Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-1877

Failure of a hangar built in embankment on soft clay

Rupture d'un hangar construit sur argile molle

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ABSTRACT
This paper presents the case of a hangar that was built in an embankment having 2000 m^2 and 6 m high. The subsoil is formed by a layer of organic clay and peat having 12 m of thickness. The hangar was constructed without knowing the bearing capacity and compressibility parameters of the clay under the embankment. The construction started with the ground being covered by a mixture of waste and remains of others constructions. A retaining wall of gabion was executed having 3 to 6 m of variable height in order to contain the embankment. During earthmoving operations, there was a slide in one corners of the ground and the soft clay and peat appeared upward. Several cracks were observed in the floor parallel to the retaining wall. Some fissures appeared in the masonry of the hangar. After some months later, the movement grew up accelerating and the complete ruin of the hangar occurred. The geotechnical characteristics of the clay deposit was made by borings (SPT), vane test and laboratory tests (oedometer and triaxial) in undisturbed shelby samples. The results of this investigation are presented as well as the back analysis of the failure.

RÉSUMÉ
Cet article présente le cas d'un hangar construit sur um remblai avec 2000 m^2 de surface e 6 m d'hauteur. Le sous-sol est formé par une couche d'argile organique très molle et de tourbe, avec 12 m d'éppaiseur. L'hangar a été construit sans la capacité portante e les parameters de deformation de l'argile. La construction a commence avec la couverture du terrain avec des déches e des restes de construction. Un mur de gabion a été construit avec une hauteur variable de 3 to 6 m , pour retenir lo remblai. Pendant le terrassement un glissement s'est produit et l'argile organique et la tourbe sont apparus a la surface. Plusieurs fissures parallèles au mur ont été observés. Quelques fissures sont apparues dans la maçonnerie de l'hangar. Des caracteristiques geotechniques de la couche d'argile ont été determines des sondages (SPT), essais scissiométriques, oedométriques et triaxiaux. Les résultats de l'étude et lánalyse du glissement son ici presentes.

1 INTRODUCTION

Embankment on soft soils has been studied by different authors all over the world (Marche and Lacroix, 1972; Coutinho, 1986; Leroueil et al., 1990; Gusmão Filho et al., 2001; Cavalcante et al., 2002) accumulating experience to understand the behavior of soft soil in loading.

Thus the construction of embankment on soft soil must present the following requirements: appropriate factor of safety against rfailure of foundation soil, some time before and after the construction; allowable displacements which are consistent with the type of construction both at the end of the construction and some time later; negligible damage to the adjacent buildings.

It is necessary to use the methods of analysis to attend these requirements in order to have a good behavior.

The area of the fill is situated at the border of a federal highway at the city of Recife, in the Northeastern coast of Brazil, and the site has an area of 10,000 m^2.

Three industrial buildings have been constructed. The biggest of them has an area of 2,000 m^2 and it is near the boundary of the area. Figure 1 shows the situation of the area when the works started.

The hangars were built by structures of prefabricated reinforced concrete whose columns had superficial foundations. The floor was made directly on the ground consisting of a concrete slab of 15 cm thickness.

The presence of soft clay and peat reached around 19 m deep. When the highway was built, explosives were used to help the fill to descent and to replace the mud below.

The embankment was built without one geotechnical project to prevent the problems of support and deformation. Several cracks started to appear on the ground parallel to the retaining wall since the beginning of the construction (Figure 2).

They have been closed immediately. Some fissures occurred in the masonry of the largest building that was situated next to the wall (Figure 3). There were also many openings of joints in the floor.

After some few months since the construction started, then a consulting geotechnical firm was called to present a solution. There were some evidences of damage made through local inspection. It was observed that the ground had two components of displacement.

One was the vertical movement of settlement of soft clay and peat due to the loading by the own weight of the fill. The other was the horizontal movement of these layers in the direction of the unevenness of the ground.

These lateral movements reflected on the structures showing cracks in the ground and openings of joints in the floor. The vertical movements from settlement caused fissures in the masonry when the columns settled.

With the damage diagnosis concluded, the consulting firm proposed two separate solutions. For the horizontal movement, it was recommended the construction of a berm in the lateral side of the area.

For the vertical movement, it was proposed to reinforce the foundations with steel piles.

In order to accompany the movements, it was asked also the instrumentation of the ground and structures with pins in the columns and surface marks in the ground.

However the berm and the foundation reinforcement were not done and the movements grew every day. After some time later, there was an acceleration of the movement of the ground and the complete ruin of the hangar (Figure 4).

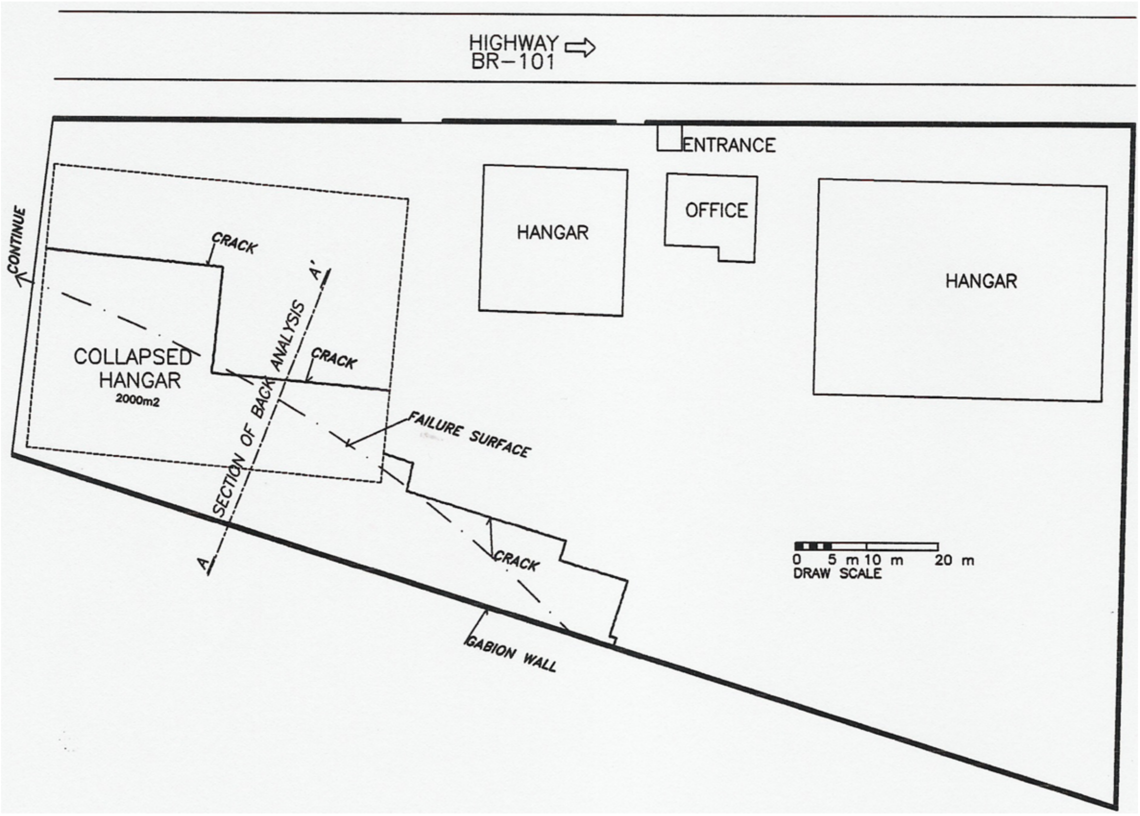

Figure 1 – Plan of the site with the failure surface.

2 GEOLOGY

The geology of the area is a large coast plain that is formed by quaternary sediments of marine origin due the advance and retrocession of the sea. The plain is crossed by rock outcrops in different places which are testimony of the resistance to the erosion.

The subsoil profile is quite variable in coast plain. The surface soil may cover deposits of compressible fine material having organic matter until great depth. The valleys of the rivers form the lower areas where the most recent sediments can be found. It is common the presence of soft clay in these areas due to the alluvium formation of these deposits. Tide ditches cross between the terraces of sand formed by the sea and they create swamp areas with the presence of very soft and deep deposits of organic clay and peat.

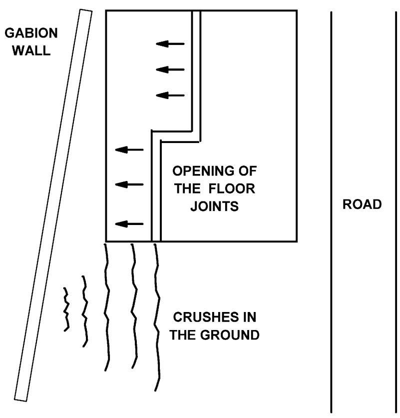

Figure 2 – Horizontal movements of the ground before the ruin of the hangar.

The tide ditch soils occur in the coast line near the river estuary and are influenced by tide movements. Soils are little developed and bad drained having a high content of salt that comes from water sea. They also have sulphur compounds that are formed in sedimentary low and swampy areas. Finally they show the occurrence of organic matter that is due to the decomposition of plants and others biological activities.

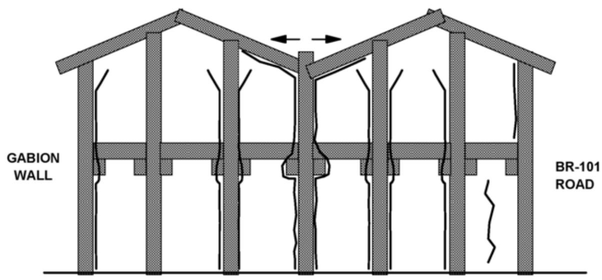

Figure 3 – Fissures occurred in the masonry of the hangar.

The local area is situated near the Capibaribe River forming a typical low area and it is has a subsoil profile that is characteristic of a tide ditch. The actual form of the deposit shows the importance of human action by the succeeding fills to permit use the ground as support of foundations of buildings and others engineering works.

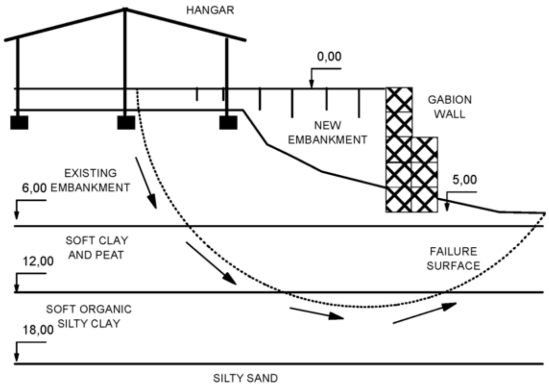

Figure 4 – Mechanism of the ruin of the hangar.

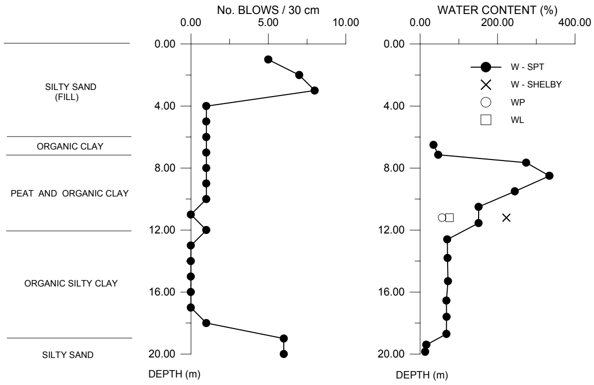

Figure 5 – Profile of the subsoil with SPT and natural water content.

3 GEOTECHNICAL INVESTIGATION

Eight borings were done at the place where the terrain ruptured. Six of them were made when the soil showed signals of rupture and the other two immediately after the collapse.

They show an initial layer of fill of sandy and silty clay that is bad compacted and is 6 m deep. It follows a layer of very soft silty and organic clay and peat that goes to the depth of 19 m. The silty clay gets medium to stiff from that depth on. The water table is situated around 4 m deep. Sand is presented in several borings at different depths. It is variable between 0.5 to 5 m thick without continuance among them.

Figure 5 shows a typical subsoil profile obtained with boring (SPT), including the natural water content of the soils that was determined by SPT sampler.

It can be observed the existence of two sublayers with different geotechnical characteristics (7 to 12 m and 12 to 18 m). It is also observed that the water content obtained by SPT sampler is smaller than that obtained by shelby sampler.

Field investigation also included natural water content, collection of undisturbed sample in shelby and vane tests.

Laboratory investigation consisted of tests of characterization, vertical consolidation and shear strength through triaxial tests types UU and CU.

Characterization tests showed that liquid limit and plasticity index were 76% and 19% respectively for the shelby sample (11.2 m deep). The content of organic matter was 67% and pH was 6.66.

3.1 *Consolidation parameters*

The consolidation test with vertical drainage was done in the undisturbed sample and its results are shown in Table 1.

In relation to the quality of the sampling, the unique sample was classified as very poor according to the approach of Lunne et al. (1997). The sample therefore it presented high disturbance degree.

It can be seen very clearly the disturbance effect on the preconsolidation pressure obtained from the samples. It presents a value lower than the vertical effective stress at 11.2 m deep and this indicates a false OCR lower than one.

Table 1 – Compressibility parameters.

γ (kN/m^3)	eo	Cc	Cs	σ'vm (kPa)	Cv (m^2/sec)
11.9	1.77	1.90	0.3	45	$1,2 \times 10^{-8}$

g – apparent unit weight; eo – initial void ratio; Cc – compress index; Cs – recompression index; s'vm – preconsolidation pressure; Cv – vertical consolidation coefficient.

3.2 *Strength characteristics*

Triaxial compression tests UU and CU were done to define the undrained strength (Su) and the angle of internal friction of the soil. Vane tests were also made in the field whose results are shown in Figure 6.

It is observed that the undrained strength in triaxial test UU has a value of 12 kPa that is below the value 33 kPa obtained by the vane test in the natural soil. However, it is very close to the vane test in the remolding soil (6.8 kPa after a large rotation in vane test). This proves that the laboratory samples were disturbed and their laboratory results should be used with special care. The effective angle of friction ϕ' was found equal to 15^o in CU test.

It is also noted that the deposit of soft soil presented a lower value of undrained strength around 12 to 14 m of depth (Su = 20 kPa). This indicates that the surface of rupture possibly passed at this depth. The result helped to have the back analysis of the problem with the stability of the fill to the bearing capacity of the maximum load.

4 BACK ANALYSIS OF THE FAILURE

After the failure it was possible identify on the ground the complete surface of rupture. In spite of the failure surface it was quite extensive (about 200 m), it was just made a two-dimensional analysis through the method of Bishop.

The adopted section for the back analysis is shown in Figure 1. The beginning of the failure surface was defined by the opening joints in the hangar floor, and the final of the surface was admitted where the ground moved upward. The correction proposed by Bjerrum (1972) was considered and the correction factor is 1.0 because the plasticity index is 19%.

Table 2 shows the parameters used in the back analysis. Figure 7 shows that FS = 1 and the circular surface passes at 12 m deep and it confirmed the vane test results, where it was noted a lower strength of the deposit at this depth.

This fact was explained with the history of the slide. After the first slide, the place was refilled to get leveling the floor. As a result, the place was submitted to a high loading without to offer an expressive increase in the bearing support of the soft soil. Then the fill collapsed sometime later.

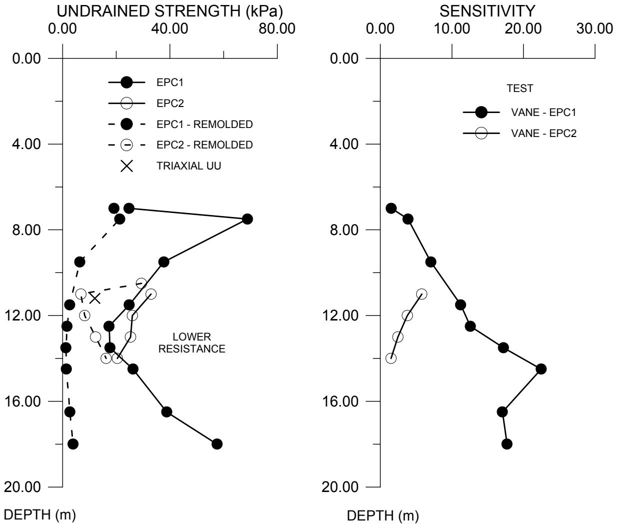

Figure 6 – Strength parameters obtained by laboratory and vane tests.

Table 1 – Compressibility parameters.

Material	g (kN/m³)	Coesion (kPa)	Friction Angle (°)
Fill	18	0	25
Soft soils	12	20	0

g – apparent unit weight.

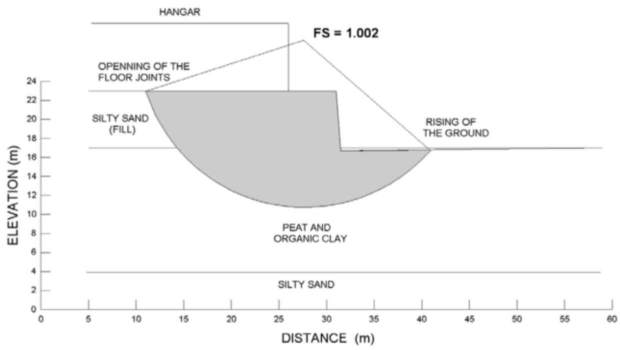

Figure 7 – Failure surface obtained from back analysis.

CONCLUSIONS

This paper presents the case of ruin of a hangar that was built in an embankment having 2000 m^2 and 6 meters high without geotechnical design. After the ruin, the geotechnical characteristics of the clay deposit was obtained by borings (SPT), vane test and laboratory tests (oedometer and triaxial) in undisturbed shelby samples. It is noted that the deposit of soft soil presented a lower value of undrained strength around 12 to 14 m of depth (20 kPa).

The back analysis of the failure showed that the circular surface passes at 12 m deep and it confirmed the vane test results, where it was noted a lower strength of the deposit at this depth. This fact was explained with the history of the slide. After the first slide, the place was refilled to get leveling the floor. As a result, the place was submitted to a high loading without to offer an expressive increase in the bearing support of the soft soil. Then the fill collapsed sometime later.

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