BEHAVIOUR STUDY OF MONITORED SOFT SUBSOIL UNDER EMBANKMENT LOADS

Aissi Adel, Bensihamedi Salim, Laboratory of Physical Metallurgy and properties of materials, University of Annaba-Algeria
Saihia Abdelah, Laboratory of soil and hydraulics, hydraulics department, University of Annaba-Algeria
Brikat Abdelghani, Laboratory of Physical Metallurgy and properties of materials, University of Annaba-Algeria

Abstract

This paper presents continuous monitoring behaviour results of soft subsoil under embankment loads. The primary objective of the analysis is to understand and analyze main factors which have lead to yielding of the embankment body during the construction phase.

Geological, hydrogeological and geotechnical investigations have been undertaken in order to better understand the behavior of embankment and their foundation before the rupture, vertical and side displacements of the embankment sub-soil as well as the variation of the water pressure for various depths are analysed. The latter are supported and checked by a numerical modeling with the soft soil model, the findings highlight various factors resulting in the failure of the infrastructure such as:-

The loading program during embankment implementation which has proven to be incompatible the ground conditions, which subsequently resulted in an excess of pore pressures. In addition, measurements taken on site have shown three main distinguished zones:

The failure zone d, with a height of 15 m have lead to a subsequent failure of zone b and c with a height of 12 and 10 m respectively.

The numerical modeling results have illustrated clearly the behaviour of the embankment particularly, the horizontal displacements and the interstitial overpressure which are underestimated by the measurements taken.

Keywords: Embankments, compressible soils, failure analyzes, overpressure interstitial, displacement, modeling.

1. Introduction

The socio-economic development of countries necessitates the improvement of their infrastructure and in particular the highways. The latter can only cross by large embankments based on soft ground and, thus, cause a serious challenge to the local authorities. This problems of the embankment stability on soft ground are known for a long time ago, such as the failure of the embankment of Scrapsgate, 1955; North Ridge Dam, 1957; and New Liskeard in 1963; Scottsdale, 1967; Palavas-lès-Flots, 1972; cited by (LEGRAND j & al, 1977) and many others. The difficulty of the behaviour's forecast is due to several sources of uncertainty; on a theoretical level; on the spatial inhomogeneity of soils, and the bad estimation of the parameters of the ground (Magnan et al, 1999; John Atkinson, 2007; Zhi-Liang Wang, 2007. These problems Paved the way towards the introduction of the embankments instrumentation during the construction so that we understand the real behaviour of embankment and its foundation by exploiting the mentoring measurements, such as La Rochelle et al, 1974; Blondeau et al, 1977; and Ramalho et al, 1983a; Brand & Premchitt, 1989; and also Lavallée et al, 1992; Rowe et al, 1995 and many others. This experimental embankment became in recent years a reference base to evaluate the capacity of development in numerical analysis and computer technology to perform a more realistic analysis of the behaviour of embankments constructed on soft soil. As mentioned in Chai and Bergado, 1993; Ph. Mestat, 2004; C.T. Gnanendran et al, 2006; Sukru, 2007; C YU and al, 2008; Paulo J, 2010; Valentina and al, 2011; Long Jin and al, 2012. Through the numerical modelling, we can rather important damage from a technical and economic point of view.

The failure of the embankment occurred in the Eastern Section of the National East-West Highway of Algeria, exactly at the kilometre point (PK) 384 km (Lot East). During the construction, when the embankment reached the height of 15 m, a tensile cracking appeared along the upper part of the embankment body. The study on the PK 384, where a rupture took place in July 2008, is a rare work that dealt with this problem in Algeria and the Arab world, which is the subject of this research in order to understand and analyse the main factors of embankments instability. To undertake our research, Geotechnical data on the ground were collected. The construction calendar as well as the measurements of the monitoring data obtained during the construction stage of the embankment before rupture was analysed. Finally, numerical modelling by finite element with the soft soil model was used to check the reliability of the results. This model used in our study is proved to be able to predict the behaviour of the short-term foundation as for former research in literature W. S. Tsai et al, 1998; YIN Zhen-Yu, 2009; Valentina and al, 2011; Jinchun Chai, 2012.

2. Materials and methods

2.1. Data Raised on the Conditions of Embankment

The National East-West Highway crosses the valley over a length of 2 km that is composed of compressible soil which is shown in the soil profile of the pk 384 in (Fig.2).

The National East-West Highway crosses over 2 km of compressible soil valley, which is shown in the soil profile in (Figure 1).

The topographical and climatologically characteristics promote an articulated system of drainage through the plains to The Mediterranean Sea; this location is classified as flood zones according to the Pacific Consultant International, 2009. The groundwater table is noticed to be between the surface and a depth of 2 meters.

The embankment has undergone during its implementation a settlement of 60 cm which was mainly related to the presence of clayed material in the twenty three meters of the subsurface. A survey conducted from extensive geotechnical investigation campaign on Soil foundation represented by in situ tests: (penetration tests (SPT) performed in accordance with the NF P94-116 standard, a Menard-type pressure-meter and in accordance with NF P94-110 standard), as well as laboratory tests: physical and mechanical. Both in situ tests and laboratory tests are carried out by the Geotechnical laboratory FONDASOIL, 2007, Public Works Laboratory - direction of Constantine, and COJAAL Camp 7 soil Laboratory, and completed by further investigation in 2009 after the failure of the embankment infrastructure.

Because the subsoil was affected by the rupture of embankment, and in order to use the representative parameters, the characteristics selected are taken before the rupture. As a result, the subsurface geological data on the site reveal the existence of four main soil layers with variable thickness. An accumulation of clay layers is found from the surface up to a depth of 23 meters. The last five meters of this clay layers include a small layer of deteriorated quality of sand, which is followed directly by a four-meter thick layer of sand. The substratum is a composed of sand and gravel, which extend to a depth of 35-50 meters.

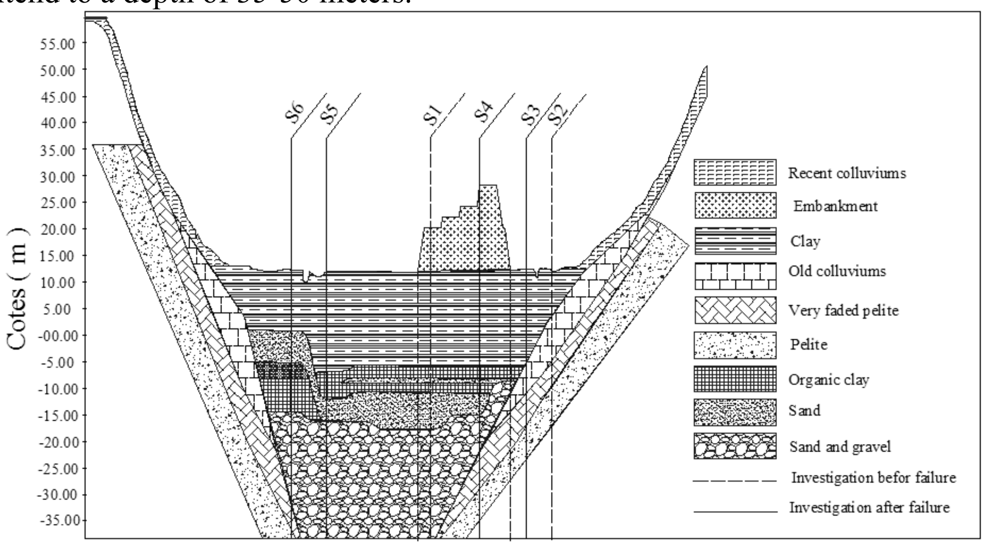

Fig.1 Longitudinal cross-section of the compressible Valley

2.2. Geotechnical properties of the subbase

The results obtained from the odometer tests have led to the following main conclusions: First of all, the clay is classified as compressible soil with a compressibility index of Cc ≈ 0.25 - 0.4. Second, the pre-consolidation pressures obtained from odometer tests indicate that the deposit is over-consolidated in the upper parts, where the over-consolidation ratio OCR=1, 6 to 3 as a result of the flood in the winter, it decreases with depth, and finally reaches an under-consolidation OCR≈0,97 to 0,57 below elevation -07 m. The natural water content of the soft clay layer varies from 20 - 25%, liquid limit 45 to 50% and plastic limit 23 to 29%. The typical principal Geotechnical characteristics of subbase soil as a function of depth are illustrated in (figure 2).

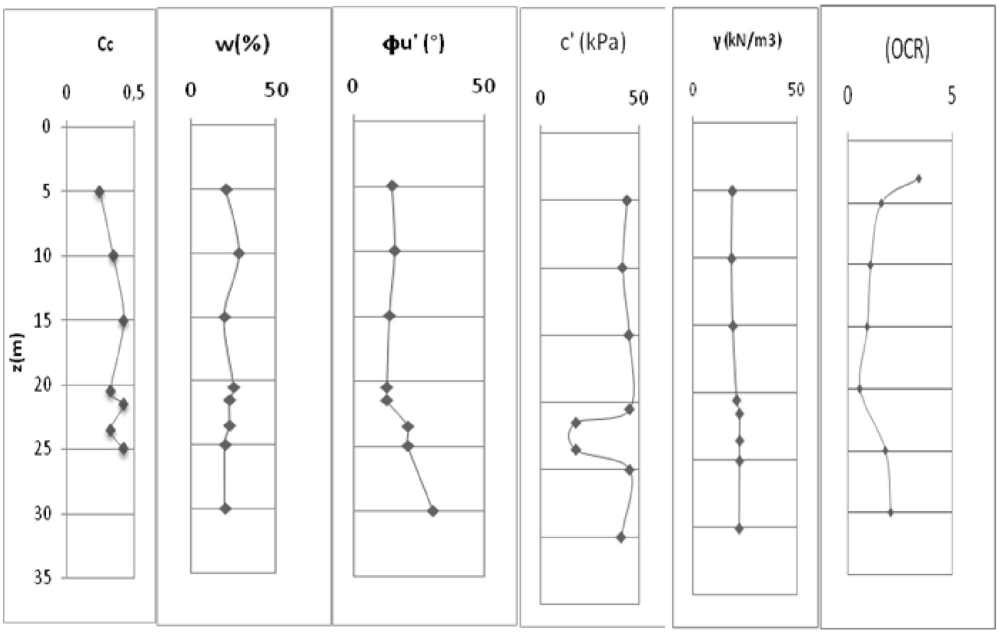

Fig. 2 Geotechnical characteristics of subbase soil as a function of depth.

2.3 Measure monitoring before rupture

The ground monitoring aims at understanding the settlement of the embankment in detail, and to supervise the stability of embankment site. In order to simplify the analysis, the embankment body in construction is divided into four zones (A, B, C, D) according to their different heights , i.e, four different conditions of loading (figure 3)

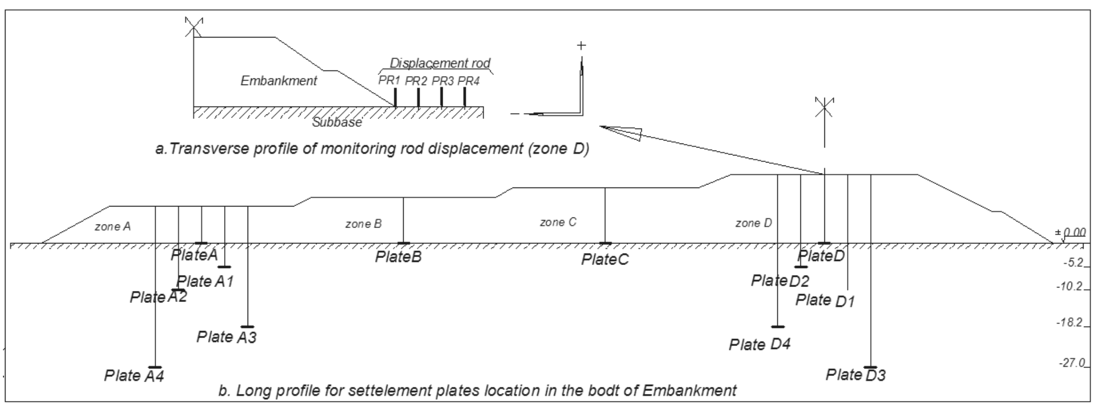

Fig. 3 monitoring instruments' position

3. Results and discussion

2. 1. Failure Mechanism Analysi

Beside hydrogeological, geological and Geotechnical detailed studies of soil, and for a reliable analysis of the main factors that led to the embankment failure, the following points are clarified:

the height of each embankment zone (geometry), methods and execution schedule, the exploitation of dynamic observation before the failure, which are supported by Finite Elements numerical modelling with staged construction in PLAXIS to achieve the real conditions of construction I.C. CHAI and D.T. BERGADO, 1992; Mohammad AL HUSEIN, 2001. By analysing the evolution of different failure phases, a weak bearing capacity at the subsoil embankment level is revealed. The relationship between the load and the settlement of subsoil is shown in (figure 4). The ultimate limit state where the load is at 5m in the settlement plate D and D2 is positioned with the lower part of the thick clay layers (figure 3). The displacement of D and D2 settlement plate is very large: equal to 30cm. In this condition, the loading stopped for about 10 days, which allowed the consolidation of the ground and became capable to support more loading. Moreover, the over-consolidation state of the superior layer of subbase prevented the appearance of the crack in the embankment.

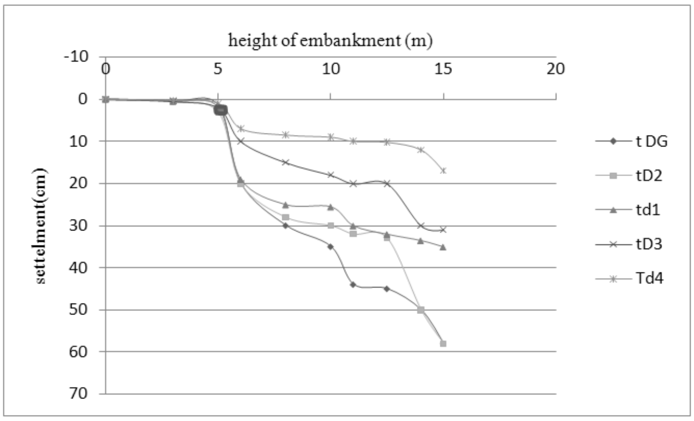

Fig. 4 Curve of loading deformation of subbase under embankment of different depths.

Through observing the rupture stage on the site, some signs of instability are shown by the appearance of a 2 cm wide crack in the middle of the embankment summit in D zone and in a part of C zone. Meanwhile, the cracking was accompanied by an explosion which generated a vibration. The eyewitnesses confirmed the intensity of the vibration which led them to leave their houses.

Through the analysis of the curves displacement with time (Figure 5), we notice the following observations:

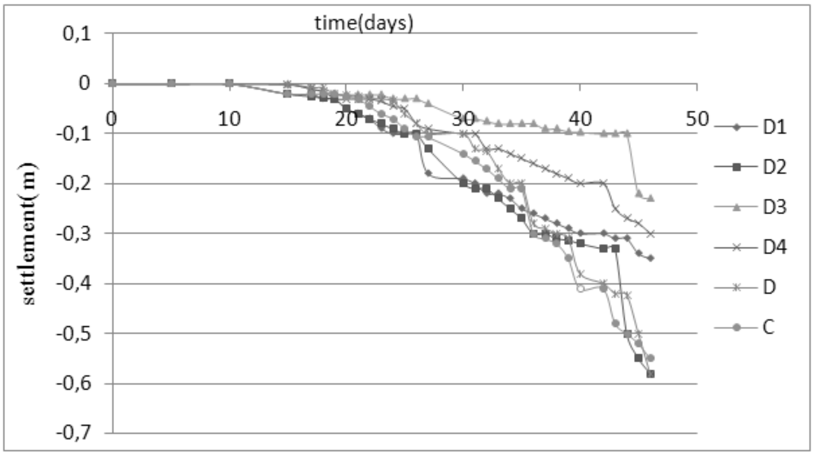

Fig. 5 Results of the vertical displacements by the settlement plate of the zone C and D.

At the initial stage of construction, the settlements beneath the embankment are small and so is the lateral displacement. This is due to the over-consolidated state of the soil, and with the next loading, we move to the second stage life of the embankment on the compressible soil as mentioned by: Bujang B and K Huat, 1993; Gavan Hunter and Robin Fell, 2003; C YU and al 2008. The subsoil becomes normally consolidated with an undrained behaviour, and caused an important vertical (57 cm) as well as lateral displacements.

The measured lateral displacement bay the stakes represents only the superficial displacements of the subsoil (figure 6). However, the real displacement was in depth and manifested in a heave up of about 1 meter under the embankment (figure 7) by the effect of concentration of shear strain beneath the embankment slope, Ramalho-Ortigao et al, 1983.

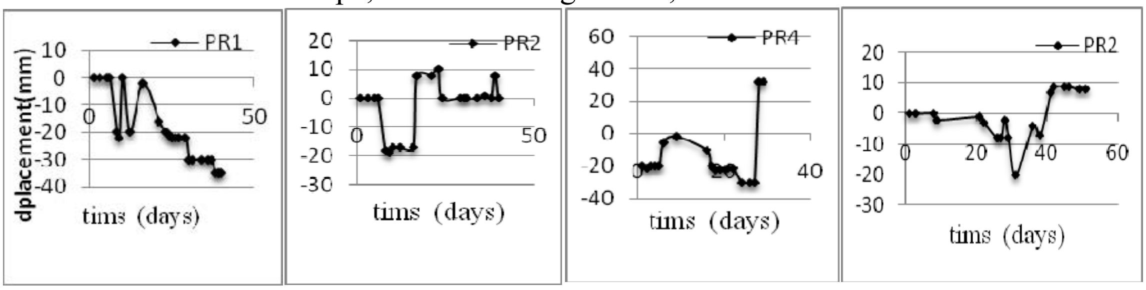

Fig. 6 Horizontal displacement of the stakes

Fig. 7 Heave up on the embankment's foot to 15m high.

The understanding of lateral displacements is a good indicator for the comprehension of the embankment behaviour of soft soil as confirmed F. Tavenas and S. Leroueiel, 1980; Gavan Hunter and al, 2003; C YU and al, 2008. However, the installation of inclinometers becomes necessary for the comprehension of lateral movements of different depths under the embankment. Bujang B and K Huat, 1993; Gavan Hunter and al, 2003; Sukru Ozcoban and al, 2007.

3. 2. Loading program of embankment

After the exposition of the situation before and during the rupture, the condition of the loadings is analysed according to the loading program of embankment (figure 8). The (figure 8) shows that embankment are constructed with rapid rates of 0,3m/day which is similar to the time construction of Kalix 0.33 m/day and Portsmouth 0.19 – 0.42 m/day Gavan Hunter et al, 2003. The loading program, thus, must take a longer period than the one imposed by the authorities; this means that this technique is inacceptable without treating the subbase.

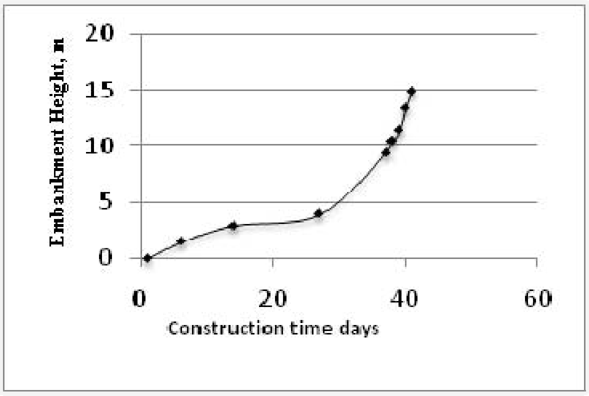

Fig. 8 Embankment implementation in relation to time

Another factor, which is so important for the analysis of embankment behaviour on the soft soil especially in our case, is the excess pore pressure. The latter occurs during the initial construction phase with a rapid loading rate equals to 0.3 m/day and almost an undrained behaviour which affects the process of excess pore pressure dissipation F. Tavenas and S. Leroueiel, 1980; Gavan Hunter and al, 2003.

Under these circumstances, no piezometrical measurement is taken into consideration as a monitoring criterion, and the reported explosion at the moment of failure signals the instantaneous dissipation of interstitial excess pore pressure.

To reproduce the scenario of the interstitial pore pressure generation under the embankment section as a function of applied load, the method developed by Tavenas and Leroueil, 1980 is used and supported later by finite elements numerical modelling, which allows us to shed light on the state of interstitial overpressure during the different phases at different depths of construction. The result is shown in (figure 9). In the early stage of construction, the effective stress level is less than the preconsolidation pressure (σp), σp is characterized by a low pore pressure B<1 (figure 9), and with the loading, the subsoil becomes normally consolidated.

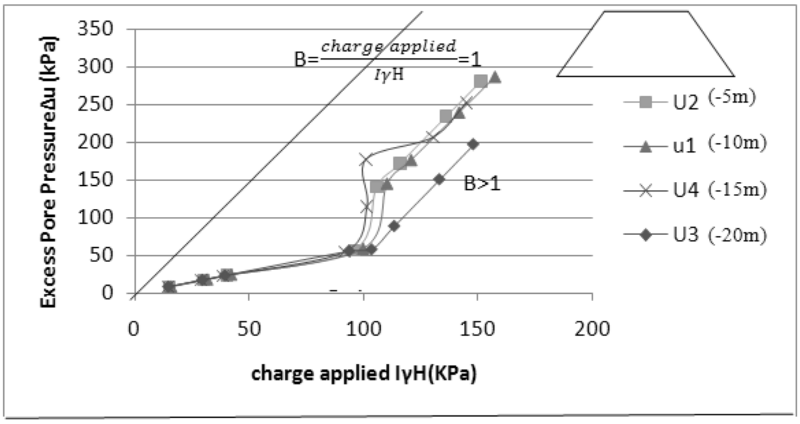

Fig. 9 Pore pressure calculated under the centre of embankment of different depth, according to the embankment load.

In this stage of the loading, the clay is characterised by high compressibility and low coefficient of consolidation (Cv) is resulting in low rates of excess pore pressure dissipation Shui-Long Shen and al, 2005 shown in (figure 9). Where B>1 is similar to the results of F. Tavenas and S. Leroueiel, 1980; Bujang B. K Huat, 1993; Gavan Hunter and al, 2003; Shui-Long Shen and al, 2005. However, the variation of the effective pressure on the subbase is assured and allows the development of the driving forces necessary to the vertical displacement, combined with the horizontal displacement of the subsoil. These factors transfer this movement to the embankment and generate the crack which is apparent at the level of the embankment (figure 10) similar to the rupture of embankment as shown in Lanester-le-Rohu Morbihan, 1969 and Muar-F Test Embankment Brand Premchitt, 1989;

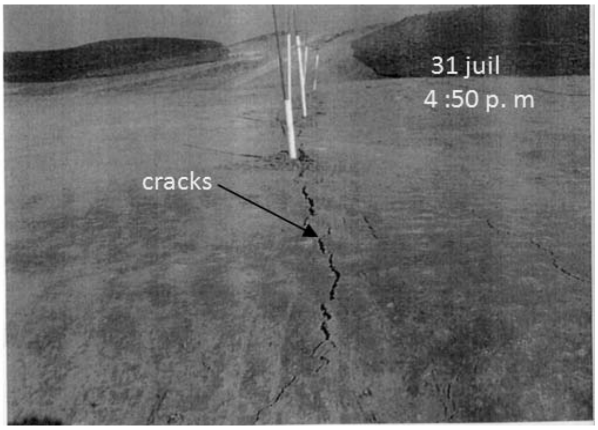

Fig. 10 Cracks on the 15m high embankment's surface.

The analysis of the displacement curves of the different zones (figure 5) and their shape on the site in different rupture phases, has shown that great displacements occur in the zone with great height of embankment (zone D), followed by the appearance of crack, and the creation of interstitial overpressure, which cause the release of the rupture in this zone whose influence reaches the zones of B= 10m, C=12m (figure 11).

Fig. 11 Final Deformation of embankment.

3.3 Numerical modelling with finite element method

Because of the irrelevance observed in the measurements taken by the stakes of horizontal displacement, and to strengthen the calculations of pore pressure which are alienated and neglected at the time of construction in spite of the availability of the measuring instruments (piezometer) which are installed on the ground, as well as the importance of the stability monitoring evaluation, we tend towards a numerical modelling by finite element with the soft soil model, due to its simplicity and its wide use. The soft soil model proves to be able to predict the behaviour of the short-term foundation loading. W.S.Tsai et al, 1998; YIN Zhen-yu, 2009; Valentina and al, 2011; Paulo J. Venda Oliveira, 2010; Jinchun Chai, 2012.

Based on the geotechnical parameters presented in (table 1) which give the results of the pore pressure evolution during the various phases by an undraind response (Figure 12) different from that observed in site in the timing of excess pore pressure peak, because of the partial drainage whilst the foundation is in on over-consolidation condition Leroueil and Tavenas 1986, Jardine and Hight 1987, Folks and Crooks 1986. This condition of interstitial overpressure is the result of the explosion heard at the rupture time of the brutal effect of the excess pore pressure.

Table 1. Parameters for soil used in numerical analysis

Stratigraphic nature of level	Model	γ (kN/m3)	*e0*	φ°	*Kv* (m/s)	*Kh* (m/s)	Cs	Cc	C	K0

Clay	SSM	18	1,64	16	$1,9.10^{-6}$	$1,9.10^{-6}$	0,006	0,246	41	0,72
Organic clay	SSM	19	1,97	13	$2,3.10^{-6}$	$2,3.10^{-6}$	0,085	0,471	17	0,77
Ground sand spreader	SSM	18,5	0,7	13	$1,8.10^{-5}$	$1,8.10^{-5}$	0,064	0,359	17,8	0,77
Organic clay 2	SSM	18,9	1,97	13	$2,3.10^{-6}$	$2,3.10^{-6}$	0,099	0,55	17	0,77
Ground sand 2	SSM	18,5	0,7	13	10^{-5}	10^{-5}	0,030	0,344	17,8	0,77
Colluvium	SSM	22	0,5	30	10^{-5}	10^{-5}	0,019	0,096	25	0,5
Colluvium 2	SSM	22	0,7	35	10^{-5}	10^{-5}	0,019	0,096	40	0,5
Embankment	Mohr Coloumb	21	0,81	25	10^{-4}	10^{-4}	-	-	5	-

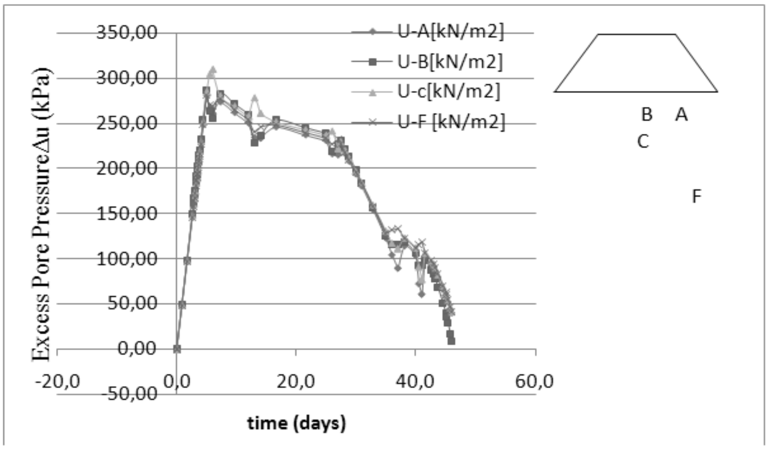

Fig. 12 Excess pore pressure variation esteemed bay FEM.

Horizontal displacements at depth are well presented by the model and gave results that agree with the behaviour on the ground (figure 13).

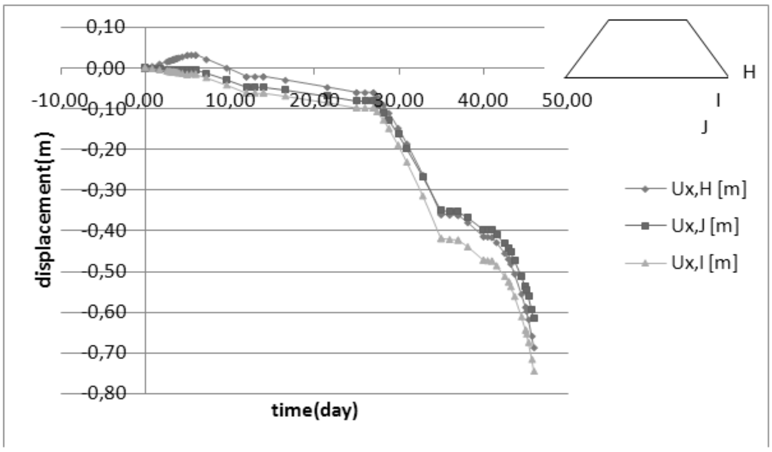

Fig. 13 The variation of lateral displacement with time esteemed by FEM**.**

4. Conclusion

The experiences acquired from the ruptures in the embankments on compressible ground demands special precaution related mainly to the behaviour of the loading effect, especially when dealing with a project that is famously known in Algeria as “the Project of the Century”. The analysis of the rupture in the embankment of Pk 384 shows that the rupture occurred by the effect of the rapid rate of loading caused the generation of interstitial overpressure, as well as the horizontal and vertical displacements in the compressible layer of the zone D.

The underestimation of monitoring measurements hides the real behaviour of the foundation. The provision of the displacement stakes besides the embankment enables to predict the real behaviour that we seek to follow. However, the installation of the inclinometers would be necessary under this condition in order to detect the horizontal movements at various depths beside the embankment.

The importance of the project requires the professionalism of the people in charge in order to avoid greater damage like that which has occurred previously. Moreover, the fact of neglecting the monitoring of the development of interstitial overpressure with time in spite of the availability of the instruments necessary reflect the fact of the lack of professionalism in the developing countries.

Numerical modelling can predict obviously the scenarios and the circumstances of rupture and their precision is related to the quality of data available and the digital techniques used in calculations.

REFERENCES

1.AL Hussein, M. Etude du comportement différé des sols et des ouvrages géotechniques . – Doctoral Thesis, University of Josèphe Fourier-Grenoble I, 2001. – pp. 259.

2.Berengo, V., Benz, T., Simonini, P., Leoni, M. Site monitoring and numerical modelling of a trial embankment's behaviour on Venice lagoon soils. – I.S.R.N Civil Engineering 2011. – pp. 11.

3.Chai, J.C., Bergado, D.T. Some techniques for finite element analysis of embankments On soft ground. Can. Geotech. J 30, 1993. – pp. 710-719.

4.Chai, J., Igaya,Y., Hino, T., Carter, J. Finite element simulation of an embankment on Soft clay – Case study. –Computers and Geotechnics 48, 2012 . –pp. 117–126.

5.Tavenas, F., Leroueil, S. The behaviour of embankments on clay foundations. – Can.Geotech. J 17, 1980. –pp. 236-260.

6.John, A. The Mechanics of Soils and Foundations, 2nd Edition. Taylor & Francis.2007. – pp. 441. Huat, B.B.K. Behaviour of Soft Clay Foundation beneath an Embankment. –Pertanika J. Sci. &Technol 2, 1994. –pp. 215-235.

7.Hunter, G., Fell, R. Prediction of impending failure of embankments on soft ground. –Can. Geotech. J 40, 2003. – pp. 209-220.

8.Hunter, G., Fell, R., Khalili, N. The deformation behaviour of embankments on soft ground. The University of New South Wales Report, Sydney, 2000. – R-391.

9.Huang, W., Fityus, S., Bishop, D., Smith, D., Sheng D. Finite-element parametric study of the consolidation behavior of a trial embankment on soft clay. – International Journal of Geomechanics 6, 2006 . –pp. 328-341.

10.Indraratna, B., Balasubramaniam, A.S., Balachandran, S. Performance of test embankment constructed to failure on soft marine clay. –International Journal of Geotechnical Engineering 118, 1992 . – pp. 12-33.

11.Gnanendran, C.T., Manivannan, G., Lo, S.C.R. Influence of using a creep, rate, or an elasto-plastic model for predicting the behaviour of embankments on soft soils. –Can. Geotech. J 43, 2006. –pp. 134-154.

12.Mestat, Ph., Bourgeois, E., Riou, Y. Numerical modelling of embankments and Underground works. –Computers and Geotechnics 31, 2004. – pp. 227-236.

13.Oliveira, P.J.V., Lemos, L.J.L., Coelho, P.A.L.F. Behaviour of an atypical embankment on soft soil: field observations and numerical simulation. – J. Geotech. Geoenviron. Eng 136, 2010 . – pp. 35-47.

14.Ozcoban, S., Berilgen, M., Kilic, H., M.ASCE, T., M.ASCE, I. Staged Construction and Settlement of a Dam Founded on Soft Clay. – Journal of geotechnical and geoenvironmental engineering 133, 2007. – pp. 1003-1016.

15.Pacific C.I. rapport specifique de remblai dans les zones inondables et humides. Autoroute Est-Ouest de l'algérie PK 384, 2009. – pp. 29.

16.Pilot, G. Etude de la rupture de cinq remblais sur sols mous. –Bull. Liaison Labo. P. et Ch 64, 1973 . –pp. 89-101

17.Shen, S.L., Chai, J.C., Hong, Z.S., Cai, F.X. Analysis of field performance of embankments on soft clay deposit with and without PVD-improvement. – Geotextiles and Geomembranes 23, 2005 . – pp. 463-485.

18.Tavenas, F., Roy, M., La Rochelle, P., Leroueil, S., Trak, B. Les remblais d'essais de Saint-Alban. – Bull. Liaison Labo. P. et Ch 98, 1978 . –pp. 42-52.

19.YU, C., PAN, L.Y., DU, G.Y., Lei, C. Numerical simulations on the lateral deformation in soft ground under embankments. –ProcDeparture of participants. of SPIE, ., 2009, 73750M.

20.Zhi-liang, W., Yong-chi L., R.F, Shen. Correction of soil parameters in calculation of embankment settlement using a BP network back-analysis model. –Engineering Geology 91, 2007. – pp. 168-177.