope stability problem and development of the optimum solution for stabilisation. The site investigation was focussed to provide the data required for design of the mini-pile wall. The instrumentation provided invaluable information on the location of the slip surface and the rate of movement of the slip. It was also a valuable tool enabling the monitoring of change in the slope behaviour during the construction phase and to confirm performance of the stabilising measures on completion.

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