

AN EQUATORIAL QUICK CLAY LANDSLIDE, SUMATRA, INDONESIA

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Summary A 1993 failure of a length of bank of the Siak River in Sumatra is interpreted as representing shearing of a highly sensitive marine clay. The characteristics of the slide and the properties of the clay are in the category of the 'quick' clay slides of Scandinavia and Canada. This appears to be the first record of a quick clay slip in an Equatorial area.

1 INTRODUCTION

On 1 July 1993 a landslip occurred in the bank of the Siak River in the town of Pekanbaru, Sumatra. The interesting facet of this failure is that it involved undrained shearing of a very sensitive marine clay which exhibited 'quick clay' properties similar to those previously described in Scandinavia and Canada and also reported in Iceland Alaska, Japan and New Zealand (Torrance, 1996). As far as the author is aware this is the first description of quick clay in an Equatorial area.

A very good summary of the properties of quick clays is given by Lefebvre (1996) and there is no need to repeat the information here.

Unfortunately, due to the remote location of the site it was not possible to obtain detailed engineering properties on the materials. However, it is hoped that the limited information given here will provide some guidance for future workers in the area.

2 THE FAILURE

The Siak River is an important artery of central Sumatra and flows through the regional capital of Pekanbaru (see Figure 1). The section of river bank where the slip occurred was a hardstand area with a simple timber retaining structure forming a wharf. The slip happened at about 7pm during unloading of 12m long precast piles from a large sea-going barge, the Java Walrus. All personnel were at dinner at the time and there were no eye witnesses. When they returned to work after their 30 minute dinner break the Java Walrus had been shoved 30m sideways out into the Siak, and 2400 square metres of hardstand and wharf had disappeared.



Figure 1 Site location plan

A sketch plan and interpreted cross-section are given in Figures 2 and 3 . Photograph 1 shows a view of the failure. The plan is based on a tape survey and the section on depth measurements made by the author from a small boat and from ground level measurements made relative to high tide. Subsurface interpretation is based on:

- exposures of the upper 2.2m of the profile in the sides of the slip, and
- extrapolation of borehole and Dutch Cone data from a site investigation conducted in 1987 approximately 300m downstream of the slip.



Photograph 1 View of part of slip area - river is to left of photo.

Key features of the slip are listed below.

- 1 The slip occurred over a 75m length of the bank and extended some 50m back from the timber retaining structure.
- 2 The slip was on the outside of a gentle bend in the river where the natural below-water bank is quite steep, averaging about 25° to 40° between about 3m and 12m below water level.
- 3 Upstream and downstream of the slip area the water depth was about 11m to 13m at high tide, in front of the slip the water depth was between 5m and 9m (average 7.5m) over the full 100m width of the river.
- 4 A large dead tree, which prior to the slip was on the edge of the bank at the upstream edge of the slip, had been displaced some 30m into the river, but remained vertical. Immediately upstream of the tree the water depth was 11.5m, while a short distance downstream of the tree it was 8m.
- 5 A large crane which was being used for off loading precast piles from the Java Walrus, together with the truck it was loading, dropped vertical several metres and was moved sideways about 25m into the river. The crane remained vertical.

The failure would be classified as a non-retrogressive landslide according to the categorisation used in Canada (Lefebvre, 1996) or as a flake-type according to the Norwegian system (Gregersen, 1981).

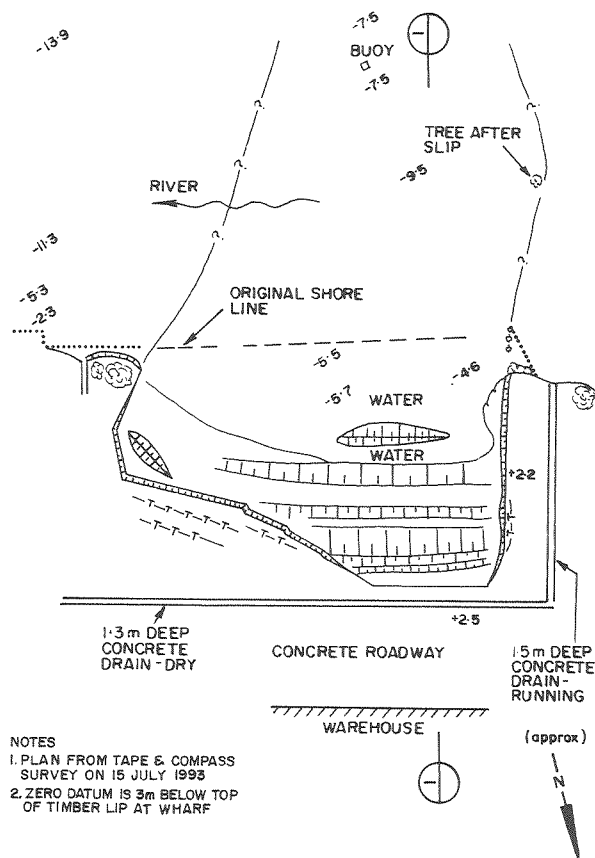


Figure 2 Plan of slip area.

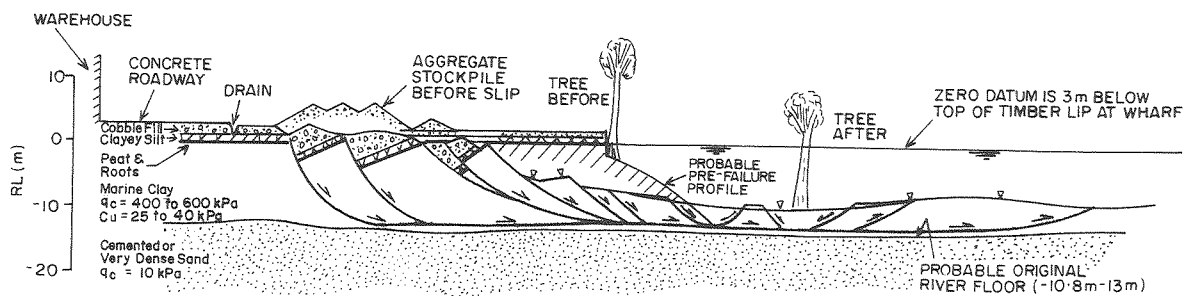


Figure 3. Interpreted cross-section through slip

3 GEOTECHNICAL PARAMETERS

Exposures in the scarp faces around the slip indicated that the upper 2.2m of the profile within the hardstand area comprised the following :

- **Fill**, 0.9m to 1.2m thick, comprising cobbles and river gravel in a sandy clay matrix, overlying
- **Clayey Silt**, about 1m thick, grey, firm and overlying
- **Peat and root mat**, at least 200mm thick (the profile below this was under water)

Based on topographic and geomorphological similarity, borehole and Dutch Cone data from 300m downstream could be used to assess the deeper profile. This was interpreted as:

- **Marine silty clay**, soft to firm, brownish grey, extending to between 13m and 14m below hardstand level, overlying
- **Cemented sand** or very dense clayey sand, having cone tip resistance values above 10MPa.

The key unit is the marine clay. The Dutch Cone data indicate the marine clay to have undrained strengths in the range 25kPa to 40kPa. As summarised in Table 1, two sets of laboratory tests gave natural moisture contents of 64.6% and 40.3%, with corresponding Liquid Limits of 66% and 52%. Plasticity Indices were 27% and 22%.

Property	Laboratory Result	
	Borehole 1 2.0 - 2.4m	Borehole 2 2.6 - 3.0m
% minus 75 micron	89	87
Moisture Content-(%)	65	40
Void Ratio	1.9	1.1
Liquid Limit - (%)	66	52
Plasticity Index -(%)	27	22
Liquidity Index	0.97	0.45
c' - Direct Shear-(kPa)	2	2
ϕ' - Direct Shear	27°	29°

Table 1 Test data from 1987 investigation.

Apart from the high liquidity index there was nothing in these results to suggest to the writer that there was anything unusual about the clay. However, facets of the slide were a puzzle. Why had it extended so far into the river and so far back behind the bank? Why did it exhibit a multiple 'graben' structure with near horizontal translation and no tilting of the old tree and the crane? Why had it happened so rapidly?

The answer to these questions came almost by chance when I visited another site further upstream where a new wharf was under construction. Excavations for dead man anchors had been made into the upper part of the Marine silty clay unit. As is the behaviour pattern of most geotechnical engineers I picked up a lump of this nice plastic clay and walked along absent mindedly kneading it in my hand. I gradually became aware that the clay was flowing from my fingers like a thick cement grout. It had changed from plastic to fluid. The light came on. Being a southern hemisphere 'dry country' geotechnical engineer I had never come across quick clay, but I clearly remembered a picture from my Imperial College days of Bjerrum pouring previously plastic clay from a cylinder. With this understanding the behaviour of the landslip became explicable. It had some of the characteristics of the famous Rissa Landslide (Gregersen, 1981), of which I had seen the amazing video.

Unfortunately there was no way of obtaining undisturbed samples of the Marine unit, let alone getting them back to Australia for testing. The only test data is that summarised above from the 1987 site investigation some 300m away from the slip. These data, summarised in Table 1, do not include peak and remoulded undrained strengths. However, by comparison with the typical Norwegian and Canadian quick clay parameters given in Table 2 it is noted that:

- Liquid Limits at Pekanbaru are higher than for Scandinavian and Canadian quick clays.
- The Scandinavian and Canadian clays typically show Liquidity Indices of greater than 1.2 whereas the Pekanbaru clays have LI values just below 1.0.
- The undrained shear strengths of the Pekanbaru clays, as interpreted from mechanical Dutch cone data are similar to quoted values from Scandinavia and Canada based on vane shear testing.

4 ANALYSIS OF THE SLIDE

Immediately prior to the slide, loading on the hardstand area comprised the following:

- Approximately 3500m³ of crushed aggregate over an area of about 30m by 30m and at least 20m from the edge of the bank,
- Approximately 460 tonne of steel plate and 180 tonne of 12m long rolled section, probably all at least 10m from the edge,
- An 80 tonne crane and a large truck, within 10m of the edge, and
- Approximately 780 tonne of 12m by 400mm square precast piles stacked 15m to 30m from the edge.

The aggregate and steel plate had been placed more than a few days before the slip but the precast piles were being unloaded from the Java Walrus barge at the time of failure. The items listed above typically equate to surcharge pressures of between 40kPa and 70kPa applied in patches ranging from about 20 to 800 square metres.

Property	Canadian and Scandinavian Quick Clays (Lefebvre, 1996, Trak & Lacasse, 1996, Gregersen, 1981)
% Silt & Clay* (%)	> 70
Liquid Limit (%)	20 to 35
Liquidity Index	0.8 to 2.5
Plasticity Index (%)	5 to 40
Undrained Shear Strength (kPa)	10 to 30
Sensitivity	> 30
Effective Shear Strength**	
C_m (kPa)	≈ 7
ϕ_m	28° to 35°
* Note that in many cases a significant portion of the minus 2 micron material is not clay mineral. ** For preconsolidation pressures of 100 to 200 kPa	

Table 2 Typical parameters for quick clays.

There is no doubt that failure was caused by the surcharge loading. Analyses were performed using 2D slip circles and a 3D limit equilibrium method (the program INSLOPE developed by the US Bureau Mines) for different surcharge loadings and clay shear strengths. The results are summarised in Figure 4. This figure shows that a Factor of Safety of less than 1.0 would be expected given the peak undrained shear strengths interpreted from the Dutch Cone data and the actual surcharge pressures. However, the point of this note is not to record such a simple conclusion but to point out that normal stability analyses would not have given any indication of the extent of the slide. In fact the analyses would have suggested that failure should only have affected the immediate river edge.

5 CONCLUSIONS

Post slide initiation behaviour and the mobility of landslides is now receiving increasing attention in Australia and is of vital importance in assessing landslide consequence. For the author the lesson from this experience was that standard

site investigation data do not provide clear cut fingerprinting of post-landslip behaviour. However, be wary of deposits with high Liquidity Index coupled with low Plasticity Index Values.

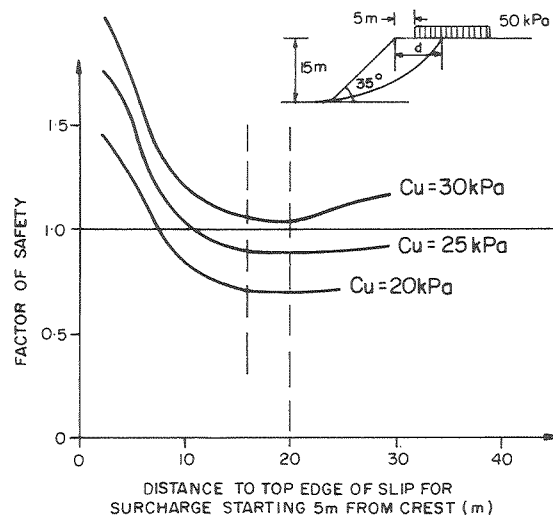


Figure 4 Analysis of the slip

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