The geotechnical evaluation for touristic development in coastal plain area, Shëngjin zone

Oltion Korini[1], Ylber Muceku[1]

[1]Faculty of Civil Engineering and Architecture, EPOKA University, Albania.

ABSTRACT

Geology and geotechnics look sometimes not interesting to the structural engineer, until a serious geotechnical hazard or problem is faced. This problem could be a soft soil layer under the foundation, a high underground water level or perhaps a slope stability situation. In these situations analysis has to be performed in order to estimate the hazard and give the most optimal solution.

This case study describes a solution for the urbanization of a problematic area according to geotechnical construction that is located south of Shëngjin town in north-east of Albania. The area taken into consideration presents high interest for possible investments in touristic resorts because it is located near the seaside. More than 60% of this region consists of soft peat soil with extremely low bearing capacity. On the current conditions (without any soil improvement), it is impossible to use as construction site because the structure would sink in this soft soil.

The solution adopted for this situation consists of soil compacting in medium loose soil and the use of geotextiles for the peat area. Geotextiles in this case are used to prevent the settlement of the basement in the very porous lithological environment such as the peat soils. The presence of a river in the south part is constant flood hazard in rainy days. A specific river protection must be performed for this reason. Also from field work, geotechnical investigations is found that the underground water table is close to surface area, 0.1-0.2m.

For places that will carry big loads such as roads and buildings other specific means have to be done. For this reason the use of geogrids is chosen. These plastic materials often reinforced with glass fibres are able to carry considerable tensile load and are used successfully in ground stabilization cases worldwide.

1. INTRODUCTION

The studied area is formed by different geological layers. It is built mainly of alluvial deposits, peats; sands and loams soils are generally soft geotechnical soil layers. On the other hand the Drini River passes in south, nearby of studied area and floods are possible.

The objective of this study is to implement methods that would improve the geotechnical characteristics of the area. The final purpose is to urbanize the area with maximum 3 story structures that can be used for touristic residences. The urbanization also means the construction of roads, parks and all other touristic components. The actual site status is in very unfavourable conditions for construction. Almost half the area is inside the sea and with peat soil. Also the Drin river that forms delta floods most of the area in periods of heavy rains. So this study has two main directions:

a) Water hazard protection and drainage of the area. All the underwater area has to be filled with material in order to make an uniform flat area that reaches until the seaside. Actually the place looks like a lagoon or marsh. After filling the area, some protection from potential river flood is required.

b) Soil improvement methods. Judging from the very weak geotechnical conditions is not possible to construct any structure without significant effort. So there must be done important improvements in soil. The most reasonable means is probably to make a flexible but enough strong layer that can support loads up to 2.5 daN/cm^2. Using deep foundations would be expensive and therefore difficult to construct. Geotextiles and geogrids are surely a good idea for achieving the above conditions. Geotextile has to be used in order to prevent material mixing from the above strong layer to the below very soft and very wet layers. Geogrids on the other hand can provide the required flexural stiffness and axial as well in order to support the predicted structures. Clay and graded fill can be used to form barriers for flood protection from the river at the south part of the area.

2. GEOTECHNICAL CHARACTERISTICS

The area into consideration is a part of Adriatic coast at the North-West of Albania.

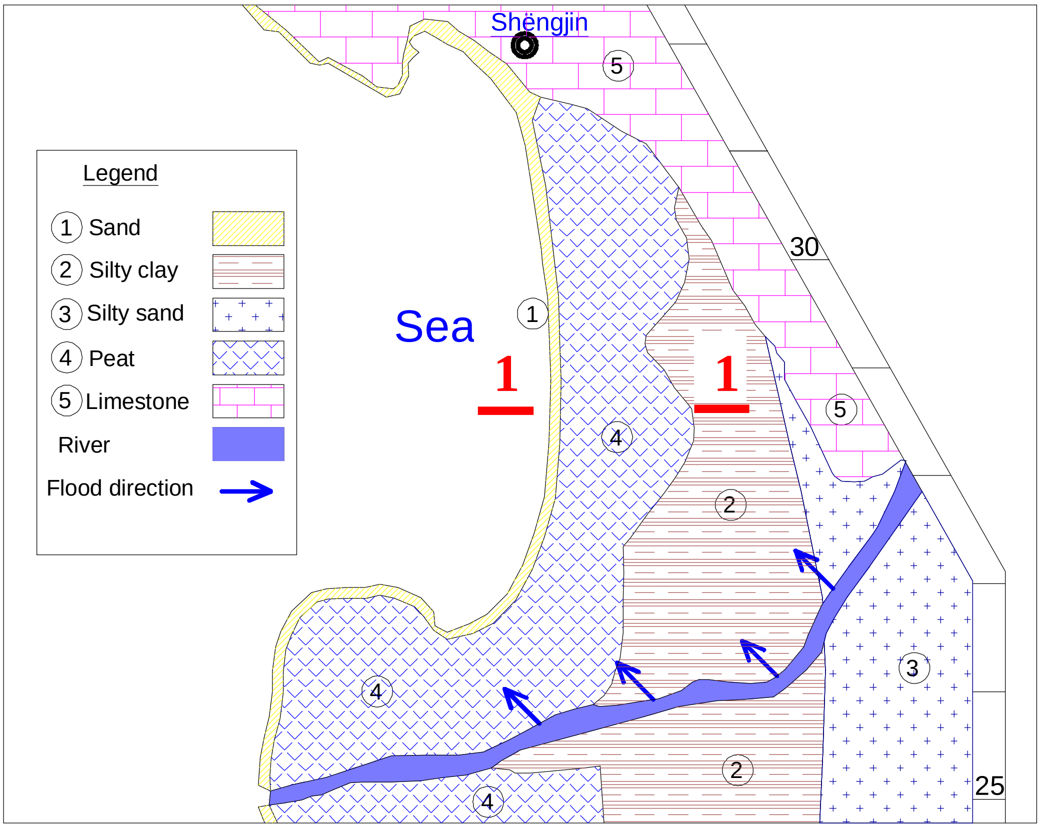

Figure 1. Geotechnical map of area Shëngjin-Drin river delta. Scale 1:50000

Here, many geotechnical boreholes are executed, from which 31 undisturbed and 43 disturbed soils samples are taken and analysed in laboratory for the physical-mechanical properties. The Shëngjin town is nearby the area that we are studying. Based on these physical-mechanical properties we determine 4 geotechnical units, which we are treating as followings:

Layer 1.

Is represented by sands soils, which contain quartz with mica and crushed shells, gray colours. This layer is saturated and is in medium condition. Represents the sands of the beach during the coastline as a strip with a width from 100 to 150 m and thickness 2.5-3.5m. For this layer we have the following mean values of physical - mechanical parameters:

γo=2.65 gr/cm^3, γ=1.88-1.90 gr/cm^3, n=41-44% E_{1-3} =60÷100 kg/cm^2, φ=25-28^0, Bearing capacity σ=1.2-1.5 kg/cm^2

Layer 2.

Silty clay-loams with alluvial origin with beige, gray and beige to brown colors, with organic matter content, pieces of trees, plant roots, etc. Layer is saturated with water, slightly compressed until average compressibility, with average plasticity. Layer has a thickness of 1.2-2.8m. For this layer we have the following mean values of main physical - mechanical parameters:

γo=2.45-2.55 gr/cm^3, γ= 1.80-1.85 gr/cm^3 n=42-47 %E_{1-3}=30÷75k g/cm^2 φ=16-22^0 σ=1.2-1.5 kg/cm^2 CBR=1.0-2.0%

Layer 3.

This layer lies under the layer no. 2 and is 2.5-5.0m thick. It is constructed of fine silty sand, originating sea – lagoon with gray to blue, to dark blue colours. It contents the pieces of marine shells, with rare and thin kelp fiber and are saturated, with compressibility to medium with poor plasticity because of the high content of silt fraction up to 40%. For this layer we have the following mean values of main physical - mechanical parameters:

γo=2.65-2.66 gr/cm^3, γ=1.90 gr/cm^3, n=45-46%, E_{1-3} =70÷100 kg/cm^2, φ=22-24^0, σ=1.4-1.5 kg/cm^2, c=0.05 – 0.1 kg/cm^2, CBR=2.0-3.0%

Layer 4.

Represented by the peat soils. They form very weak subbases in geotechnical aspect. Here are some characteristic parameters of this soil type.

n=92,30 %, k=4 x 10^{-6} m/s, σ<0.5kg/cm^2, CBR=0.2%

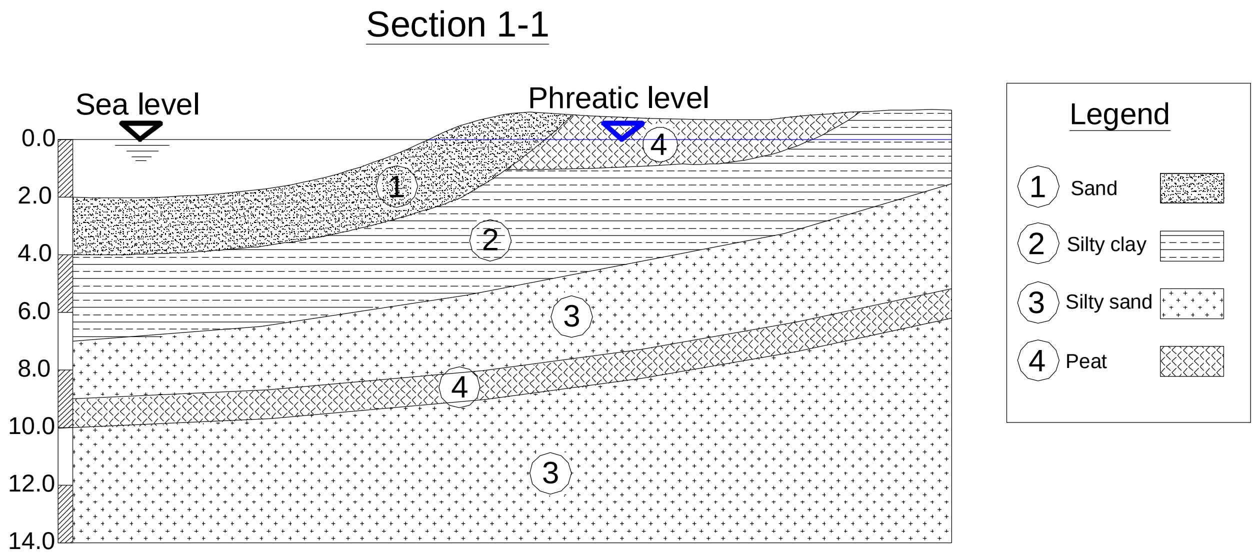

Figure 2. Geological section 1-1

3. WATER HAZARD PROTECTION METHODS

The Drin river flows at the south part of the area has a very shallow riverbed and therefore is a constant threat for flooding in the area. For this reason specific protection is required. The normal solution is to construct a dam embankment that would have two effects:

1. It will prevent the river to flood the area. For this reason a height must be specified for the embankment that would cover the maximum flow of the river.

2. Considering the flat area and very high water levels it is necessary to protect the region also from underneath infiltrations of the river. In order to achieve that a certain depth must be considered for preventing this occurrence.

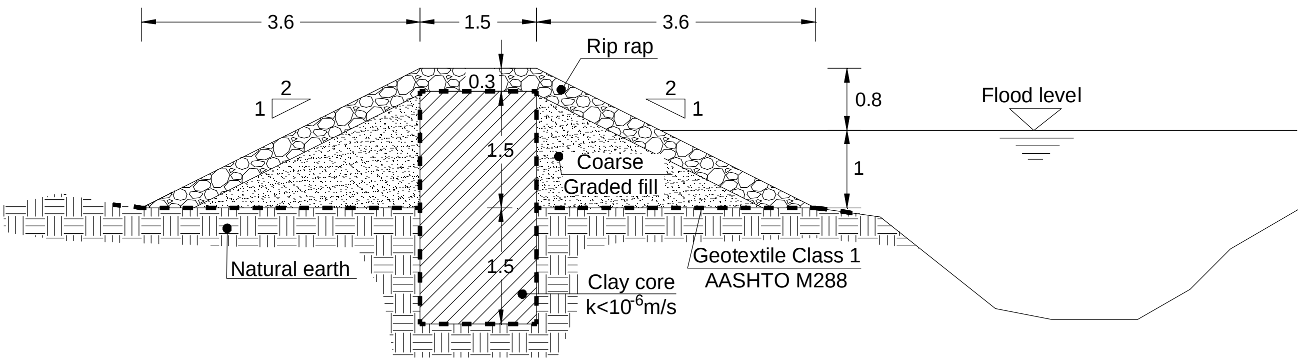

Figure 3. Small dam for river flood protection

The area in consideration is currently partly under water or the same with sea level. These conditions require the following means: The level of the whole area must be raised for at least 1.5m above the sea level. This has to be done to protect the area from varying sea levels. Also the variable and very close to surface underground level of water requires a permanent underground drainage system. This can be done with 1m deep channels filled with granular material with diameter min 1cm. These channels will be wrapped with geotextile for preventing material mixing. The drainage capacity is calculated as below.

If the gravel of the drain has 30% voids and these voids are 70% effective most of the time the capacity to carry water is:

$$S = 0.3 \cdot 1m \cdot 1m \cdot 0.7 = 0.21 m^2 / ml \qquad (1)$$

Considering the area is almost flat the drains may have max 0.5% slope. So the water removal capacity is:

$$Q = v \cdot S = a \cdot t \cdot S = 0.005 \cdot g \cdot 1\sec \cdot 0.21 = 0.0103 m^3 / s \qquad (2)$$

The drains have to remove rain water at a normal time period, e.g. 20minutes. If we consider the maximum rainfall for 20minutes at 100 year return period: I=0.065 m^3/m^2, taken from “Rainfalls in Albania” 1975. So the distance between 2 drains is:

$$d = \frac{Q \cdot t}{I} = \frac{0.0103 \cdot 1200}{0.065} = 190m \qquad (3)$$

The distance between 2 drains is finally is accepted 200m as a more practical value. The drains start 50-100m beyond peat area and finish 50m before seaside. Along the river dam is put a drain all the way until the seaside.

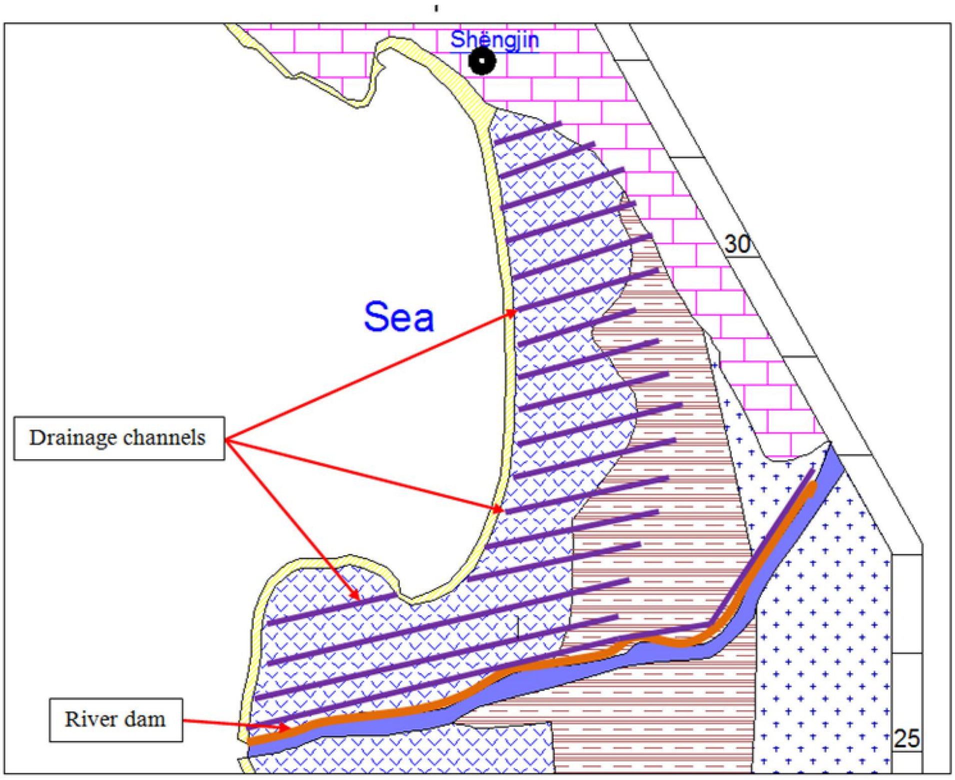

Figure 4. Water hazard protection plan view

4. SOIL IMPROVEMENT METHODS

General Area - Places where no asphalt roads or buildings are planned to be constructed are classified as general areas. For these areas are predicted lighter reinforcing methods. For a general reinforcing are foreseen the following:

In actual dry areas, firstly is removed the topsoil and then compaction is performed. In actual wet or peat areas geotextile is spread. After this in all the area are constructed drainage strips 1m high of crushed stone wrapped with geotextile. After the drainage is done is spread 0.5m thick layer of clay between them. The clay layer is with variable thickness leading to 10cm near the drainage. Above the clay, granular fill with diameter above 1mm is put until the top of drainage level. 0.5m fill with random soil (except peat) for the whole area.

See figure 5 for layers layout. In dry places like east side, geotextile is not put. In these areas only compacting is performed after removing topsoil 0.3m.

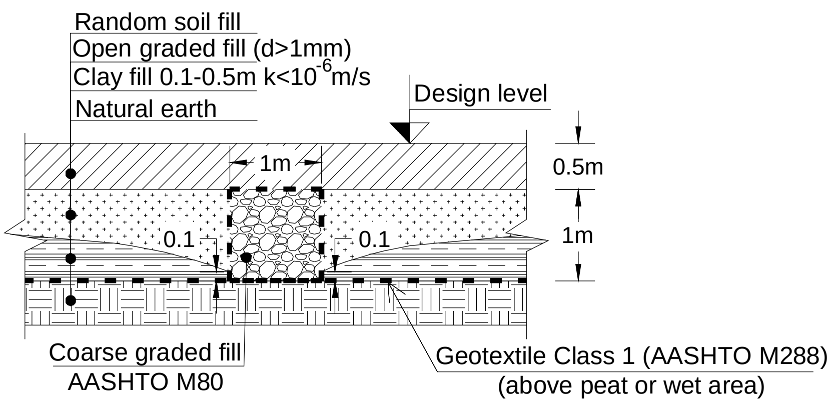

Figure 5. Water drainage and soil layers detail.

Roads - The subbase of the road is planned to have additional protective layers and reinforcement so that it will deform as uniformly as possible under vehicle loads. The use of geogrids is very beneficial is this case because it can assure very good stability with relatively thin layer of fill above it. The following layers are recommended for road after general treatment: 1. Geogrid extended 20% outside the road pavement, 2. Minimum 30cm granular fill (capping layer) 3. Asphalt layers.

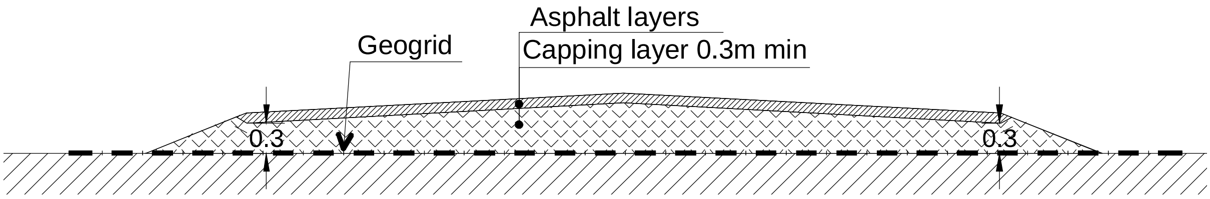

Figure 6. Road layers typical cross section

Buildings - Special attention must be paid to the foundations of the buildings. They require a medium value of 2.0 daN/cm2 bearing capacity but also good water removal and soil stability. For making safe structures the following measures have to be taken:
1. Structures are not recommended to have underground facilities because of the high underground water level.
2. A geogrid layer has to be put above the drainage level with minimum tensile strength 5kN/m (0.5m random soil to be removed again). This layer must extend beyond the foundation plan limits with at least 30% more in both directions.
3. Above the geogrid only coarse graded fill (AASHTO M80), or crushed stone with particle diameter above 10mm has to be used. This will protect the foundation by not allowing water to go up, and will spread uniformly the stresses of the structure. This coarse graded fill under the foundation must be at least 1m thick and extend 30% as the geogrid.
4. If structures higher than 3 stories are predicted, additional improvement has to be done to the foundation layers.

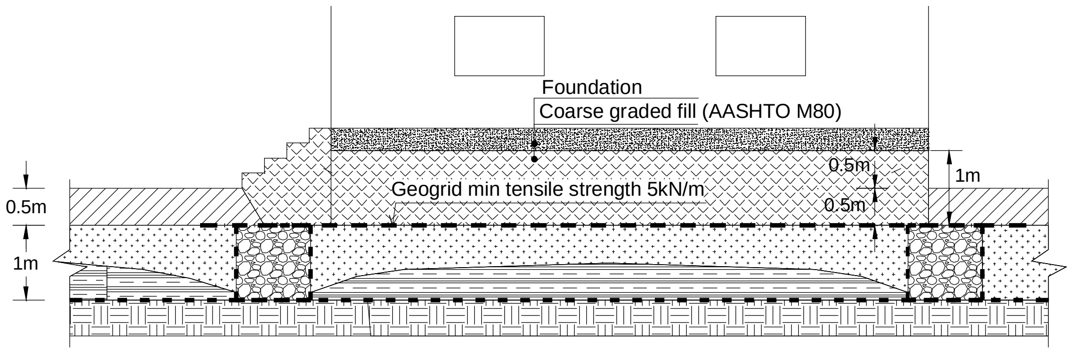

Figure 7. Under structures layers typical cross section

5. CALCULATION OF SOIL DEFORMATION WITH FINITE ELEMENTS

This calculation is performed with finite element software PLAXIS v8.5. This software is using professional and up to date soil models that cover a wide range of geotechnical situations. The following layers (starting from surface) are used in the calculation:

a) 1.5m gravel fill + geotextile below, **b)** 2.5m peat, **c)** 3m silty clay, **d)** 3m silty sand, **e)** 1m peat, **f)** >10m Silty sand.

For all layers except peat is chosen the Mohr-Coulomb model (elastic-perfectly plastic). But for peat is chosen a more specific model, Soft Soil Creep model because it is expected that this layer to exhibit the biggest deformation. The parameters for this type of calculation are taken from Huat et al 2011, who has performed tests with several peat samples. The values are shown in Table 1.

Table 1. Parameters of Soft Soil Creep model for peat (Huat et al 2011)

Parameter	**Value**
Material model	Soft soil creep
Type of behavior	Drained
Soil unit weight (γ)	11.0 kN/m^3
Poisson's ratio (υ)	0.35
Cohesion (c)	1.0 kN/m^2
Friction angle (φ)	20°
Dilatancy angle (ψ)	0°
Modified swelling index (κ*)	0.022
Modified compression index (λ*)	0.12
Modified creep index (μ*)	0.006

Roads – Model is like figure 6 + layers above. Geotextile is chosen Class 1 according to AASHTO M288 specifications. Traffic loads are taken from Eurocode 1 (1991-2). The chosen load in this case is the most unfavourable which is Load Model 1 with axle load 300 kN. Each tire will have 150 kN and contact width is 0.5m. The most unfavourable case is when two heavy axles are put together considering the road has two lanes. An asphalt layer with 10cm thickness is put on the surface with the following properties: E=3000 MPa, φ=35, c=120 kPa, according to Fwa et al 2004.

The vertical displacements are calculated in two phases: **a)** 1 month construction time of road. **b)** Consolidation for period of 10 years.

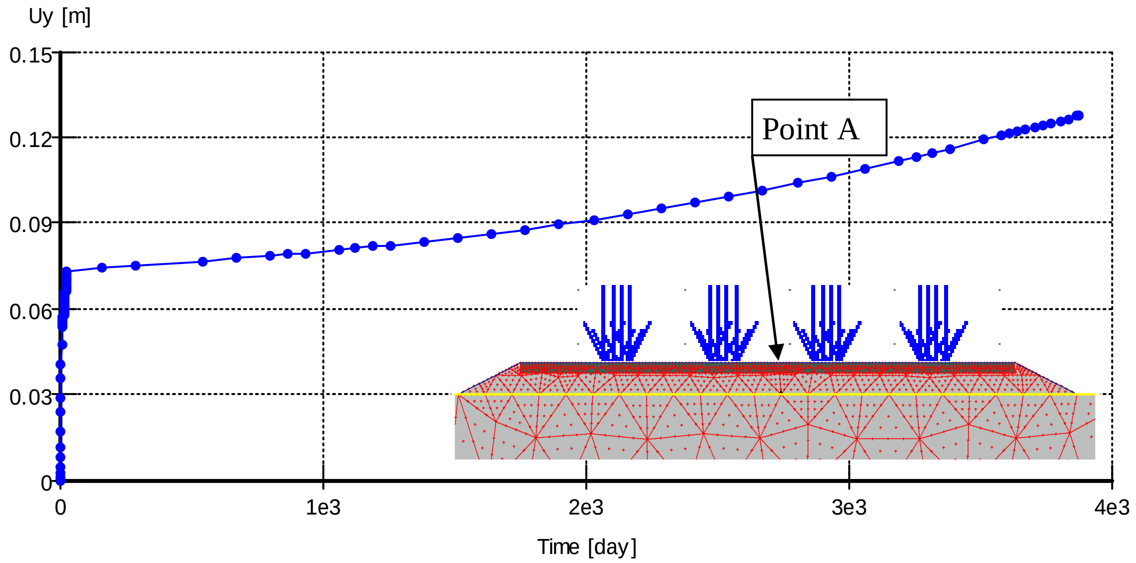

Figure 8. Vertical displacements variation in time

The displacement-time graph in figure 8 is plotted for point A as shown above. The displacements are in quite acceptable range. During construction is expected to happen 7cm settlement which is half of total value 14cm. The proposed road layers with total thickness 0.5m are satisfactory from the above calculations. Geogrid is very important in the role of soil stabilization under heavy loads. It is of course possible to use thicker layers for road design reasons.

Buildings - For this calculation, is modelled the situation described in figure 7. The effect of the structure is represented by a static distributed load calculated as follows: Dead Load (slabs+beams) =6.0 kN/m2, Live Load(walls+furniture+people)=4.5 kN/m2 Safety factor sf=1.3, Floors n=3

Total load is: (DL+LL)*sf*n= (6+4.5)*1.3*3 = 40.95 kN/m2.

The vertical displacements are calculated in two phases: 2 month construction time of 3 story building. Consolidation for period of 100 years.

In the proposed sketch fig 4.2 there is 0.5m of extra fill below the foundation level so the structure is safe. But still this kind of vertical displacement is considerable and needs further attention. Considering a construction time of 2 months it is possible to check when do the deformations happen and judge for a solution.

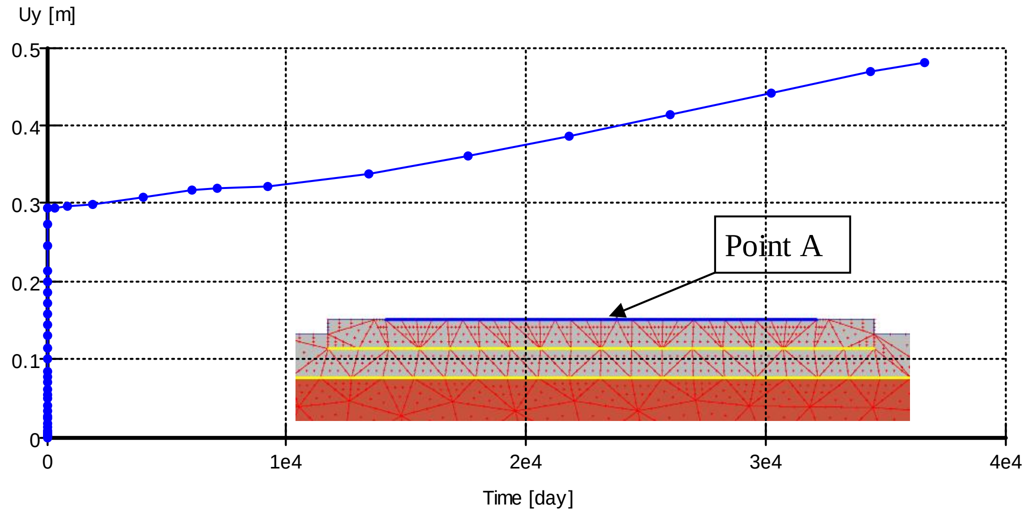

Figure 9. Vertical displacements variation in time

The displacement-time graph in figure 9 is plotted for point A as shown above. The displacements are in quite acceptable range. During construction is expected to happen 29cm settlement which is more than half of total value 48cm. In this case the most optimal solution could be to increase the fill layer by the first stage deformation. Until the final period of time (100 years) there remains only 19 cm deformation to happen.

Normally the final height of the fill has to be defined by the architectural levels of the structure by considering the displacement data above. One other factor is the importance of the structure that could lead to a higher reinforced earth basement or a deep foundation.

6. REFERENCES

[1] AASHTO, Standard specifications for transportation materials and methods of sampling and testing. 2010.

[2] A study of the compressibility behavior of peat stabilized by DMM: Lab Model and FE analysis. Bujang B. K. Huat. 2010.

[3] Eurocode 1(EN 1991-2) Actions on structures - Part 2: Traffic loads on bridges.

[4] Fwa T.F, Tan, S.A. Reexamining c-φ concept for Asphalt Paving Mix Design. ASCE 2004.

[5] “Rainfalls in Albania” published by ex-Hydrometeorologic Institute, 1975.