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THE STABILITY AND DEFORMATION LIMIT STATE CORRESPONDING TO THE HIGH ROAD EMBAKMENTS CLOSE TO A BRIDGE

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ABSTRACT

The paper presents the analysis of a road embankment with variable height (between 4-10m) placed at Iassy, Romania, in the first year of service. It's about access ledge at a bridge with total length of about 300m. At this time the embankment presents longitudinal cracks parallel with road axle, lateral knobs corresponding to each compacted layer, infiltrations through the backfill from the top (faulty pluvial system) and from the bottom (flooded foundation soil without drainage system).

Foundation soil is a metastable clay. Triaxial tests type CKoD on stress loading paths have shown that this soil is sensitive to moistening at shear stresses. The embankment is made also from a clay unusually used for such type of structures and several geosynthetic layers. At the top there is an elastic pavement. The footwalks over the embankment are from reinforced concrete and are bracket assembled. The slopes are 10:1 and are air-placed concreted.

The finite element model has taken into account various hypothesis: 1. Model with the soils in natural state, 2. Model with foundation soil in flooded state, 3. Model with foundation soil in flooded state and different artificial consolidation on embankment width. The article presents stresses, strains, and also the stability analysis in seismic conditions imposed by P100-2006.

It also presents a prognosis of general state under seismic conditions, analysed limit state of stability and also limit state of bearing capacity of foundation soil.

INTRODUCTION

The object of this paper is to present the analysis of a road embankment with variable height (between 4-10m) placed at Iassy, Romania. This embankment presents from the first year of service longitudinal cracks parallel with road axle, lateral knobs corresponding to each compacted layer, infiltrations through the backfill from the top and from the bottom. The owner employed an investigation team to identify, analyse and propose consolidation work for this embankment.

Investigation team lead by the first author paid a visit to establish the "to do" list. First of all we identify the problems named before. After this we have made an geotechnical study to identify correctly the soil parameters, dimensions of foundations and water level. Based on this parameters we were able to do an analysis of this embankments using Plaxis software. The analysis contains the following models: 1. Model with the soils in natural state, 2. Model with foundation soil in flooded state, 3. Model with foundation soil in flooded state and different artificial consolidation on embankment width.

Fig. 1. General view of the embankment

Fig. 2. Lateral view of the embankment.

Fig. 3. Longitudinal view of the embankment.

Fig.4. Zone between embankment - bridge.

Fig. 5. Lateral view of the embankment.

2. GEOTEHNICAL INVESTIGATIONS

Geotechnical studies shows the followings:

- the lithology of soil is: vegetable soil 0,5m, black/yellow plastic clay for up to 5,00m (Bahlui clay), saturated sand, saturated sand with gravel (5-7m) and marl clay from 12m;
- underground water from 2-4m from terrain level, this level can be ascensional with 0,8m;
- peak ground acceleration a_g=0,2g, Tc=1sec (P100-2006);
- Bahlui clay is very active, with high compressibility and big variations of volume (shrinkage-belly);
- plasticity index $I_p = (30\div45)\%$;
- saturation degree $S_r = 0.80\div0.90$;
- oedometric modulus $M_{2-3} = 4.000\div10.000$ kPa;
- modulus of linear deformation $E \approx 50.00$kPa;
- dry volumic weight $\gamma_d = 14.8\div15.5$kN/m^3;
- natural volumic weight $\gamma = 18.75\div19$kN/m3;
- porosity $n = (40\div45)\%$;
- void ratio e = 0.838;
- angle of internal friction $Ø = 12^o\div16^o$;
- cohesion $c = (15\div25)$kPa.

For construction supervision of soil works have been made the following tests:

(a) tests in open system (CK_0D), for which the specimens during shearing until breaking have been in contact with water from the beginning, soil being free to change his humidity with the raising the intensity of shearing force.

(b) tests in closed system (CK_0D-A), for which the specimens during shearing untill breaking have been in natural state humidity without any contact with a free source of water.

For both type of tests the specimens are consolidated under stress states corresponding to "K_0 line", after which they are sheared as presented above.

The following picture presents stress paths for all specimens and also the results obtained, for parameters of shearing resistance for both loading systems.

We can see that on both loading systems, in the zone of normal stresses σ' < 0,8 daN/cm^2 intrinsec curve has big values for angle of internal friction and low values for cohesion and in the zone of normal stresses σ' > 0,8 daN/cm^2 situation is reversed. Also it can be seen that for closed system of testing intrinsec curve near the origin of axis Bahlui clay has values 4 times bigger for apparent cohesion c', and in the zone of normal stresses σ' > 0,8 daN/cm^2 presents values a little bigger for apparent angle of internal friction ϕ'.

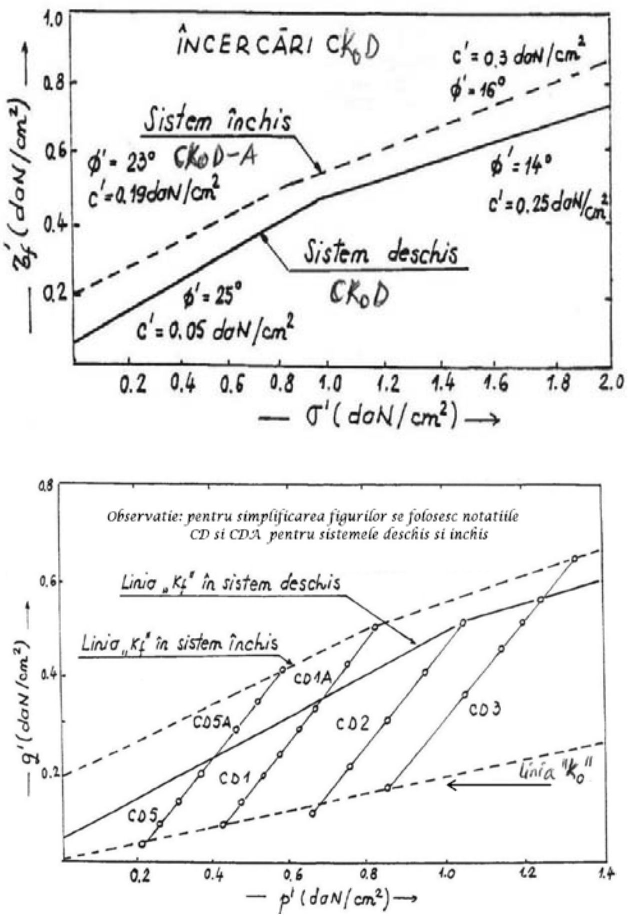

Fig. 6. cK_0D tests.

From Fig. 7 it can be seen that, invariable, the specific volume deformation of specimens tested in open system, correspond to a reduction of void ratio through shearing and for specimens tested in closed system specific volume deformation correspond to a mechanical growing – dilatancy who appears in a specified point in load path function of lateral pressure σ_3. Also it is important to note that dilatancy appears when deviatoric stresses q is in direct rapport with spheric stress p and volume variation depends of q. This experimental observation has a great practical importance because it shows the zones in which dilatancy occurs function of the report between deviatoric stress and spheric stress.

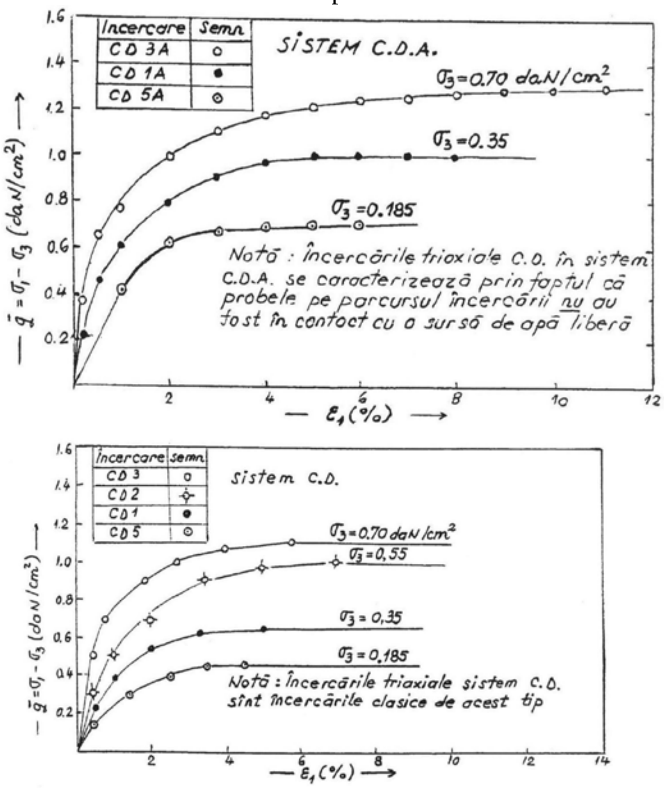

Fig. 7. $\varepsilon_{ax} \div \bar{p}$ $(\varepsilon_1 \div \bar{q})$ relations.

In conclusion material properties for analysis are:

1. Bahlui clay:
   a. Dry state:
- γ=17kN/m^3
- Ø=23°
- Cd=20kPa
- E=15.000kPa
- ν=0,30
   b. Flooded state:
- γ=21kN/m^3
- Ø=25°
- Cd=5kPa
- E=5.000kPa
- ν=0,35

2. Backfill for embankment:
- γ=20kN/m^3
- Ø=20°
- c=50kPa
- E=18.000kPa
- ν=0,30

3. Loose backfill:
- γ=20kN/m^3

- Ø=20°
- c=50kPa
- E=10.000kPa
- ν=0,30

4. Stone layer:
- γ=20kN/m^3
- Ø=25°
- c=1kPa
- E=30.000kPa
- ν=0,30

5. Asphalt:
- γ=22kN/m^3
- E=20.000kPa
- ν=0,20

Loads are:
- self weight,
- on road – 100kN/m^2,
- on sidewalk – 10kN/m^2.

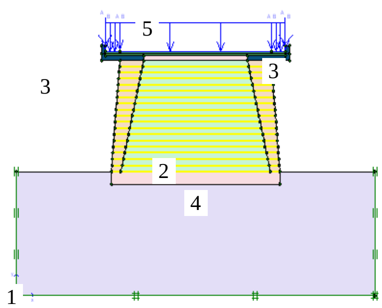

Fig. 8. General section with materials.

4. FINITE ELEMENT ANALYSIS

The analysis was made using PLAXIS software. Model was plane strain with 15 node elements.

The analysis was made to predict future behavior of the embankment. Different models were taken into consideration taken into consideration the following:
- foundation soil of embankment is almost every time of the year flooded. Bahlui River is not flood controlled in that area (Fig. 7).
- backfill was loose on the edges of the embankment due to the lack of technology used in civil works (Fig. 6).

Fig. 9. View from the top of embankment to the right side.

The 3 models taken into analysis are:

a) MODEL 1. Model with soils in natural state
b) MODEL 2. Model with foundation soil in flooded state
c) MODEL 3. Seismic response due to earthquake with foundation soil in flooded state

a) MODEL 1. Model with soils in natural state.

This model is the simplest model taken into consideration. This means that the properties of materials is in natural state.

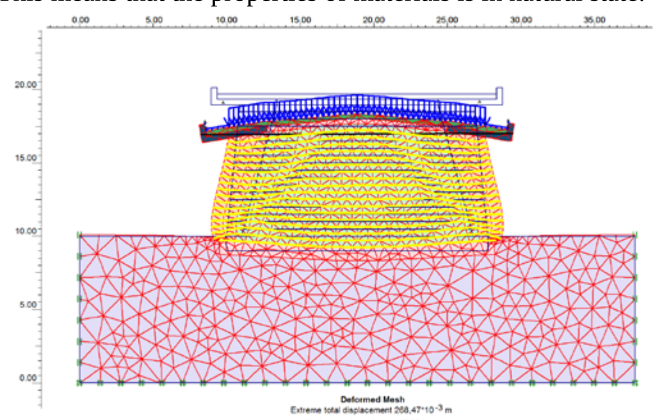

Fig. 10. Deformed mesh.

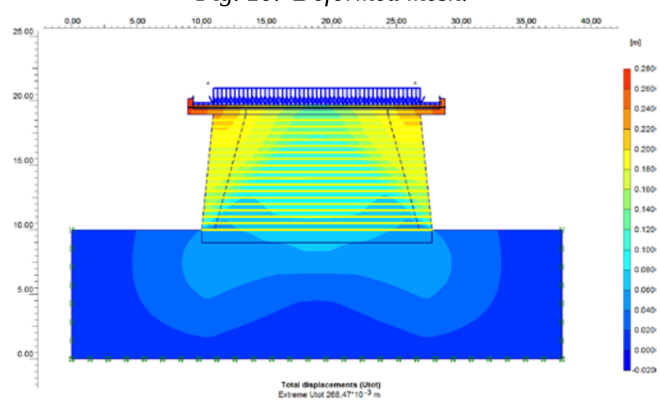

Fig. 11. Total displacements.

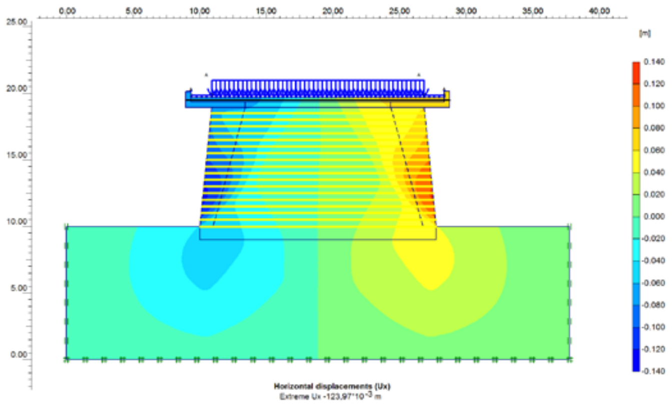

Fig. 12. Horizontal displacements.

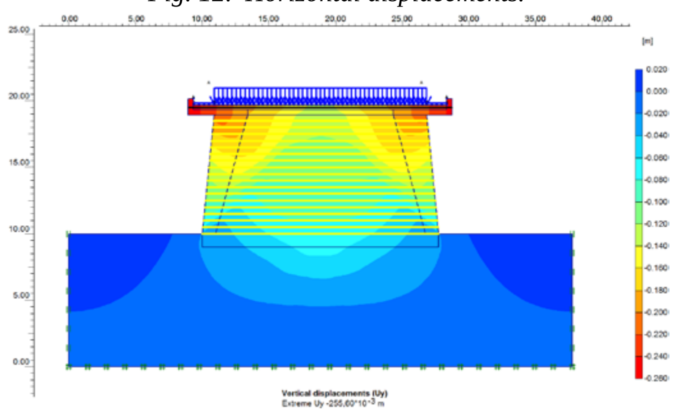

Fig. 13. Vertical displacements.

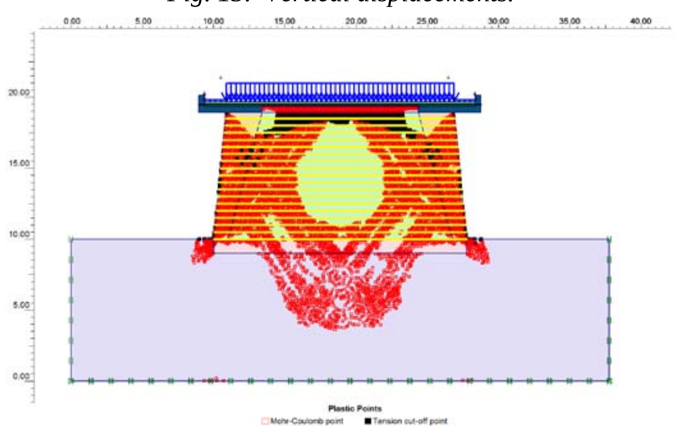

Fig. 14. Plastic points.

The conclusion of this calculus is that the maximum total displacement is 280mm (at the edge the sidewalks - Fig. 9). As it can be seen in Fig. 10 the plastic points appear at the edges of the embankments and in the central part of soil foundation at an approx. depth of 5m. Also, a very important notice is that the plastic points also appear at the edge of the embankments at maximum 1m around the body of backfill.

b) MODEL 2. Model with foundation soil in flooded state.

This 2nd model is taken into consideration the flooded state of materials.

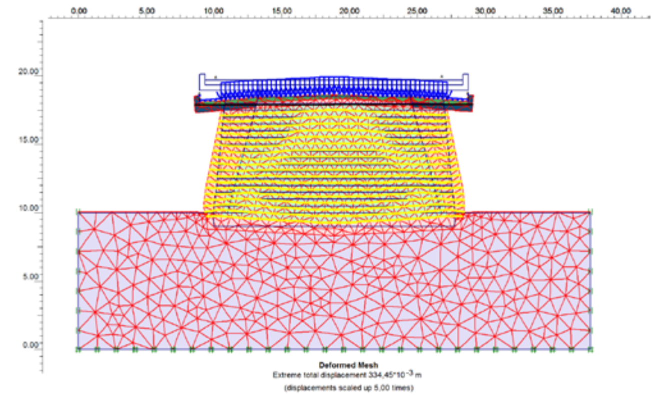

Fig. 15. Deformed mesh.

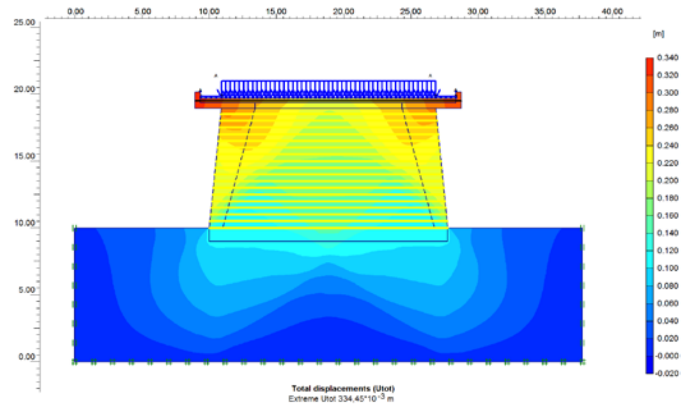

Fig. 16. Total displacements.

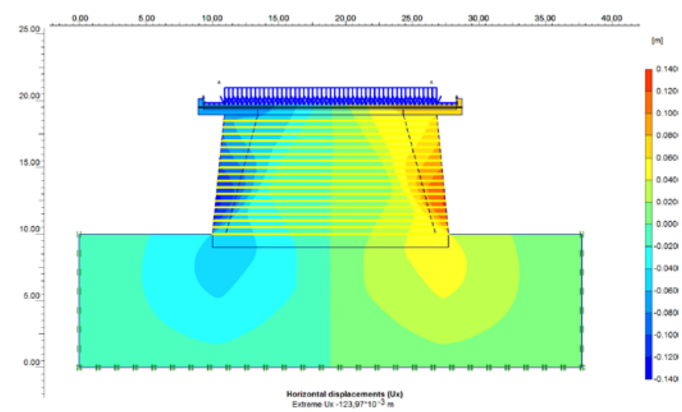

Fig. 17. Horizontal displacements.

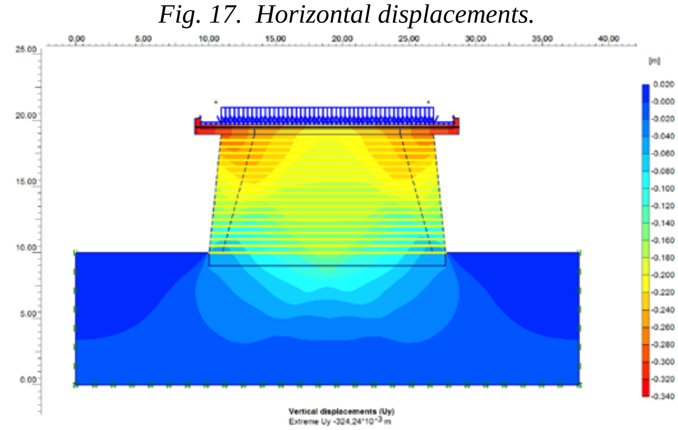

Fig. 18. Vertical displacements.

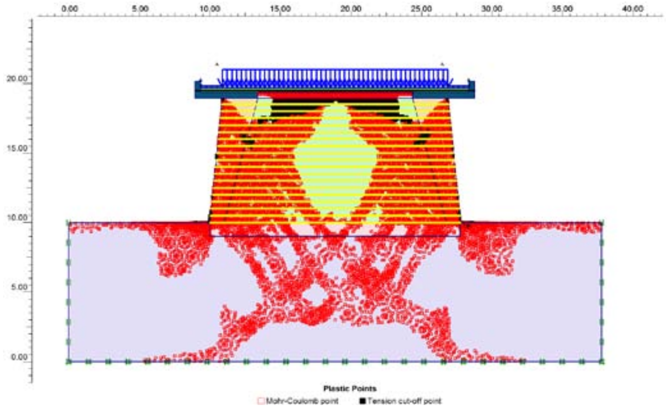

Fig. 19. Plastic points.

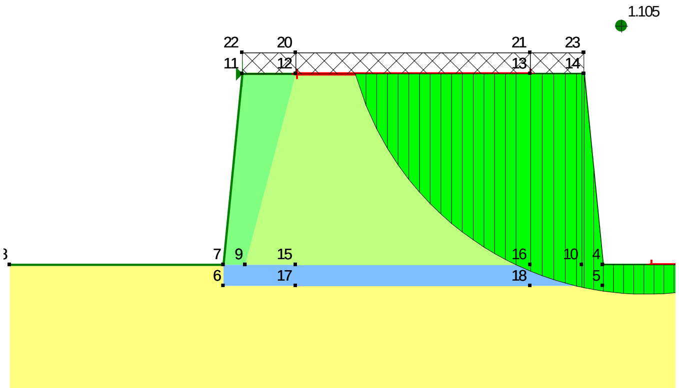

Fig. 20. Critical slip surface (k_a=1,105).

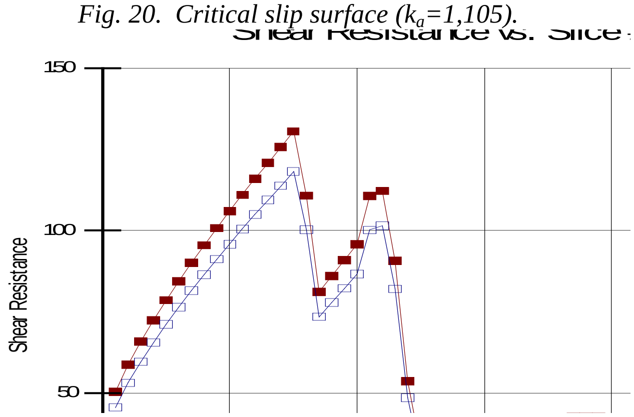

Fig. 21. Shear resistance versus slice.

The conclusion of this calculus is that the maximum total displacement is 33,5mm (at the edge the sidewalks - Fig. 12). As it can be seen in Fig. 15 the plastic points appear at the edges of the embankments and in the central part of soil foundation at an approx. depth of 10 m. Also, a very important notice is that the plastic points also appear at the edge of the embankments at around 5m around the body of backfill. From Stability analysis we can see that the structure is almost permanent at limit having a factor of safety 1,105. Fig. 17 show us that the embankment structure has a very small reserve in strength for seismic action.

c) MODEL 3. Seismic response due to earthquake with foundation soil in flooded state.

For this model GEOSLOPE is used for analysis. Here a dynamic analysis is performed according to the Romanian seismic code P100-2006. A scaled accelerogram was used with peak ground acceleration of 0,2g and 15s. Time increment was 0,02s and results were saved at every 10 steps.

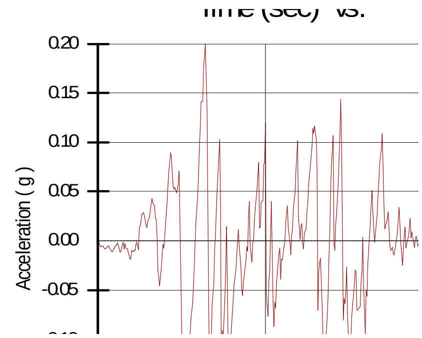

Fig. 22. Acceleration - input.

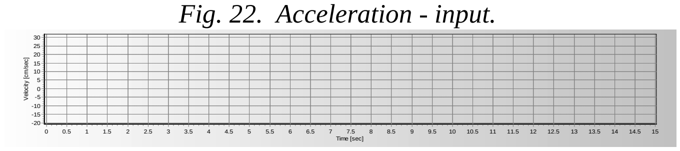
Fig. 23. Velocity.

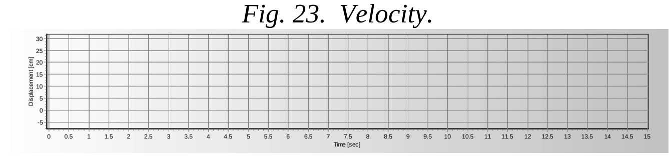
Fig. 24. Displacement.

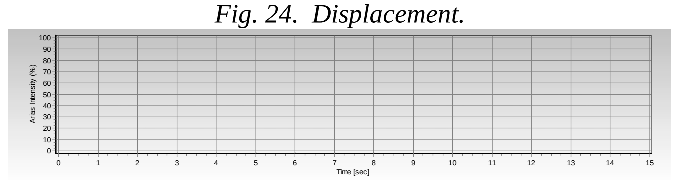
Fig. 25. Arias Intensity.

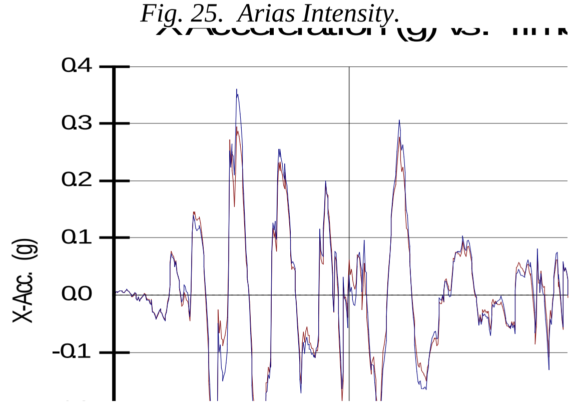

Fig. 26. Time history (x-acceleration) - output.

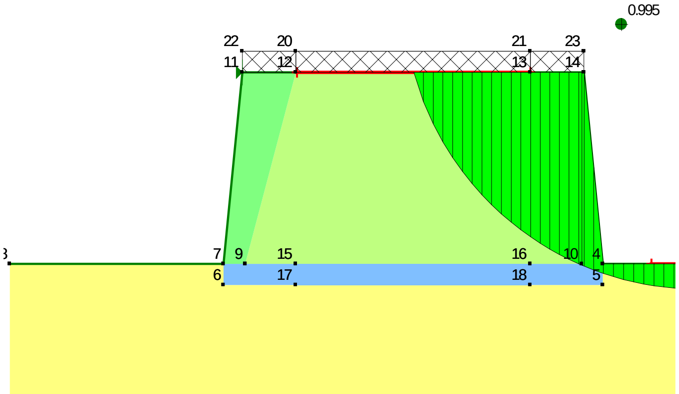

Fig. 27. Critical slip surface (k_a=0,995).

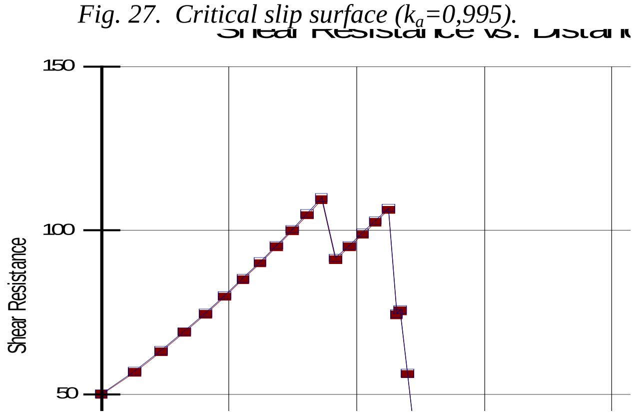

Fig. 28. Shear resistance versus slice.

The conclusion of this calculus is that the maximum horizontal acceleration at the top of the embankment is 0,35g. From stability analysis we can see that the structure is unstable having a factor of safety 0,995. Fig. 26 show that the embankment structure has no reserve in strength.

5. CONCLUSIONS

a) Model 1: the maximum total displacement is 280mm (at the edge the sidewalks - Fig. 9). As it can be seen in Fig. 10 the plastic points appear at the edges of the embankments and in the central part of soil foundation at a approx. depth of 5m. Also, a very important notice is that the plastic points also appear at the edge of the embankments at maximum 1m around the body of backfill.

b) Model 2: maximum total displacement is 33,5mm (at the edge the sidewalks - Fig. 12). As it can be seen in Fig. 15 the plastic points appear at the edges of the embankments and in the central part of soil foundation at an approx. depth of 10 m. Also, a very important notice is that the plastic points also appear at the edge of the embankments at around 5m around the body of backfill. From Stability analysis we can see that the structure is almost permanent at limit having a factor of safety 1,105. Fig. 17 show us that the embankment structure has a very small reserve in strength for seismic action.

c) Model 3: the maximum horizontal acceleration at the top of the embankment is 0,35g. From stability analysis we can see that the structure is unstable, having a factor of safety 0,995. Fig. 26 show that the embankment structure has no reserve in strength.

d) All tests and calculations made underline high strain and low bearing capacity of flooded state soil foundation.

e) Soil foundation is high compressibility terrain with great sensibility at moistening under stresses according to specific macrostructure.

f) To realise this works uncohesive soils are recommended; all tests on Bahlui clay show that this material is not proper to be used safely for embankments.

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