The Santana Port accident: Could it be a sensitive clay flowslide under the Equator?

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ABSTRACT
In March, 28, 2013, an abrupt slope movement involving about half a million cubic meters of soil took place at a Port situated in the North margin of the mouth of the Amazon river. Most of the soft clayey silt flowed under water for a distance of about 350 meters. After the accident a site and laboratory geotechnical testing program has been carried out. This paper is the first publication after the accident and intends to present:

- Basic sequence and mechanism of failure;
- Typical geotechnical characteristics of the soft soil
- Preliminary analyses of the movement
- Speculations on the cause of failure and on the factors that may have contributed to the sensitivity soil.

The flow that followed failure is similar to those that occur in Canadian and Scandinavian highly sensitive clays. This kind of behavior in clayey soils does not seem to have precedents on the Brazilian coast or in any other tropical areas.

RÉSUMÉ
Le 28 mars 2013, s'est soudainement produit un glissement de terrain impliquant un demi-million de mètres cubes de sol dans un port situé sur la rive nord de l'embouchure du fleuve Amazone. Le gros du silt argileux mou, s'écoula sous l'eau sur une distance d'environ 350 mètres. Après l'accident, un programme d'essais de terrain et de laboratoire a été réalisé.
Cet article est la première publication après l'accident et a l'intention de montrer:

- Les principales séquences et le mécanisme de rupture.
- Les caractéristiques géotechniques typiques du sol mou.
- Des analyses préliminaires du mouvement.
- Des spéculations sur la cause de la rupture et sur les facteurs qui ont contribué à la sensibilité du sol de Santana.

La coulée qui a suivi la rupture est semblable à celles qui se produisent dans les argiles très sensibles canadiennes ou scandinaves. Ce type de comportement ne semble pas avoir de précédent sur les côtes du Brésil ou de n'importe quelle autre région tropicale.

1 INTRODUCTION

An iron ore terminal Port situated in the city of Santana, State of Amapá, in the North margin of the mouth of the Amazon river, some 170 Km from the Atlantic Ocean, as shown in Figure 1, suffered a mass movement involving about half a million cubic meters of soil, in March, 28, 2013. The failure caused six casualties and the complete interruption of the operation of the Port.

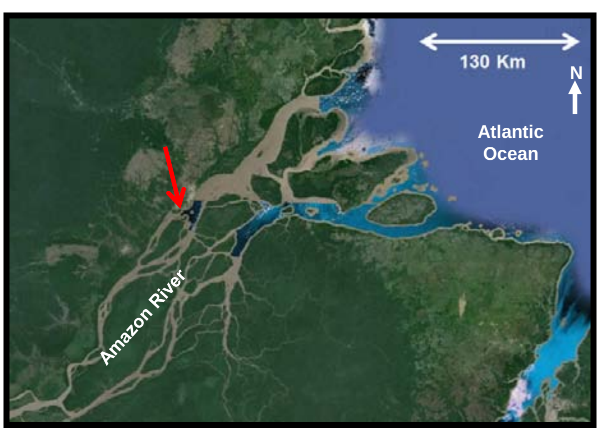

Figure 1. Location.

The Port was built in 1955-1956 and exported manganese from 1957 to around 1997. From 2007 to the present day the Port has exported iron ore. An inclined aerial photo taken after failure with the failed area enhanced is shown in Figure 2.

The Port had from the beginning of operation up to the moment of failure, a lower area on soft soil in which only operational loads (such as vehicles and small piles) were applied and an upper area on hard soil in which the heavy ore piles were stockpiled. These areas are indicated in Figure 3.

This paper is the first publication since failure and intends to present:

- Basic sequence and mechanism of failure.
- Typical geotechnical characteristics of the soft soil.
- Preliminary analyses of the movement.
- Speculations on the cause of failure and on the factors that may have contributed to the sensitivity of the Santana soil.

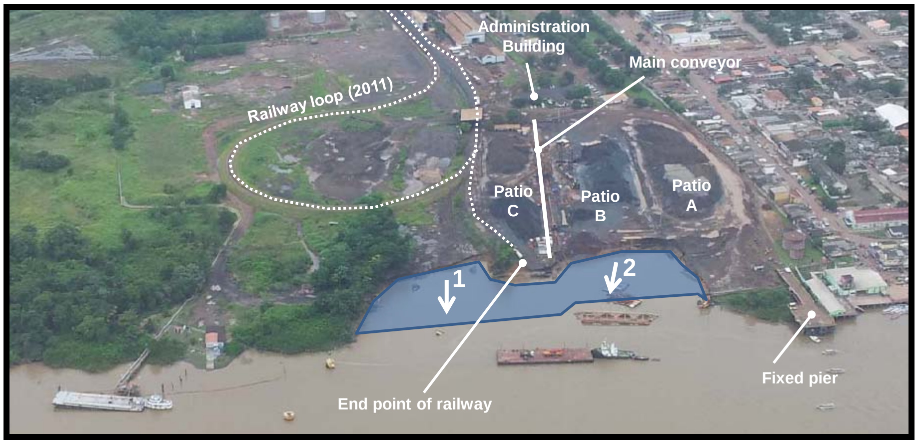

Figure 2. Aerial inclined photo taken after failure. Shading indicates failure area.

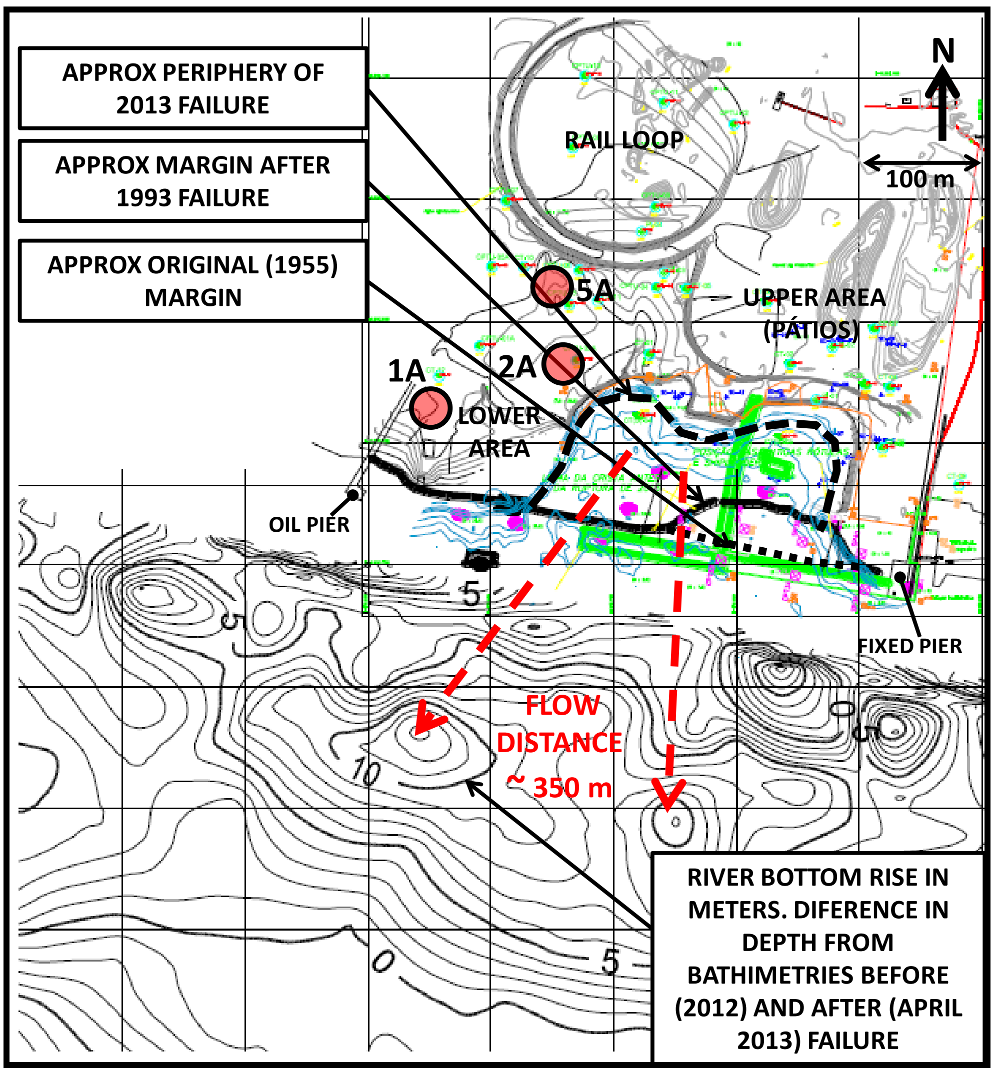

Figure 3. Plan of failure area comparing situations before and after.

2 SEQUENCE OF EVENTS

The ground in the lower area is composed by soft soils down to depths exceeding 40 meters, deposited in the last 10000 years. The subsoil in the upper area is composed by hard soils of tertiary age. The lower area was originally inundated by tides. In 1956 these lowlands have been heightened to elevations around +2.20 m with a fill with average thickness of 2.50 meters.

The Port operated without geotechnical problems from 1957 to 10 October 1993 when a slide involving an area of about 2,000 m^2 and a volume around 30,000 m^3 happened in the East side of the lower area, as schematically indicated in Figure 3. From the limited information available about this failure, one learns that no load had been applied on the failed area nor in its vicinities, that no previous signs of impending failure had been observed and that failure was sudden, caused damage to the ship loading system (but, fortunately, no casualties) and generated strong waves.

In 2007 the ore handling system of the Port has been adapted to augment efficiency of ship loading. The velocity of the main conveyor system was increased. No additional load has been applied to the ground in the lower part. A railway loop built in 2011 was not affected by the 2013 movement (see Figure 2).

A large failure happened abruptly at 00h:10m:28s of 28 March 2013 as registered by a security camera which was obliterated a few seconds after failure started. In the moment of failure the river level was low. Some 30 min before it had reached its spring tide minimum, around elevation -1.70 m. At the moment of failure the river level was around -1.60 m. Weather had been rainy for a month or so with intensity inside the normal range for the period. A typical general section before and after the failure, can be seen in Figure 4.

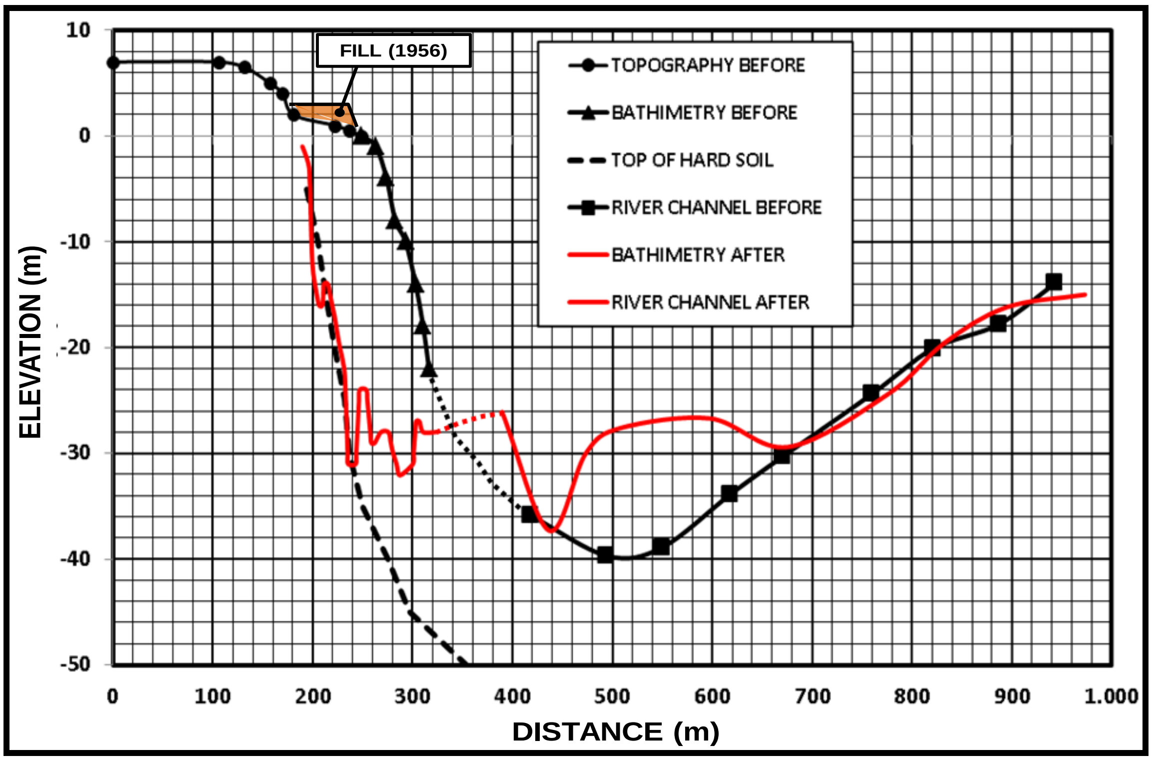

Figure 4. Typical general section.

The comparison of bathymetries before and after failure indicates that a considerable part of the failed material flowed under water to the river channel, for an average distance of about 350 meters, causing the river bottom to rise up to about 13 meters, as shown in Figures 3 and 4.

Based on the images of a security camera positioned at the fixed pier (see Figure 3) and on description given by eye witnesses it has been reasonably established that the mass movement happened in two stages, as indicated by the arrows in Figure 2. The first stage was in the West side and involved some 350,000 m^3. The second stage involved some 150,000 m^3, happened about 2 to 4 minutes later and was most probably triggered by foot removal induced by the first stage. The images from the security camera also indicate that no signs of movement could be noticed until 00h:10m:28s of March, 28, 2013. About five seconds later complete failure had happened. Slight movements of the floating components of the ship loader registered by the security camera some 4 seconds before failure might be interpreted as indicating that a precursory relatively small slide occurred.

Overall the landslide affected an area of 20,000 m^2, had a width of 220 m, regressed by 50 to 100 m and involved an average thickness of 30 m of sediments (see Figures 4 and 5).

3 GEOTECHNICAL CHARACTERISTICS

3.1 Geotechnical investigations

A typical geotechnical section before failure is shown in Figure 5. There are four main materials of interest from top to bottom: a clayey fill placed in 1955-1956, a clayey (probably desiccated) crust, a soft soil and a hard soil which underlies the soft soil in the failure area and reaches the surface in the upper (patios) area. The soft soil is a grey clayey silt with SPT values around zero. The hard underlying soil is mottled clay with SPT N values in excess of 20.

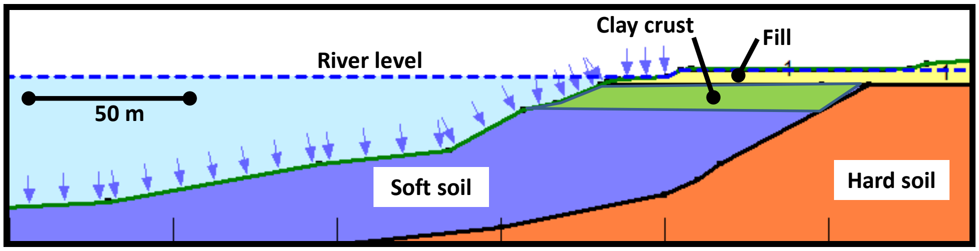

Figure 5. Typical section before failure.

Two main geotechnical investigation campaigns have been carried out: one before failure in 2007 and the other after failure, in 2013. The 2007 campaign comprised boreholes with SPT and piezocone vertical profiles with dissipation tests, some being in the failed area. After failure a large number of field and laboratory tests have been carried out, including: boreholes with SPT, thin tube stationary piston sampling (metal tubes with diameter 7.5 cm, length 60 cm and area ratio around 18%), piezocones (10 cm^2 cone driven with 2 cm/sec speed) with dissipation tests, dilatometer tests and vane tests (130 mm high, 65 mm diameter, rotation rate 6 degrees/minute) in the field. Samples were transported 6,000 km south by plane to laboratories for testing. Laboratory testing included characterization, falling cone, consolidation and triaxial tests. Results from three investigation clusters, points 1A, 2A and 5A, see location in Figure 3, adjacent to the failed area, will be considered. The 2013 and 2007 piezocone results, carried out by two different firms, are very similar which indicates that the area adjacent to failure is representative of the soil involved in the movement.

Figure 6 shows the water content, LL, PL, grain size and liquidity index values obtained in thin tube samples from point 1A. The plasticity index is around 25% (20% to 35%). The right hand diagram of Figure 6 shows pore water salt content and radiocarbon dating. Without any surprise the salt content is less than 1 g/l. The age of deposition varies between 8.5 kyears at depth of 20 m and 10 kyears at depth of 40 m.

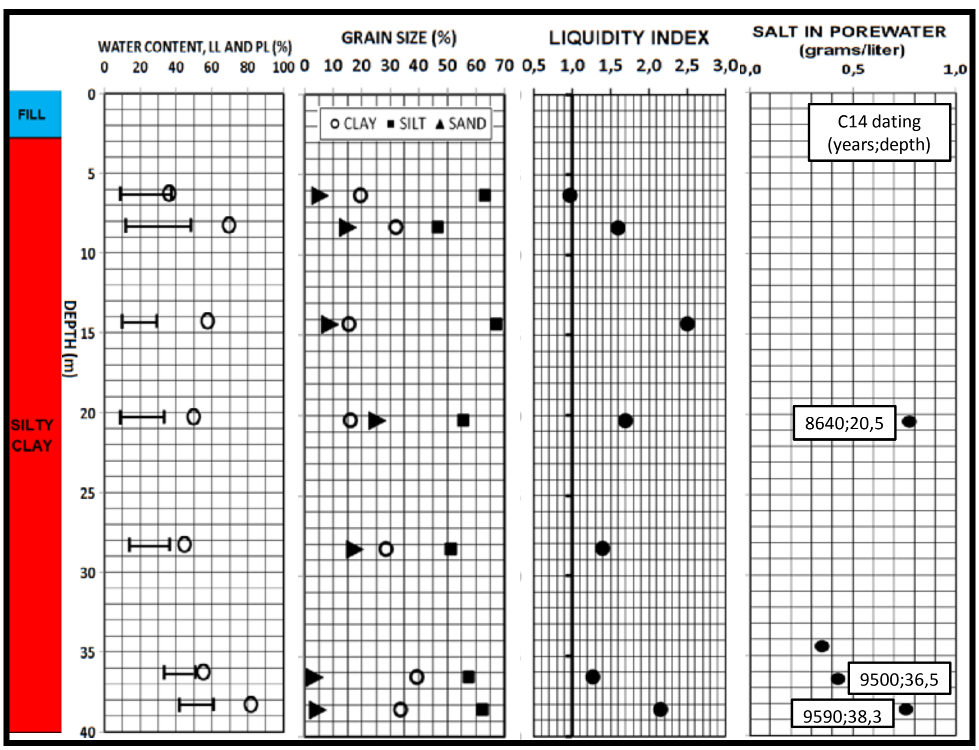

Figure 6. Characterization tests, pore water salt content and radiocarbon dating at point 1A.

Sensitivity values have been obtained with fall cone laboratory tests in a few samples. The highest sensitivity values obtained were 23.4 and 16.2 (Location 1A, depths of 36.4 m and 38.3 m, respectively). The corresponding fully remoulded strength obtained was 2.1 kPa and 1.7 kPa. More data on this subject is surely needed. IL values obtained in several points of the Santana soft soil is between 0.8 and 2.0 which, according to the empirical relation S_{ur} (kPa) = 1/(IL - 0.21)^2 forwarded by Leroueil et al (1983), would correspond to remoulded strength values between 0.3 kPa and 2.8 kPa, indicating a much higher sensitivity. A new sampling campaign using very thin wall tubes is under consideration and further sensitivity tests should be carried out.

Conventional incremental 24hr oedometer tests and constant rate of strain consolidation tests have been carried out. The quality of the samples, verified using the procedure proposed by Lunne et al (1997), has shown to be very poor, in spite of the care that has been taken to obtain, transport and extract them: boreholes have been filled with heavy liquid to compensate change in vertical pressure, the tubes have been inserted in the ground hydraulically, transportation in non-pressurized airplane and extraction of samples in the laboratory in accordance with Ladd & DeGroot (2003). The poor quality can be due either to the delicate structure of the soil or to the rather high area ratio (around 18%) of the sampling tubes that have been used. It is hoped that the results to be obtained in the thinner walled tubes to be collected in the future will bring better compressibility results.

CIU and CKoU triaxial tests (with pore pressure measurement) yielded the failure points (maximum deviatoric stress) plotted in $p' = (\sigma'_1+2\sigma'_3)/2$ x $q = (\sigma_1-\sigma_3)$ diagram in Figure 7. They indicate friction angle of 23° and effective cohesion of 16 kPa.

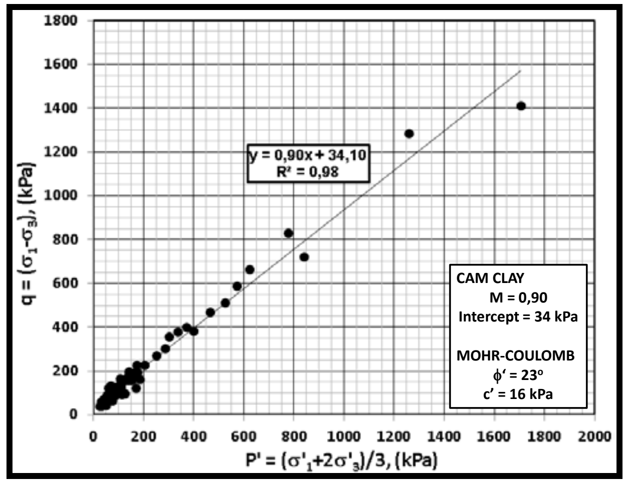

Figure 7. Effective stress failure envelope from CIU and CKoU triaxial tests. (Results of 39 CIU and CKoU tests in specimens carved from shelbies - Points 1A, 2A and 5A).

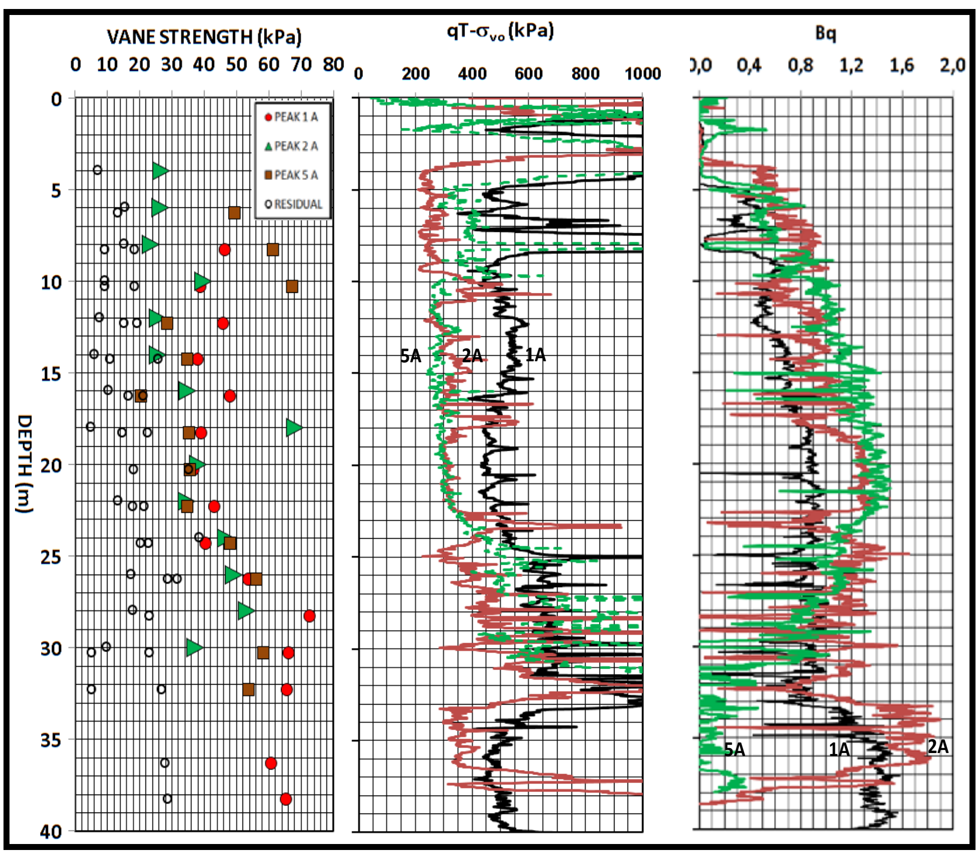

Figure 8. Field vane and piezocone tests results from points 1A, 2A and 5A. (Residual vane values obtained after ten rapid full turns).

Results from field vane and piezocone tests carried out at positions 1A, 2A and 5A are presented in Figure 8 and show a more or less constant value of $S_{u\ vane}$ and of (q_T-σ_{vo}) values for depths between 10 m and 20 m. Below 20 m these values present a linear increase with depth. B_q values for depths in excess of 10 m are frequently above 1.0 and reach 1.4 and 1.6 (which are unusually high values as will be discussed later). Values of the consolidation coefficient obtained from piezocone dissipation tests at depths between 5 m and 40 m, varied between 2 and 8 $x10^{-3}$ cm^2/s. N_{kt} values were found to be in the interval from 8 to 14, with a mean value around 12.

As for the OCR value of the Santana deposit a study has been carried out on the basis of the test results considered of good quality (vane, piezocone, dilatometer and the few laboratory consolidation tests with acceptable quality). The results indicate that the soil in the layers below depths around 20 m, is either normally consolidated or slightly over consolidated (OCR not exceeding 1.3 or 1.4). At depths smaller than about 20 m and up to 10 m the soil appears to be somewhat more over consolidated (OCR reaching values around 1.50). From depth 10 m upwards the soft soils seems to have OCR values above 2 (up to 4).

3.2 Particularities of the Santana soft soil

This item comments those geotechnical investigation results that seem to indicate that the Santana clay has characteristics which are not usually observed in Brazilian soft soils.

SPT N values are practically equal to zero and collide with peak vane strength values in the 35 kPa to 50 kPa or more range (Figure 8). The SPT values to be expected from these peak vane values would be, in any soft Brazilian coastal soils, in the range of 3 to 5. Similar relation applies to silty clays from several parts of the world as shown by Kulhawy and Mayne (1990). There are no shells or fibrous organic matter that might induce unusually high vane strengths and the N_{kt} values, as seen above, are normal (around 12).

B_q values well above unity reaching 1.6 or more have been regularly obtained (Figure 8). These are unusually high values in any soil and have never been systematically observed, to the Author's knowledge, in Brazilian clays. The usual Brazilian soft clays show B_q values up to around 1 and usually quite less (typically 0.6 to 0.8). It is noteworthy that B_q values in sensitive Eastern Canadian clays are generally between 0.6 and 0.9 (Demers & Leroueil, 2002) and, in sensitive Scandinavian clays are usually below or slightly above 1 (Karlsrud et al, 2005; Schnaid, 2009). The high B_q values obtained in the Santana soil cannot be credited to q_T being unusually low since, as mentioned above, N_{kt} values are in the normal range. It has to be noted that the high B_q values have been obtained in two different site investigation campaigns, 2007 and 2013, carried out with quality equipment by well trained personnel.

IL values quite above 1 have been determined in samples of the Santana soil from different depths. The usual Brazilian soft clays show IL values up to around 1 and usually less (0.8 is typical). High values of IL some exceeding 2, as shown in Figure 6, are unheard of in the Country as far as the Author's knowledge goes. Values of IL in excess of about 1.2 have been considered to characterize sensitive clays that can make a landslide becoming a flowslide (Lebuis et al., 1983).

Some of the sensitivity values obtained with fall-cone tests were high (in the 16 to 23 range) and the corresponding remolded strength were low (around 2 kPa) as mentioned before. Also, large liquidity indices indicate even lower remoulded strengths and higher sensitivities. On the other hand, sensitivities determined from the vane field tests where low, around 2 or 3, with a few exceptions. The residual vane strengths were around 6 kPa to 12 kPa as can be seen in Figure 8. It is the view of the Authors that vane tests tends to underestimate sensitivity since the strength obtained after repeated rotations of the vane should not be considered as representing the fully remoulded condition.

Zero N values, B_q values larger than 1.0, IL values larger than 1.0 and high sensitivity values point to a metastable soil with a predisposition to flowslide ("quick") behavior. In fact, a flowslide happened upon failure as confirmed by the evidence presented in Figures 3 and 4 and discussed later on. To the Authors' knowledge, this would be the first behavior of this kind ever reported on a soft coastal fine soil in the equatorial region.

4 ANALYSES

4.1 Loads in the failure area

The 2013 emerged failure area, some 20,000 m^2, is shown in Figures 2 and 3. The typical section is given in Figure 5. The upper hard soil area in which the heavy ore piles where stockpiled, was not involved in the movement, except for part of a pile affected by secondary retrogressive margin slides, whose remnants can be seen in Figure 2. Failure happened when the river level was low (around elevation -1.60 m previously indicated). Rain intensity was normal in the period preceding failure.

During the construction of the Port, in 1956-1957, the lower area has been filled with a 2.50 m-thick embankment (see Figures 4 and 5) which represents an added load of about 95,000 metric tons in the 20,000 m^2 area that came to fail in 2013.

The lower area, where the soft soil lies, received since the beginning of operation only moderate loads such as trucks and small ore piles which would not exceed some 50 to 60 metric tons. On the other hand heavy cyclic loadings did persistently intervene in the lower area, such as:

- The river level fluctuates twice a day from about 1.50 m above and 1.50 m below average level. Therefore a cyclic loading with a period around 12 hours and a magnitude of some 3 m of water was continuously acting upon the margin.
- Monitoring of water wells indicate that the water level in the superficial fill which covers the lower area rises about one meter during the rainy season. This seasonal change in water level propitiates a cyclic load which every year applies (from January to Mars) and removes (April to

December) a load of about 10,000 metric tons in the area that came to fail.

4.2 Limit equilibrium analyses

Limit equilibrium analyses in terms of effective and total stresses indicate that:

- Effective stress analyses. Seven VW piezometers have been installed at points 1A, 2A and 5A at several depths and consistently indicated that excess pore pressures in the soft soil are less than 5 to 10 kPa above or below hydrostatic pressure. Considering the peak effective stress parameters obtained from the triaxial tests ($c' = 16$ kPa and $\varphi' = 23°$, Figure 7), a safety factor of 1.98 is obtained, as shown in Figure 9a.
- Total stress analyses. Interpretation of field vane results at points 1A, 2A and 5A led to dividing the clay in six layers as shown in Figure 9b. Total stress ($\varphi=0$) analyses, which have no meaning in this case of natural slope failure, yield a safety factor of 1.79.

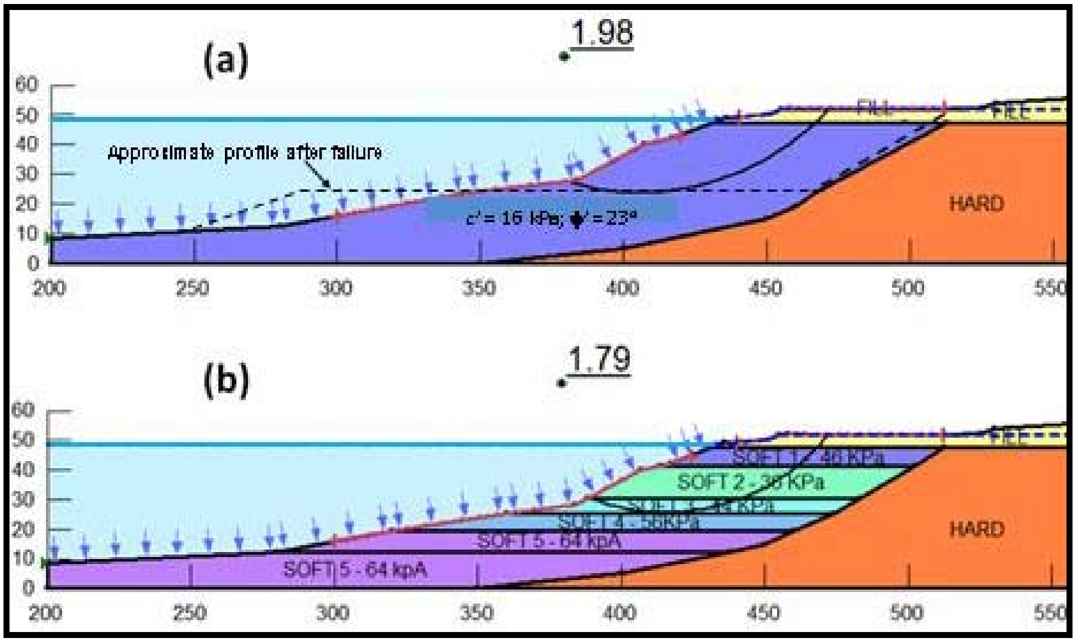

Figure 9. Limit equilibrium analyses with peak parameters: (a) Effective stresses; (b) $\phi = 0$.

Also to be noted is the fact that the critical failure surface obtained in the effective stress analyses shown in Figure 9a is not conditioned by the hard soil as it is known to have happened upon actual failure (see Figure 4 and after failure profile in Figure 9a).

The high values of safety factor indicate that failure is not adequately represented by limit equilibrium analyses with peak strength parameters. Failure seems to have mobilized average strength parameters substantially lower than the peak ones or to have induced high positive excess pore pressures. To get safety factors around unity it is necessary to reduce the effective cohesion to zero and to consider some loss in the effective strength angle. The results of such reduced strength analyses indicated that the failure would happen along a surface close to the face of the slope. Such a shallow failure surface would be compatible with the occurrence of a smaller triggering slide, which in turn could be associated with the movements registered by the security camera a few seconds before general failure. Such initial failure could also have initiated other failures that would have retrogressed to the final scar. Another possibility that could be consistent with the high calculated safety factor and also the fact that the failure crater is much larger than the volume indicated by the critical circle (Figure 9a) is the development of progressive failure and spread (Locat et al., 2011 and 2014).

4.3 Post-failure analyses

For post-failure analyses, the BING software (Imran et al., 2001) has been used. It considers the flowing material as a Bingham material characterized by a yield strength, τ_c and a viscosity, μ. The shear strength, τ_f is then:

$$\tau_f = \tau_c + \mu\gamma \qquad [1]$$

where γ is the strain rate.

As Locat & Demers (1988) observed that μ (Pa.s) is close to 0.001 τ_c (Pa), this was assumed for preliminary analyses. The analyses were performed considering yield stresses of 2, 3, 4.5 and 6 kPa.

Figure 10a shows the considered section, based on Figure 4, with the landslide crater and the zone of debris. Figure 10b shows the observed debris and debris thickness obtained from the analyses. The following comments can be made:

- Strengths of 3, 4.5 and 6 kPa show debris close to 10 m in the crater, which does not seem to correspond to the observations. τ_y = 2 kPa gives a more realistically empty crater, but the runout distance is too long compared to the observations.

- The observations in terms of runout distance would correspond to strength of about 4 kPa.
- It appears on the basis of this simple analysis that the average strength of the flowing soil is somewhere between 2 and 4 kPa.

This average strength is slightly larger than the remoulded shear strength measured in the laboratory, which is logical as the entire volume was probably not remoulded. Also, in comparison to peak strength of 30 to 50 kPa (Figure 8), this indicates a sensitivity of 8 to 25, confirming the high sensitivity of the clayey silt.

For a landslide to be a flowslide, Tavenas (1984) gives several conditions that have been specified by Leroueil et al. (1996). These conditions are:

- To have a first failure. This, as seen above, might have been the case.
- Failure should provide enough potential energy for remolding the clayey silt. For Eastern Canada clays, this corresponds to a stability number $N_U=\gamma'H/S_U$ > 3 to 8, increasing with plasticity. In the present case N_U would be in the order of 4.5 and could explain remolding or at least partial remolding of the clayey silt.
- To have a liquidity index larger than 1.2, which is satisfied.

So, at least according to Eastern Canada criteria, it is possible that the Santana landslide was a flowslide.

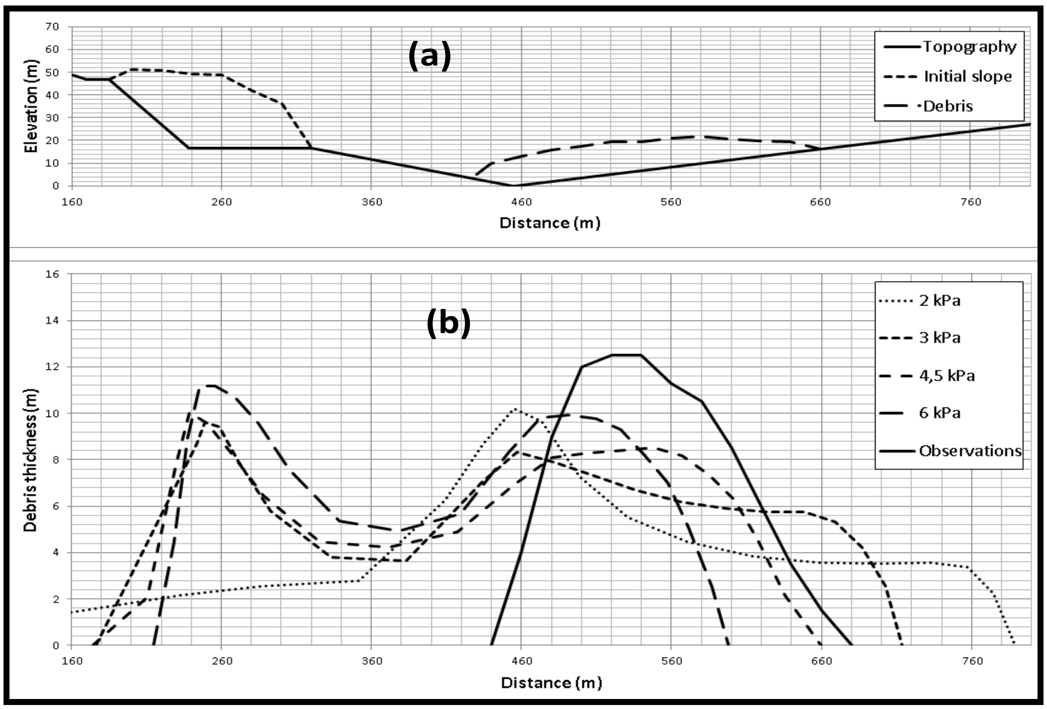

Figure 10. BING analyses: (a) Section; (b) Results.

5 FAILURE AND POST-FAILURE: DISCUSSION AND CONCLUSION

Several possible causes for the 2013 failure have been examined. No evidence has been found of river erosion, there was no dredging in the area and erosion caused by ships has been discarded by hydraulic studies. The region is free from relevant earthquakes. The pluviometry in the period that preceded the movement was normal. No relevant external loads have been applied to the failed area.

There was a previous smaller failure in 1993 which also happened without river erosion, dredging, earthquakes, excessive rain or applied loads.

Both the 1993 and the 2013 movements happened after a period of decades without signs of impending instability problems.

It is believed, by exclusion, that the best explanation for both, the 1993 and the 2013 accidents is strength deterioration by fatigue. Lacerda (1997) gave such interpretation to the Cabrito's hill landslide that occurred in residual soil in Rio de Janeiro in 1988. Demers et al (1999) also gave a similar interpretation to the Maskinongé slides. The deterioration could have been induced by cyclic loading associated with daily tidal water level fluctuations and with annual weight change associated with water level rise in the superficial fill. It can be imagined that degradation of safety factor happened with time. The steeper average slope in the area of the 1993 slide would propitiate a quicker deterioration rate. After the 1993 slide the average slope of the 2013 first stage slide area became steeper and together with an "instantaneous" reduction of the safety factor, it's velocity of degradation would also have been increased.

The flowslide that followed the 2013 accident is typical of sensitive clay. It does not seem to have precedents in soft fine soil deposits anywhere in the Brazilian coast or any other tropical climate areas, to the writers' knowledge.

Flowslide-prone clays are usually young fine grained sedimentary deposits composed of low activity minerals that have been deposited in marine environment and have been leached after uprising of the earth crust. The Santana soft soil is a clayey silt that has been deposited in the last 10000 years in fresh water (see Figure 6) and its major mineralogical components are quartz, kaolinite, chlorite, mica and illite according to X-ray diffraction tests (Barreto, 2015). It has thus not been deposited in marine environment and it has not been leached because the Santana soil has been deposited in fresh water (geological evidence) and water kept fresh to today (pore water salt very low, as shown in Figure 6).

Sensitive clay behavior is associated with a flocculated structure. Flocculation may occur during deposition in salt, brackish or sweet water (Torrance, 1983). In sweet water, a high sediment concentration would propitiate flocculation in the absence of adequate electrolytic conditions. Judging from the carbon dating and thicknesses of deposition (see Figure 6) the sedimentation rate of the Santana soft soil was around 2 cm per year between the depths of 20.5 m and 38.3 m. This rate is high in comparison with other soft soil deposits in the Brazilian coast and could be argued as having facilitated flocculation during deposition of the Santana soil (Quigley, 1980; Torrance, 1983).

Among the factors that may control sensitivity there are those that induce high undisturbed strength, such as cementation (Quigley, 1980). To identify the existence or absence of cementation one can look for indirect evidence. Scanning electron microscope images of the undisturbed Santana soil have been investigated with EDS and some iron oxides has been noticed at grain contacts (Barreto, 2015). On the other hand, as seen before, OCR values are close to unity which points to low (if any) cementation, except possibly at depths lower than 20 m where the undrained shear strength is about constant with depth and the estimated OCR values are in the 2 to 4 range.

In preliminary terms, and reckoning that much further study is needed, the origin of Santana's soil sensitivity that led it to behave as a sensitive clay during the 2013 movement, could be attributed to a flocculated fabric formed concomitantly with deposition, perhaps with some cementation with iron oxides introduced in the soil structure at or just after deposition.

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