Stabilization of a Landslide on the M25 Highway London's Main Artery
Estabilización del Deslizamiento de Tierra en la Autopista M25, la más Transitada de Londres

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Abstract
This paper describes the lessons learnt in the assessment, stabilization and subsequent monitoring of a landslide, which mobilized over 90,000 m^3 of material and threatened to completely close England's busiest highway, the M25 around London. The wedge shaped slide occurred on the 19th December 2000 and extended some 80m up the slope with a slip surface up to 10 m below the ground surface. Due to the strategic importance of the M25, the design and construction was fast tracked to be completed before the fall rains when further movements would be inevitable. The adopted solution used a combination of 1050 mm diameter augered piles, a deep cutoff trench and counterforts at the toe and was extensively instrumented. The collected data for the winter 2001/02 demonstrates that not only were the remedial works successful in stabilizing the slope but that the proposed design method can accurately predict the bending moments and forces produced in the piles.

Resumen
Este artículo describe las lecciones aprendidas en la evaluación, estabilización y subsiguiente monitoreo de un deslizamiento de tierra, el cual movilizó más de 90,000m^3 de material y amenazaba con cerrar por completo la autopista más transitada en Inglaterra, la M25 alrededor de Londres. El deslizamiento en forma de cuña ocurrió el 19 de diciembre de 2000 y se extendió aproximadamente 80m hacia arriba con una superficie de deslizamiento de hasta 10m de profundidad. Debido a la importancia estratégica que tiene la M25, el diseño y la construcción de la solución fueron adelantados rápidamente para evitar las lluvias, las cuales habrían generado desplazamientos adicionales. La solución adoptada usa una combinación de pilotes taladrados de 1050mm de diámetro con una trinchera profunda con contrafuertes en la base. El sistema se instrumentó ampliamente. La recolección de datos para el invierno de 2001/02 demostró que no solamente la solución implementada fue exitosa en estabilizar el talud, sino que también el método de diseño propuesto predice acertadamente los momentos flexionantes y las fuerzas generadas en los pilotes.

1 INTRODUCTION

On the 19th December 2000 during the wettest winter in English history, a 200 m long section of Flint Hall Farm cutting failed and threatened to close England's busiest highway, the M25 around London, which carries over 120,000 vehicles a day. The failure extended from the hard shoulder for 80 m upslope and mobilized over 90,000 m^3 of material. During January and early February further rainfall triggered additional movements. Given the location of the failure a fast track program of investigation, design, construction and monitoring was instigated in order to stabilize the slope before the wet fall weather returned and further movement occurred.

2 SITE DESCRIPTION

Flint Hall Farm Cutting was constructed between 1976 and 1979 and lies to the south of

London immediately to the east of Junction 6. The cutting takes the Highway through the lower slopes of the North Downs escarpment and is crossed by Flower Lane Bridge. The slope stands at between 11 and 15 degrees with a maximum height of 25m and is considered an area of outstanding natural beauty.

3 INVESTIGATION & MECHANISM

Immediately after the failure, 16 pairs of wooden stakes were installed across the toe and backscarps and their separation measured to determine if the slip was still moving. These pegs showed that the slip was marginally stable and responded rapidly to rainfall events with further movements as shown in Figure 1.

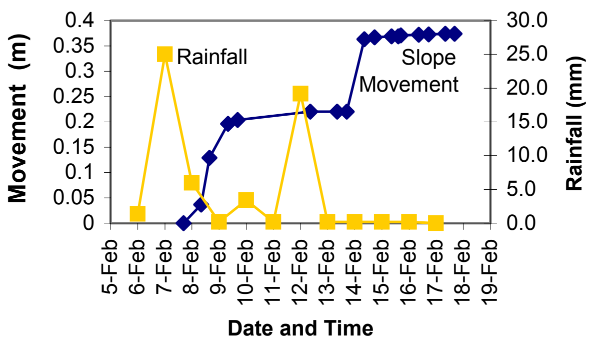

Figure 1 Rainfall & Slope Movements from Pegs

The first stage in the investigation was geomorphological mapping. This showed that the failure was of greatest extent to the east of the bridge as shown in Figure 2.

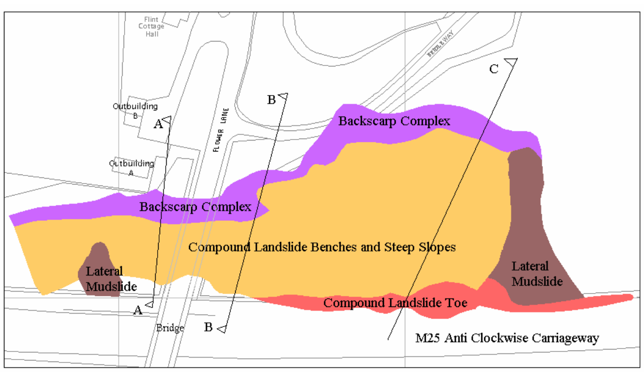

Figure 2 Geomorphological Map

In order to investigate the cause of the failure, data was obtained from a local water-monitoring borehole in a Chalk aquifer. This showed that the groundwater level reached an historical high in December 2001 exactly when the slope failed. Although this data relates specifically to the Chalk, it is considered to be indicative of the ground water level in the area of the failure.

The investigation revealed the slip to be within Divisions 9(iv) and 9(iii) of the Gault, Owen, (1976), which is a stiff to very stiff gray fissured clay, with overlying Head deposits mantling the upper portion of the slope as shown in Figure 3.

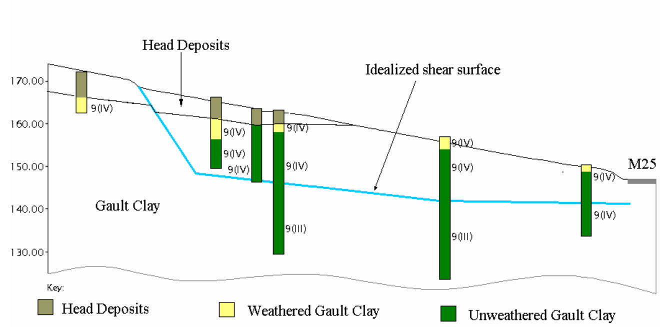

Figure 3 Geological Section

Up slope, the Upper Greensand strata and the Lower Chalk were encountered, from which much of the Head deposits are derived. For the purposes of design, the Gault was not subdivided by grade, but residual properties were applied as appropriate, and are given in Table 1. The Head deposits, being remoulded significantly during the process of solifluction also exhibited principally residual behavior.

Table 1 Geotechnical Materials

Material	Design Parameters		Typical Properties		
	c' kPa	φ'	Cu kPa	PI (%)	W/C
Head	0	14	50		40
Gault	1	24	100	45	35
Residual Gault	0	14	50		

During the course of the investigation a variety of instrumentation was installed to monitor slope movements including inclinometers, slip indicators and survey stakes. In order to monitor the ground water both shallow and deep piezometers were installed. The former displayed variable levels at or near the ground surface, whilst the deep piezometers were more consistent with levels between 2m and 6m below ground level. A dewatering trial consisting of 6 ejector wells was also carried out in the Gault. The results showed that the deposit could be effectively dewatered by up to 6m. Furthermore this confirmed the results of *in situ* permeability tests, which indicated significant fissure flow and permeabilities of between $5x10^{-6}$ and $7x10^{-7}$ m/s. However the trial did not affect the water levels within the head deposits demonstrating that the shallow and deeper water tables were in fact independent.

The inclinometer deflections combined with the identification of zones with increased moisture content indicated that the basal shear plane, was up to 10m below ground level. This combined with the observed steep backscarp, shown in Figure 4, and other geomorphological evidence, suggested a principally wedge shaped failure. Inclinometers installed to the south of the highway and the negligible deformation of the hard shoulder carrier drain, confirmed that the shear plane did not extend beneath the carriageway.

Trial pitting also exposed the Head/Gault boundary. This was found to be highly polished with an elevated natural moisture content suggesting that significant movement had occurred and that the two formations were acting independently.

Figure 4 Shear surface exposed in a trial pit

4 REMEDIAL WORKS DESIGN

From the outset of the project it was clear that the time available for design and construction was very limited. This meant that throughout the design, consideration was given to build ability and speed of construction. The main stages in the design of the remedial works were:

- Understanding the current condition and developing a design model.
- Development of a design philosophy.
- Establishing the restoring force, F_x required to achieve a desired Factor of Safety (FoS).
- Select a pile diameter, length and position within the slip.
- Calculating the required spacing, shear and bending resistance for the piles.
- Examine the stability of the slope above and below the wall.
- Establish the combined FoS.

4.1 Understanding the current condition and develop a design model

In order to design effective remedial works it was vital to understand what controlled the failure and be able to model it. Three sections were therefore chosen across the site; section A was to the west of the bridge, section B immediately to the east of the bridge and section C in the area of greatest apparent movement, as shown in Figure 2.

A model of the failed slope was then created in SlopeW by Geosolve, using the surveyed extent of the slip and the soil parameters derived from the site investigation and literature. By adjusting the watertable in line with the observed monitoring data the factors of safety where seen to vary in line with the field observations giving confidence in the parameters, mechanism and model. A summary of the lowest FoS are given in Table 2.

	Section A	Section B	Section C
FoS	1.01	0.98	0.93

Table 2 Lowest FoS for Current Conditions

4.2 Development of a design philosophy and best method

The stabilization of landslides of this size is not normally attempted as the costs are rarely justified. Consequently, there was little precedent and no nationally recognized design methodology in terms of desired factors of safety. The client, the Highways Agency did however specify a 60 year design life. This suggested that a design relying solely on drainage, which could block over time was not appropriate, especially as the counterforts installed during the initial construction had not prevented the failure. In addition to the time constraints, the lack of space and restrictions on the finished appearance ruled out regrading and the construction of a toe berm. The combination of limited access and a very deep slip surface also made any kind of excavate / replace impractical. A 'hard' structural solution to pin the sliding mass to the underlying Gault clay was therefore required. Following a Technical Options Review involving the client, consultant and contractor, a line of discrete bored piles emerged as the preferred option. This solution offered benefits of shorter lead times, lower cost and simpler access.

Given the uncertainties early in the project and after consultation with the client, it was decided to adopt a robust approach to the design. The piles were sized to provide a 20% increase in the overall stability of the slope ignoring the beneficial effects of any drainage. Improvements in the latter would then further increase the overall factor of safety, whilst also improving the stability of shallow slips. This approach meant that if the drainage blocked, shallow slips may occur but the whole slope would not be at risk.

4.3 Establishing the restoring force, F_x

By using the SlopeW model the additional stabilizing force, Fx, required to give a 20% increase in the FoS was determined and is shown in Table 3.

Table 3 Forces Required for 20% Increase in FoS

Sect	Current FoS	Target FoS (0.2 increase)	Force Fx kN/m width
A	1.01	1.21	52
B	0.98	1.18	222
C	0.93	1.13	301

4.4 Selecting a pile length and position

As the number of possible permutations for the pile design was large, a number of the variables were decided upon at an early stage of the design process. These included the position on the slope, which was fixed at 25m from the back of the kerb. This position is approximately one third of the way up the failure and reduced the possibility of a significant failure above or below the line of piles. It also had the added advantage that should the piling rig topple over the mast would not reach the highway. The monitoring suggested that the depth of movement along the proposed line of the piles would be approximately 8m and therefore the forces in the piles would be significant. These high forces required a high percentage of steel reinforcement and, in order to reduce potential problems with reinforcement congestion, the pile length was adjusted such that lapping of reinforcement would not be necessary. This led to the choice of 16m long piles with the top of the pile, 1m below the topsoil to enable the farmer to plough over the top of the piles.

4.5 Calculating the diameter, spacing, shear and bending resistance for the piles.

The spacing of the piles was dependent upon a number of factors including the diameter. Preliminary calculations showed that 0.75m diameter piles would need to be spaced at 1.2 times the diameter, 1m diameter piles at around 2.5 times the diameter and 1.2 m diameter piles at 3.5 times the diameter. The spacing of the piles is critical to the performance of the design as the design should maximize the amount of soil arching between the piles whilst minimizing the flow of soil between piles. It is generally considered that flow between the piles is possible at spacings above 5 diameters, Carder & Temporal (2000). All three options were closer spaced than these criteria. However the 750mm diameter piles were rejected on the basis that the spacing would not make full benefit of soil arching between them. The 1m diameter pile appeared to offer a number of benefits over the 1.2m diameter pile, including improved constructability and a greater confidence that soil flow would be minimized. This size was therefore adopted.

The design of the piles was based on the method proposed by Viggiani, (1981). By adjusting the sizes and capacities of piles it is possible using this method to control the ultimate failure mechanism. In the case of Flint Hall Farm cutting, a non brittle ultimate failure mechanism would be preferable to a brittle one and hence Viggiani's Mode C was considered to be appropriate. In this mode, the pile does not develop a plastic hinge and is fixed in the firm underlying soil. If the destabilizing forces are increased until failure occurs, the soil will flow around the piles as opposed to a more brittle failure of the pile itself. The detail design therefore used 16 m piles with the shear force (T_c) = restoring force (F_x) = 301 kN/m for an 8m deep slip which represents the most critical case, ie Section C. Note the bearing capacity factor within the sliding soil 'n_1' in this case is approximately 1.5 rather than the value of 4.0 sometimes assumed. The working load of the piles (M_w) was also restricted to the ultimate capacity/ 1.5.

Although the analysis suggests that the required stabilizing forces decrease as you move westwards the same pile design was adopted throughout. This was considered appropriate as this reduction in Fx was influenced by the pinning effect of the existing bridge piles. There were also signs of movement behind the bridge and by installing the same pile design in front of it, the new piles will stabilize any potentially deeper slips passing through or under the bridge piles and not just the current shallow failure in that area. The stabilizing pile design consisted of:

- Seventy four 16m long, 1.05m diameter piles installed at 2.5m centers through the main slope. The main reinforcement was 24 No 50mm, 14m long bars (yield strength 460 N/mm^2) arranged so that the top and bottom 2 m of the cages, with the least bending moment and shear, only had half the reinforcement quoted above. The shear reinforcement was 16mm bars at 0.3m centers.

- Fifteen 9m long, 310mm diameter piles installed at 750mm centers beneath Flower Lane Bridge. The main reinforcement was 8 No 16mm bars with 12mm bars at 150 mm centers as the shear reinforcement.

4.6 Design of the Drainage

The drainage was designed to lower the groundwater table on the slope and to consequently increase the FoS of the whole slope over and above the improvement provided by the piles, Hutchinson, (1984). The drainage also provided a minimum FoS above and below the piled wall of 1.2, which was vital as shallow slips still had the potential to close the highway.

At the top of the cutting, a horseshoe shaped filter drain up to 5m deep was installed to intercept groundwater from the Head deposits and any other perched water. This improved the stability of both the existing deep seated failure and any potential shallow failure in the upper one third of the slope. Where the filter drain crossed the backscarp and reverse scarp of the landslip, a flexible joint was installed in case of further small movements.

In order to stabilize the bottom half of the slope 3 m deep counterforts were installed at 6 m centers extending from a drain at the slope toe to between 20 and 23m up the slope. These counterfort trenches illustrated in Figure 5 also picked up the temporary pumped sumps, which had been installed in the slope toe as emergency works, to lower the water table and control the mudflows.

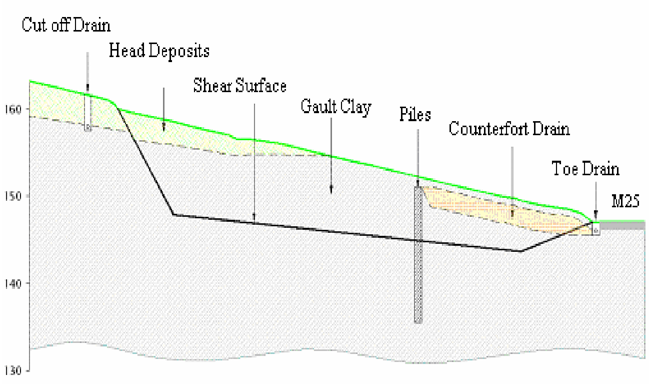

Figure 5 Schematic of Remedial Design

4.7 Establish the Factor of safety with both piles and drainage

Once the drainage design was complete it's effect was included in the model and the combined factor of safety determined and is shown in Table 4.

Table 4 FoS once Piles and Drainage Installed

	Section A	Section B	Section C
FoS	1.44	1.28	1.23

5 CONTRACT STRATEGY, CONSTRUCTION AND RISK

The client took the unusual step of using the Engineering Construction Contract Option E (Cost Plus) as the Contractor, Raynesway Construction would be on site before the design was completed. As previously mentioned the contractor was involved from very early in the project and a partnering approach was used throughout which was considered to be vital given the fast-track nature of the project. A good example of this approach is that throughout construction a member of the design team was always on the site, shown in Figure 6.

Figure 6 General view of the site from the bridge

Their role included amongst other things, gathering information to confirm and update the geotechnical model. For example water strikes in the pile bores confirmed the elevation of the main shear surface and that it become shallower to the west. Reverse shear surfaces were also identified in the toe drain trenches indicating the presence of a zone of compression associated with the toe bulge at the front of the landslide. This updated ground model was then used to analyze all the various construction stages to ensure that the stability of the slope was not adversely affected, something which as temporary works would traditionally be regarded as the contractors responsibility. Despite the resulting restrictions, the installation of the piles was complete on the 21st September after which the slope was regraded to its original profile, the subsoil broken up and the slope topsoiled and seeded.

As the presence of water triggered the failure, the ejector wells were maintained operational so that if necessary an increase in stability could be obtained by drawing down the water table in the Gault, as had happened during the dewatering trial. This secondary purpose had been considered before the trial and resulted in the wells being located in what was considered to be the most critical area of the slip.

The major residual risk not addressed by the design is a different landslide to the existing failure, which is only one of many potential landslide hazards on the Gault slopes in the area. Geomorphological studies have highlighted three such features in the direct vicinity of Flint Hall Farm, one of, which is active. In order to mitigate this risk in the area to the west of the failure, thirteen counterfort drains were installed to extend the drainage to a more natural end point.

6 POST CONSTRUCTION MONITORING

A number of instruments were installed and monitored to continually reduce any residual risk at the site by:

- Further increasing the understanding of the site behavior.
- Provide a system of trigger levels.

In addition the monitoring programme provided data for part of the Highways Agency research project with the Transport Research Laboratory (TRL) into the use of piles to stabilize slopes. The monitoring system comprised of:

- Daily rainfall records.
- Inclinometers installed in the slope within and outside the slip.
- Inclinometers in both the bored piles and between adjacent pairs of piles.
- Standpipe piezometers installed in the slope within and outside the slip.
- Vibrating wire piezometers installed within and outside the slip.
- Strain gauges installed within the bored piles.
- Displacement transducers and tiltmeters on the north abutment and central pier of Flower Lane Bridge.
- Leveling of the bulged hardshoulder.

This instrumentation was a combination of existing instrumentation installed during the investigation, supplemented by both replacements for those lost during construction and instrumentation installed by TRL. As the data was collected it was compared against predefined trigger levels, which were based on the expected behavior of the slope or the predicted load / deflection behavior of the piles. The trigger levels were based on a traffic light system and allowed for instrument accuracy and reaction time and if exceeded required a predefined action to be taken.

As the behavior of the slope is controlled by water one of the key records was daily rainfall and is illustrated in Figure 7.

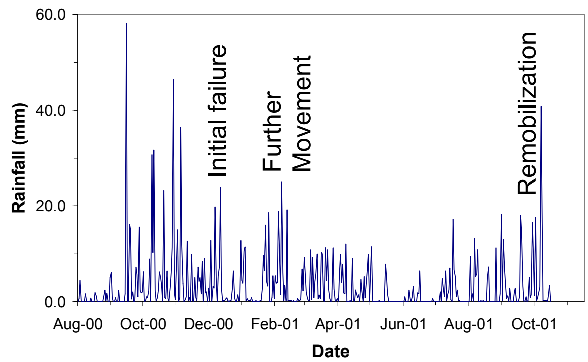

Figure 7 Daily Rainfalls from Kenley Airport

As can be seen there was a significant rainfall spike in early October comparable to the levels, which triggered secondary movements of the landslip during spring of 2001. This rainfall event and subsequent smaller events caused the perched water level in the Head deposits to rapidly rise whilst the lower water level in the Gault clay rose more slowly and stabilized after rising approximately 2m, to 8m below ground level. Subsequently the perched water level exhibited high amplitude and short wavelength variations in correlation to rainfall, but did not rise above 4m below ground level when water began to flow into the new filter drains, as shown in Figure 8.

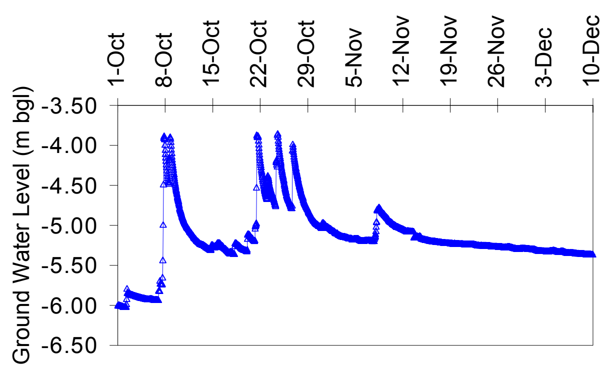

Figure 8 Water Level in BH22A

This rise in water levels as expected, caused the failed block to remobilize. Inclinometers I1 and I2 in piles 21 and 23 moved by 12mm at their head and down to a depth of approximately 9m as the piles mobilized their strength to carry the load. This pattern was repeated in the mid slope BH4 plot shown in Figure 9, which experienced approximately 13mm of movement along the slip plane. No movement was recorded outside of the landslip or in inclinometer I3 between piles 22 and 23, confirming that the pile spacing was sufficient to promote arching and prevent flow between the piles.

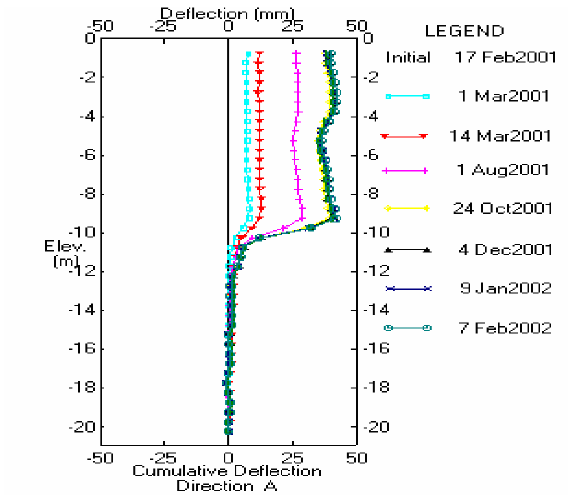

Figure 9 Deflection of Inclinometer BH4

The bending moments recorded in the two strain gauge instrumented piles showed a maximum of approximately 500 kNm at 9m below ground level. This correlates well to the deflection profiles and shear surface location and is well within the pile capacity. Since October 2001 no further movements or bending moment increases have occurred despite high rainfall in early February.

During the course of the works eight survey stations were installed along the kerb in the region of the toe heave. These points were then monitored and no further movement occurred including after the October rains. However, the survey results showed a settlement of this region by up to 50mm when the adjacent toe drainage trench passed by which is believed to be an expression of the release of lateral pressure in this area.

7 BACK ANALYSIS

Once all the data from the winter of 2001/2002 had been collected a back analysis was carried out to improve our understanding and establish how well the design method could predict the measured loads in the piles.

In October, the slope as expected remobilised and moved until the piles had generated sufficient resistance to provide a Factor of Safety of greater than 1.00 and halt the movement. The bending moment profile recorded by the strain gauges at this point was then compared against the bending moment profile predicted by the design method for various different values of Fx. As shown in Figure 10, the recorded profile closely matched the design profile for a Fx of 40kN/m.

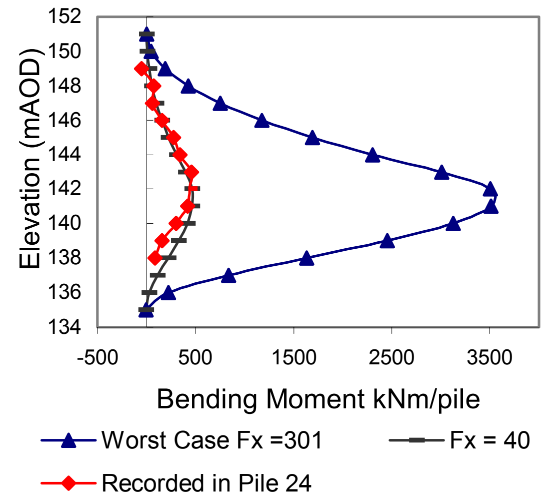

Figure 10 Predicted & Recorded BM's for pile 24

This Fx was then compared to Figure 11, which shows that under the recorded winter conditions which were less onerous than the two design cases also shown, the design model would predict a factor of safety of 1.06. This suggested that the combined instrument, design and modeling error was less than 6%.

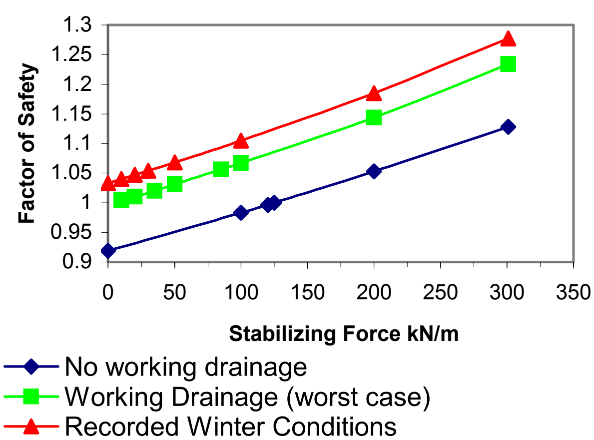

Figure 11 Fx vs factor of safety

During the creation of the model the parameter with the greatest uncertainty was the residual phi angle of the Gault clay as literature quoted values. of between 9 and 25 degrees, Forster et al, (1995). As shown in Figure 12, the phi value has a marked effect on the predicted factor of safety and the entire modeling, design and instrumentation error equates to approximately a 0.7 degree reduction in phi, which is considered to be within the accuracy of the site investigation.

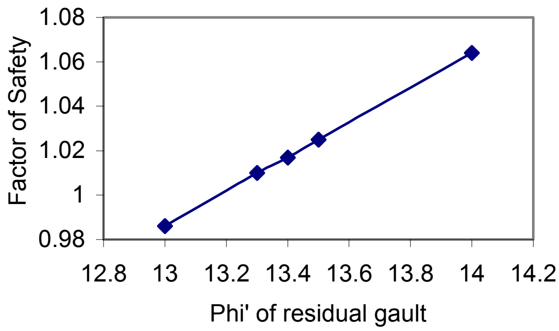

Figure 12 Effect of Reducing Phi'

8 CONCLUSIONS & RECOMMENDATIONS

The failure was clearly triggered by an abnormally high ground water table following exceptional winter rainfall. The pairs of pegs installed immediately after the failure were particularly effective in monitoring the early movement of the failure and showed that it responded rapidly to heavy rainfall with renewed movements. Given their low cost and speed of installation they offer a very cost effective and useful way of monitoring large movements and are recommended. Geomorphological mapping was also extremely useful in determining both the mechanism of the current failure and the general geological setting. This included the identification of other historical landslides and areas at risk, information, which is essential to the future management of the area.

A thorough phased site investigation was vital to the success of the scheme and as demonstrated the selection of appropriate soil parameters is critical to an accurate and effective design. Continual checking and development of the ground model as more information became available also gave confidence in the model and mechanism, as construction progressed.

The post construction monitoring has shown that despite heavy winter rain sufficient to remobilize the slip, the failure has been effectively stabilized. The subsequent back analysis has also shown that the ground model accurately represented the actual failure and that given known conditions the design method of setting Tc= Fx can accurately predict the loads carried by the piles. The back analysis also showed that the errors associated with selecting the soil parameters can easily be more significant than the combination of instrument accuracy, errors associated with idealizing a 3D failure to produce a 2D model, and assumptions in the design and analysis.

The dewatering trial clearly demonstrated that it is possible with ejector wells to lower the ground water table in stiff fissured clays. This principle although not exploited with this particular failure could be applied over a larger area to improve the stability. The implications of maintaining and operating such a system over any length of time would however suggest that this is best suited to a temporary solution.

The early Contractor involvement (ECI) and a ‘partnering approach’ throughout yielded a constructive and positive atmosphere and enabled the stabilization measures to be completed ahead of the demanding schedule and under budget.

9 ACKNOWLEDGEMENTS

The authors would like to express their thanks to all the other members of the site investigation, design, construction and monitoring team.

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