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# Assessment of slope failure and engineering behaviour of raised marine deposits of the River Clyde, Scotland

## Évaluation de la rupture de pente et du comportement technique des dépôts marins surélevés de la rivière Clyde, en Écosse

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**ABSTRACT:** The raised marine deposits (RMD) of the River Clyde, Scotland comprise mostly laminated silts and clays that were originally deposited below water in marine/estuary conditions at the end of the last glacial period. Following melting of the ice sheets the deposits were elevated above current sea level. Recent highway widening works, which comprised soil nails within slopes of RMD, resulted in slope instability. An investigation into the instability has shown that the deposit is likely sensitive, with the slope having a prior history of instability. The slope has likely been subject to creep movement over time causing localised strain softening, resulting in progressive slope failure. The recent highway widening works likely triggered the failure of an already weakened slope. A monitored piled wall was used as remedial works to stabilise the failed slope.

**RÉSUMÉ:** Les dépôts marins surélevés (RMD) de la rivière Clyde, en Écosse, se composent principalement de limons et d'argiles stratifiés qui ont été initialement déposés sous l'eau dans des conditions marines/estuariennes à la fin de la dernière période glaciaire. Après la fonte des calottes glaciaires, les dépôts se sont élevés au-dessus du niveau actuel de la mer. Les récents travaux d'élargissement de la route, qui comprenaient des clous de sol à l'intérieur des pentes de la RMD, ont entraîné une instabilité des pentes. Une enquête sur l'instabilité a montré que le gisement est probablement sensible, la pente ayant des antécédents d'instabilité. La pente a probablement été soumise à des mouvements de fluage au fil du temps, ce qui a entraîné un ramollissement localisé des déformations, ce qui a entraîné une rupture progressive de la pente. Les récents travaux d'élargissement de la route ont probablement déclenché la rupture d'une pente déjà affaiblie. Un mur en pieux surveillé a été utilisé comme ouvrage de réparation pour stabiliser la pente en rupture.

**Keywords:** slope failure, raised marine deposits

## 1 INTRODUCTION

On 20 November 2015 during widening works along a section of the M74 motorway to the east of Glasgow, Scotland, a tension crack was identified along an existing highway slope above a recently widened section of motorway (Figure 1). The tension crack represented the onset of slope failure and comprised outward movement of the slope above a live section of the M74 motorway. To prevent further slope movement of the failed slope a temporary berm of granular material was placed at the toe to act as support. The berm was located within the widened zone adjacent to the M74 running lanes and allowed the motorway to remain open.

Following the discovery of the tension crack the slope was continually monitored for signs of

movement. Slope movement was essentially arrested following the placement of the temporary berm. Subsequently, the temporary berm was removed, and the failed section of slope remediated with a piled wall.

This paper provides an assessment of the cause of the failure, which occurred within slopes formed in raised marine deposits.


*Figure 1. Location plan of failure site (Bing, 2018)*

## 2 TOPOGRAPHY & SITE HISTORY

The failure site was located on the existing highway slope above the M74 fronting the west-bound lane approaching Glasgow.

The slope was heavily vegetated with a mix of mature/semi-mature trees and scrub. The existing slope inclination varied from 22 to 26° with a height of about 13m (crest at 40m OD). The lower part of the slope was supported by a gabion basket wall with a retained vertical height of about 2m.

As part of the recent widening works the gabion basket wall was removed and the toe of the existing slope cut to form a 60° slope about 4m high supported by soil nails.

A review of historical maps and aerial photography showed that the failed slope was originally an incised natural valley slope that was subsequently modified over time. Historical maps suggest that the failed slope may have been susceptible to instability prior to any development in the area, due to the presence of under-cutting at the toe by the Bow Burn stream (Table 1). More recent aerial photographs showed the failed slope was supported by gabion baskets at the toe, and investigations on the slope following the failure exposed granular drainage blankets and extensive surface drainage.

*Table 1. Historical development*

| Year | Description |
|---|---|
| 1859 | The failure site comprised a north-facing wooded natural valley slope of the Pow Burn. Burn had under-cut and eroded the slope, indicating possible slope failure. |
| 1898 | Railway line constructed within the Pow Burn valley. Railway line cutting in the north facing valley slope at the location of the failure. |
| 1936 | Residential development at the crest of the slope. Railway line in operation. |
| 1968 | M74 motorway built along Pow Burn valley. The railway line abandoned and cut slope subsumed into the highway slope. |
| 2015 | Failure site is a wooded highway slope beside the M74. Gabion basket wall present at the toe of the slope. |

## 3 GROUND CONDITIONS

### *3.1* General

The superficial ground conditions in the area comprise a complex stratigraphy of glacial, lacustrine and marine soils deposited during essentially the last Ice Age, see Figure 2.

Glacial activity within the Clyde Valley during the last Ice Age involved several stages, each of which was accompanied by deposition (Finlayson, 2012). It is considered that an initial ice-sheet advanced into the Clyde Valley from the northwest which coalesced with ice from the south. During deglaciation, the ice sheets retreated and separated, which resulted in

deposition of lacustrine and deltaic sediment into a glacial lake (Clough et al., 1911). Retreat of the northwest ice sheet was accompanied by advance of the contemporary sea, in which marine sediments were deposited (Browne and M

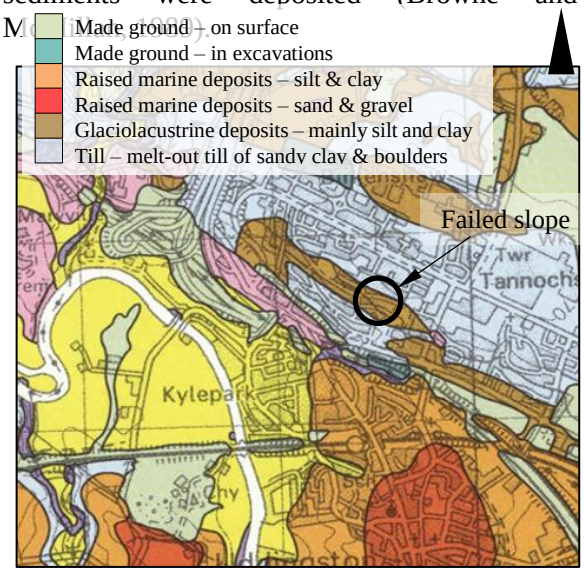


*Figure 2. Plan showing soil deposits (BGS, 1992)*

The raised marine deposits (Clyde Beds) were originally deposited below sea level but are now up to 40m above current sea level (OD) as a result of rise in global seal level following melting of ice sheets and isostatic uplift following the last Ice Age (BGS, 1992).

The slope failure lies on the northern margin of the raised marine deposits, which coincided with the incised valley of the Pow Burn (Figure 2). The raised marine deposits overly more competent over-consolidated lodgement till or bedrock.

Raised marine deposits originally deposited below marine water can be prone to leaching of salt causing quick conditions, that is excessive loss of strength on remoulding, though there is only limited evidence of this in the Clyde Valley area, see for example Clark et al (1979).

## *3.2* Raised Marine Deposits (RMD)

Site investigation was carried out on the failed slope, comprising boreholes, trial pits with the installation of piezometers and inclinometers.

The failed slope comprised RMD overlying bedrock at depth. The RMD comprised a visually monotonous deposit of soft becoming firm below about 7m depth greyish brown laminated to very thinly bedded, locally slightly sandy CLAY/SILT of intermediate to high plasticity.

Figure 3 shows index test results (natural moisture content (NMC), liquid limit (LL), plasticity index (PI) and liquidity index (LI)) with depth. The results indicate that above 12m depth the RMD comprises a more recent slightly over-consolidated deposit with plasticity of range from generally intermediate to high. Below 12m depth the RMD appears to be an older over-consolidated deposit with low plasticity.

The liquidity index (LI), which is a measure of relative moisture content of the soil, provides an indication of soil strength and sensitivity. Higher LI would indicate greater sensitivity and lower strength. LI is notably high between depths from about 5 to 6m, and locally at 8.5m depth.

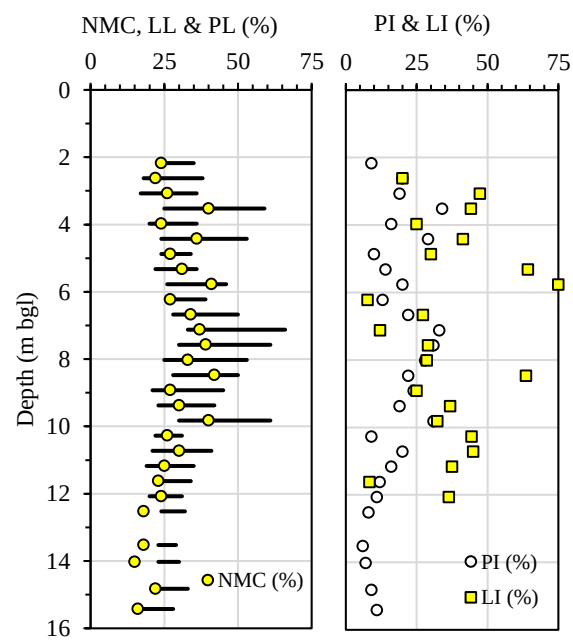

*Figure 3. RMD index results with depth*

Strength was determined both by insitu and laboratory testing, with the latter providing notably low values with no discernible trend with depth which is considered to be related to samples failing along laminations. Figure 4 shows the insitu strength with depth.

Hand vane results were also used to determine the sensitivity ($S_t$) of the RMD (ratio of peak to remoulded undrained shear strength). The average sensitivity was 4.3 (range 2.4 to 7.3). A sensitive soil would have a sensitivity of 4 to 8 (Skempton and Northey, 1952). Results show (Figure 4) a higher sensitivity from 6 to 8m depth and at greater depth below the toe of the slope.

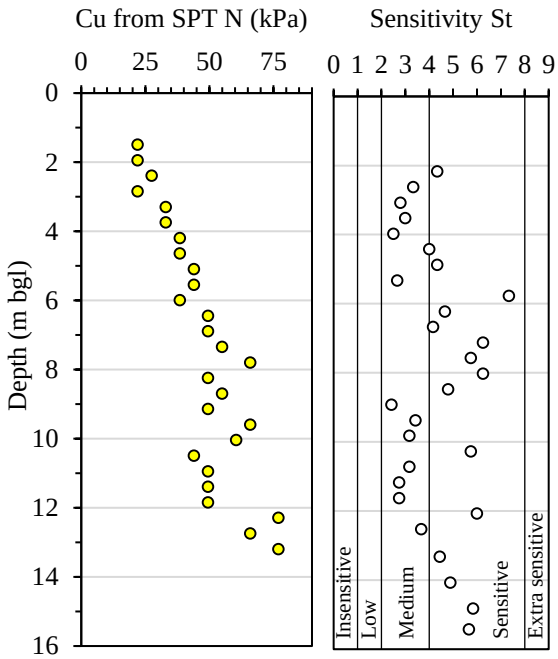


*Figure 4. RMD strength (SPT N in $c_u$) and sensitivity with depth*

The drained strength (internal angle of friction, φ') for peak and residual conditions was derived from shear box testing. The results showed a peak strength of φ' = 30° (based on best fit line). The residual strength was φ' = 14.5° degrees (also based on best fit line). The change from peak to residual value indicated a notable reduction (brittleness) in strength should the slope be subjected to strain as a result of say long-term slope movement.

## 4 RAINFALL

A review of rainfall data prior to identification of tension cracks on 20 November 2015 was carried out using data from the nearby Glasgow Bishopton rain station (Met Office, 2018).

Preceding the failure the rainfall amount in November was significantly high, with elevated rainfall amounts occurring from 9 to 18 November (Figure 5). From the beginning of the month up to 20 November about 240mm of rainfall had occurred which was about 180% of the long-term average monthly total rainfall of 131mm for November.

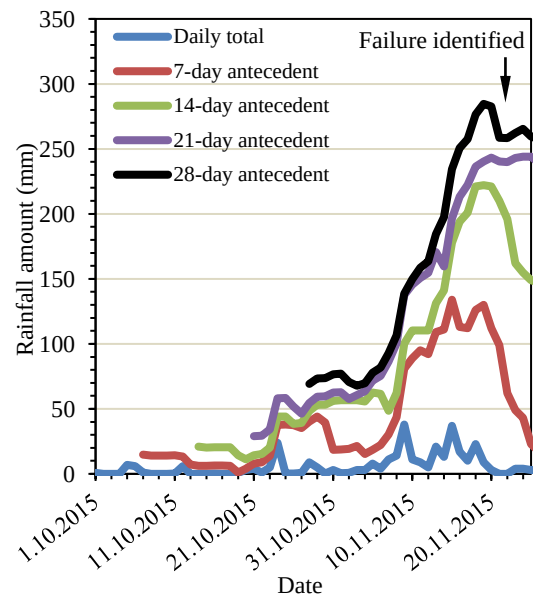


*Figure 5. Antecdent rainfall prior to failure*

The monthly total for November 2015 was 304mm, which exceeded the previous monthly record total of 299mm recorded in December 2006 (historical rainfall data from 1959 to 2013 for the Glasgow Bishopton weather station).

The antecedent rainfall amounts (for 7, 14, 21 and 28-day periods) were also examined, see Figure 5. This showed a significant rise in antecedent rainfall amounts from about 9 November onwards with a peak on 19 November 2015, a day before the failure was identified.

The antecedent rainfall amount in particular is considered significant as there would be a lag between rainfall and an associated response and elevation in water pressures in the slope. High antecedent rainfall is typically associated with deeper landslides (for example in Scotland see Postance et al, 2018).

## 5 SLOPE FAILURE

### *5.1* General

The slope failure occurred a short time after the installation of soil nails in the lower slope, which was carried out in October and November 2015. The failure was initially identified on 20 November.

The ground conditions in the failed slope comprised cohesive RMD with evidence previously installed shallow drains and granular drainage blanket, indicating likely earlier slope repair.

The initial indications of slope failure were bulging around a number of the lower rows of soil nail heads and particularly bulging and heave at the toe of the nailed slope (Figure 6).

*Figure 6. Bulging at slope toe (inide dashed line) indicating onset of slope failure*

A back-scarp/tension crack was subsequently identified and comprised an arcuate crack about 60m in length running close to the crest of the slope at the highest point of the existing slope. The back-scarp had a vertical displacement of up to 1.5m with horizontal displacement of less than 0.9m. Beyond the crack no further distress was identified.

The geometry of the back-scarp suggested a predominantly deep rotational failure. The failure surface appeared to pass below the soil nail block and daylight at the toe of the nailed slope. It was likely the failure surface essentially passed around the soil block and daylighted below the lower row of nails. This coincided with the observation of bulging at the toe of the nailed slope (Figure 6).

Though there had been significant rainfall preceding the failure, there was no apparent evidence of spring lines or water seepage from the slope before, during or following the failure.

The existing slope above the soil nails was well vegetated comprising low vegetation and trees; the majority of trees showed no indication of slope creep.

Ground displacement profiles with depth from inclinometers installed shortly after the failure showed there was a reasonably consistent ground displacement above about 6m depth. From below about 6m bgl the ground showed little displacement. Based on these readings, the basal shear surface was assumed at 6m depth at the inclinometer location, and confirmed the deep-seated nature of the failure (Figure 7).

Ground investigation boreholes following the failure encountered either no groundwater or slight seepages at 2 to 5m depth. Installed standpipe piezometers recorded groundwater at 9.7 to 9.9m depth, which is towards the base of the slope. Whilst evidence of elevated groundwater was not recorded in the failed slope the presence of localised high pore water

pressures within particularly localised sand layers within the slope cannot be discounted.

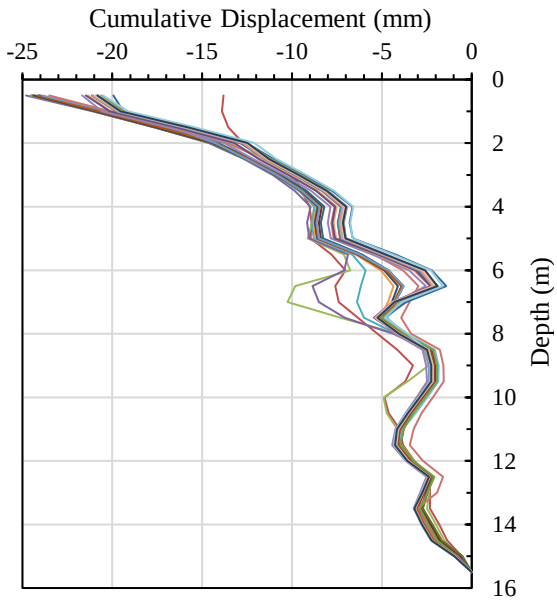


*Figure 7. Displacement with depth from inclinometer readings (Dec 2015 to Mar 2016)*

## *5.2* Stability Analysis & Remedial Works

To determine the likely operational shear strength and groundwater conditions within the slope at failure a stability analysis was carried out with the factor of safety (FoS) for the slope assumed to be 1.0, with all partial factors set to 1 (BSI, 2010).

A drained (long-term) analyses and an undrained (short-term) analysis was undertaken both of which modelled the pre-failure slope including soil nails using Talren software.

The drained analysis was carried out for a typical range of soil friction values (φ') and a range of $r_u$ (Figures 8 and 9). The drained analysis results showed that for failure to occur at peak strength (φ' = 30°) then a notable groundwater level ($r_u$ = 0.4) was required in the slope. The actual recorded groundwater level in the slope was relatively low. As such, for failure to occur the operating soil strength in the slope would have had to be notably lower than peak strength. The analysis suggests the presence of a weaker soil in the slope was required to cause the failure.

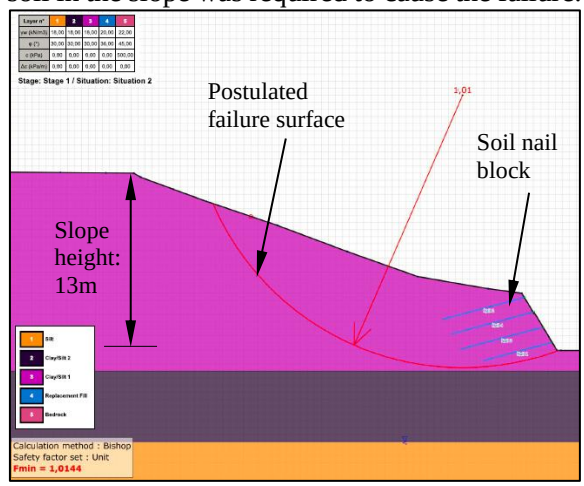


Figure 8. Slope geometry and stability analysis (peak φ' = 30° and $r_u$ 0.4)

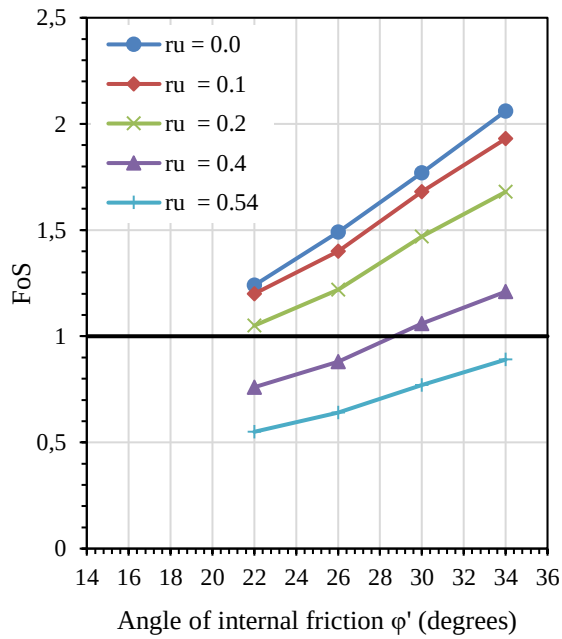


*Figure 9. Slope FoS versus internal friction angle (φ') for a range of $r_u$*

As the slope-forming material was described as mostly soft, an undrained analysis was also carried out. For failure to occur along the actual failure surface the operational undrained strength at failure would need to be about 20kPa.

Based on the results of the stability analysis, for failure to occur the operational soil strength in the slope was notably less than that determined from ground investigation. The weaker soil in the slope was likely due to the presence of a previous part failure surface in the slope, or possibly long-term slope movement resulting in localised strain softening (reduction in strength) over time, leading to progressive failure.

Using the back-calculated operational strength from the above analysis a remedial design was carried out. The failed slope was remediated using a cantilevered contiguous piled wall prior to the removal of the temporary slope support.

The contiguous piled wall was monitored using Shape Accel Arrays (SAAs) to confirm the cessation of ground movement.

## 6 CONCLUSIONS

The main conclusions are as follows:

(1) Within the Clyde Valley there is significant local variation in soil deposits as a result of a complex glacial depositional history.

(2) The Clyde Beds are cohesive RMD, which were originally deposited within the Clyde Valley below sea level and due to isostatic uplift are now found at elevations up to 40m OD. The RMD are slightly over-consolidated generally soft to firm laminated SILT/CLAY, and are locally variable.

(3) A failure occurred in a highway slope formed in RMD following widening and installation of soil nails at the toe. A review of the slope history showed that the slope was originally an incised natural valley slope of the Pow Burn that had a history of instability (Table 1). The slope was subsequently modified and incorporated into a railway and then a highway slope.

(4) Investigation of the RMD in the failed slope showed that the soil had a low undrained strength with a variation in index properties and sensitivity with depth (Figure 4).

(5) Stability analysis was carried out to determine the likely operational strength on the actual shear surface at failure.

(6) For failure to occur, the operational soil strength in the slope would have had to be notably lower than peak strength (Figures 8 and 9).

(7) The analysis suggests the presence of a weaker soil in the slope. The weaker soil in the slope was likely due to the presence of a part pre-existing failure surface, or possibly long-term slope movement resulting in reduction in strength over time, leading to progressive slope failure.

(8) Using the operational strength, the stability of the original slope prior to widening was marginal. The excavation due to the widening for installation of the soil nails likely triggered the failure of an already weakened slope.

(9) The installed soil nail block provided support locally to the steepened soil nailed slope but overall the slope failed as a result of global failure around the soil nail block.

(10) Prior to the failure, the rainfall was significant, with antecedent rainfall peaking the day before the failure was identified (Figure 5). The coincidence of a significant period of rainfall, approaching record levels, and the occurrence of the slope failure are considered to be related.

## 7 ACKNOWLEDGEMENTS

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