

# Landslide FE Stability Analysis

L. Kellezi

*Dept. of Geotechnical Engineering, GEO-Danish Geotechnical Institute, Denmark*

S. Allkja

*Altea & Geostudio 2000, Albania*

P. B. Hansen

*Dept. of Geotechnical Engineering, GEO-Danish Geotechnical Institute, Denmark*

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ABSTRACT: FE stability analysis of a landslide is carried out. Site investigation (SI) and laboratory testing are performed defining the soil conditions, which consist of firm to stiff silty clays, mudstones and sandstones. The water table (WT) is considered to vary from the surface of the mudstone/sandstone bottom layer, to the recently measured level. The sliding planes are defined and implemented in the analysis. For the considered WT conditions, residual friction angles  $\varphi_R = 12^\circ$  and  $\varphi_R = 15^\circ$  are respectively evaluated to be the minimum required for equilibrium. The measured values vary from  $\varphi_R = (10 - 14)^\circ$ . On these conditions engineering measures such as 4 rows of cast-in-place piles of diameter  $D=1.0$  m and spacing  $(2-3)*D$  in combination with the drainage system, are analysed as a measure to stabilize the landslide.

## 1 Introduction

"A landslide devil seems to laugh at human incompetence". This is a saying from (Bjerrum 1967). A lot of research has been carried out throughout the years towards landslide stability problems. The first sign of an imminent landslide is the appearance of surface cracks in the upper part of the slope, perpendicular to the direction of the movement. These cracks may gradually fill with water, which weakens the soil further and increases the horizontal force that initiates the slide.

A landslide is primarily the result of a shear failure along the boundary of the moving soil or rock mass. Landslide calculations indicate that the maximum shear stress occurs at or close to the toe of a slope, the shear strength of the soil is first exceeded at this point and the failure then spreads up the slope, (Broms & Wong 1991). If a natural slope fails, it is much more probable that the failure has been caused by a gradual decrease of the shear strength, than by extreme conditions at the time of failure. The addition of water to a slope increases the load owing to the added weight of the water. The shear strength is reduced owing to the increase of the pore water pressures. Water in fact has been implicated as the main controlling factor in most slides.

Landslides can be classified according to their state of activity into: active, dormant, and stabilised. As part of a road project in Albania, stability analysis of a sliding zone, which has been active for years is investigated. Some engineering measures have previously been taken, which have not solved the problem. Geotechnical investigation and laboratory testing are recently carried out in the area. The derived soil parameters are applied to the stability calculations. Regarding the methods of

landslide stability analysis they can be divided into deformation and ultimate state. Considering the geometry of the sliding zone, the heterogeneity of the soil layers and extend of the sliding planes, FE modelling is chosen as the most appropriate method for the current landslide stability problem. The analysis is carried out with PLAXIS FE program.

## 2 Location of the landslide

The landslide is located on the left side of the Vjosa River in Albania. The inclination of the valley is different at different levels and is conditioned from the geological formation. The parts composed of sandstones are steeper than the parts composed of mudstones. Different parts of the zone are shown in Figure 1. The incline is afforested with small plants mainly bushes. Some almost horizontal parts exist created by Vjosa River representing the old terraces. The inclination is composed of mudstone and sandstone covered by 5-10 m colluviums deposits.



(a)



(b)

Figure 1. (a) The central part of the sliding zones (b) The downfall part seen from the upper part.

## 3 Geotechnical conditions and previous engineering measures

Geological survey, geotechnical SI and laboratory testing are carried out in the area. Seven soil layers are distinguished and given in Table 1. The formations consist of quaternary and rock deposits. Boreholes to depths 15-20 m are performed and undisturbed and disturbed samples are taken and used for laboratory testing. Shear box tests are carried out for evaluating the soil strength parameters. For Layer 7 axial compressive strength test is performed defining the undrained shear strength. Layer 1 is the fill material of the road embankment. Layer 2 is the active top layer of the sliding section. The other layers follow with depth.

The active sliding planes are identified from the boreholes, and the residual parameters are determined from the laboratory testing. It is investigated at the site that the movements of Layers 2, 3 and 4 are more active and visible. The movements of Layer 6 and 7 are slower and more in equilibrium. This is confirmed by the measurements carried out in the area.

The reasons of the slope stability problems are the colluviums deposits composed of silty clays with gravel. When fully water saturated they loose their strength and start to move in the downfall direction, taking with the weathered beds of mudstone and sandstone. The erosion phenomenon from rainfall and the Vjosa River at the sliding foot, are also important.

Plastic pipes are installed in order to estimate the dynamics of the soil movement and to monitor the

underground water level. The fluctuation of the WT in spring is  $-1.5$  m while in September (the driest season) is  $-2.5$  m. The engineering measures undertaken, consisting of two retaining walls, and lowering the WT, which goes down to the depth of the gravel layers in the road area, have intended to reduce the weight of the moving body. After the measures, the sliding has a temporary stability. Its upper masses at the road area move very slowly. However, the deepest parts of the sliding are still active, but they move with a low speed and it is hard to be noticed at the surface. At the upper part 10-15 cm thick fractures are found.

Table 1. Soil layers and parameters at the sliding zone/landslide.

L	Soil Type	Average thickness (m)	W <sub>L</sub> [%]	W <sub>P</sub> [%]	I <sub>p</sub>	W [%]	G <sub>s</sub> [T/m <sup>3</sup> ]	γ [T/m <sup>3</sup> ]	e	φ [°]	C [kPa]	φ <sub>R</sub> [°]
1	GRAVEL with sand medium dense	7.5						1.9		42	3.0	
2	CLAY, firm, silty	5.0	41.5	23.2	18.3	24.6	2.7	1.9	0.78	18	25	10-11
3	MUDSTONE, Weak, very weathered	3.5	42.7	23.5	19.2	23.7	2.72	1.98	0.76	20	35	11-12
4	CLAY, firm to stiff	4.0	39.8	23.4	16.4	24.8	2.7	1.9	0.78	19	25	10.5-11
5	CLAY, firm to stiff	2.0	38.8	22.3	16.5	24.7	2.71	1.92	0.79	18	30	10-11.5
6	SANDSTONE, weak, very weathered rock	1.6	28.6	23.4	5.2	22.4	2.7	2.18	0.7	26	15	
7	MUDSTONE and SANDSTONE Weak	Large depth	43.6	22.1	21.5	14.6	2.73	2.34	0.48	26	165	13-14

#### 4 FE modelling of the landslide

The analysis is carried out with PLAXIS FE program, (PLAXIS, 2002) It is important for the stability of the slope to model the section to the lowest quotes reached at the river flow. Two-dimensional (2D), plane strain FE modelling is carried out as shown in Fig. 2 where the dimensions are in meters giving the real geometry of the considered most critical section.

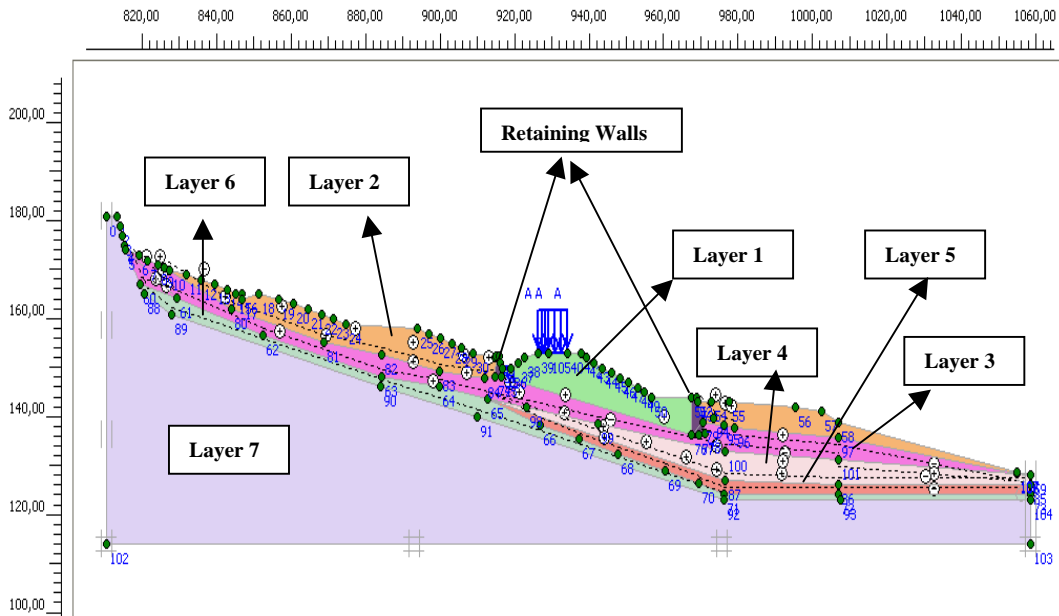
Fixed boundary conditions are applied at the bottom of the FE model considering no deformations to occur at that depth of Layer 7. On the left side the model is terminated at the start of the Layer 7 and on the right side at the lowest quote assuming zero horizontal deformations to occur. This might be rather optimistic considering the erosion effect from the river flow.

The traffic load is modelled as uniformly distributed vertical load according to (DS410E, 1998) as shown in Fig. 2. The dynamic effects from the traffic or seismic activity are not taken into account. in the current analysis.

The soil layers and the soil parameters are employed as referring to the data given in Table 1. The deformation parameters, such as deformation modules are determined on the basis of general experience. The Mohr Coulomb elasto-plastic constitutive soil model is applied for the gravel/sand in drained conditions. According to the site investigation beds of sandstones have high permeability. In addition, the friction angle derived from the shear box test for Layer 6 is higher. Based on those data Layer 6 is also modelled in drained conditions. The rest of the soil layers, which consist mostly of clay and mudstones are modelled in undrained conditions as more critical for the stability of the zone. Layer 7 is modelled based on the data derived from the axial compressive strength.

The WT is investigated in two scenarios; Low level corresponding to the surface of the Layer 7, and high level corresponding to the level recently measured.

The 6-node triangular plane strain FE elements are applied. The model is composed of 7171 elements, 16778 nodes and the stresses are calculated at 21513 stress points. Average FE size is 1.5 m.



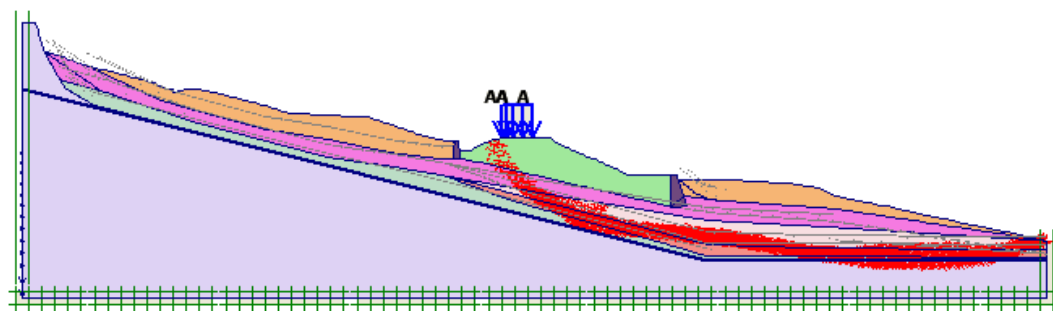
## 4.2 Results of the FE stability analysis

Results from the FE stability analyses are given in Fig. 3. Two scenarios are considered regarding soil strength parameters: Design parameters with partial coefficient  $\gamma_m=1.0$  (characteristic parameters) and design parameters with partial coefficients to friction angle and cohesion based on the (DS415,2002).

Several calculations are carried out to investigate the effect of the layered soil profile, variation of the soil strength parameters, modelling in drained or undrained conditions and especially the implementation of the sliding planes, which are decisive for the stability of the slope.

Firstly, the FE calculations of the model deactivating the sliding planes, are carried out. The results of the calculations consisting of the failure mechanisms are given in Fig. 3 (Incremental Strains) for low and high WT.

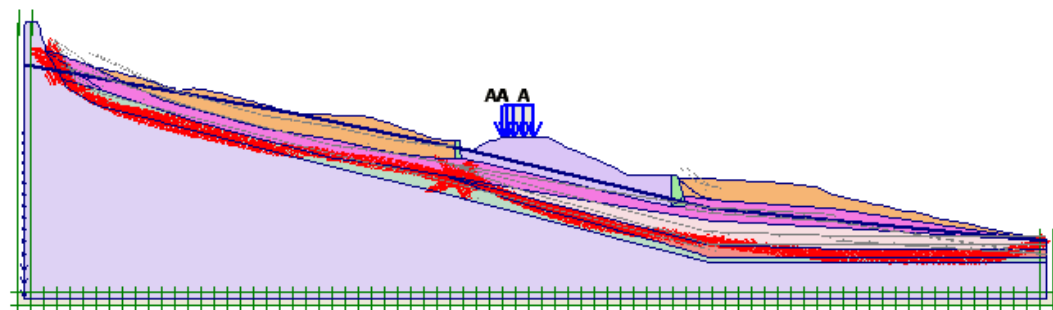
a)



**Incremental strains**

b)

Extreme principal strain increment  $-29,07 \cdot 10^{-3} \%$



**Incremental strains**

Extreme principal strain increment  $-7,62 \cdot 10^{-3} \%$

Figure 3. Failure mechanisms for deactivated sliding planes. (a) Low WT (b) High WT.

In the calculations the initial stress conditions are calculated using the gravity load procedure due to the inclined layering of the model. The WT is initially considered at the low level and the water pressure is generated.

Staged construction is applied activating different soil layers, building the road embankment, applying the traffic load, increasing the WT to high level and activating sliding planes. Deformation analysis in a staged construction gives the amount of deformations generated in the model. For the deformation parameters used for the soil layers, the horizontal deformations is of few centimetres when sliding planes are not considered.

After the plastic calculation, safety or stability analysis is carried out employing the  $\varphi - c$  reduction method to calculate a global safety factor  $F_s$ . In this approach the cohesions and the tangents of the

friction angles are reduced with the same ratio.  $F_s$  is defined as the ratio of the shear strength maximum available, to the shear strength needed for equilibrium, or the ratio of the true strength, to the computed minimum required for equilibrium.

Ignoring the sliding planes  $F_s$  varies from 1,77 to 2,15 and 1,39 to 1,73 respectively for low and high WT and characteristic and design soil parameters. Regarding the failure mechanisms, rotational slip is found for low WT and successive slips for high WT, cf. Fig. 3.

The results of the stability calculations considering active sliding planes are given in Fig. 4, (incremental displacements). Only the results from the models with interface soil properties modelled with the friction angle equal to the minimum required residual value and zero cohesion, are presented.

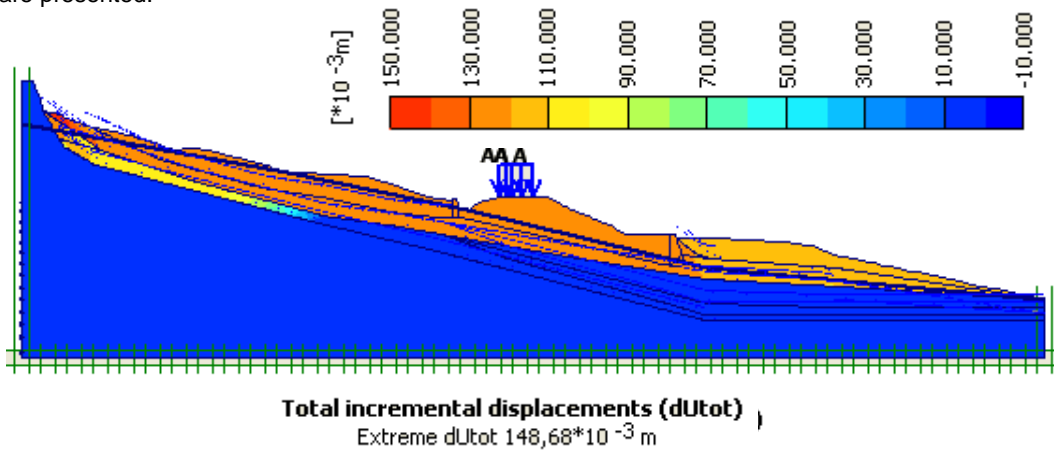


Figure 4. Failure mechanism, active sliding planes,  $\phi_R = 15^\circ$ , high WT.

Sensitivity analyses are carried out assuming a higher interface friction angle than measured and shown in Table 1 and reducing it to the minimum value required for equilibrium. For low WT residual  $\phi_R = 12^\circ$  and for high WT  $\phi_R = 15^\circ$ , characteristic values, are evaluated to be the minimum required for equilibrium. In Fig. 4 the failure mechanism is given for high WT and residual  $\phi_R = 15^\circ$ . Horizontal deformations of (45 - 50) cm are observed at the head of the sliding zone in this case, as shown in Fig. 5.

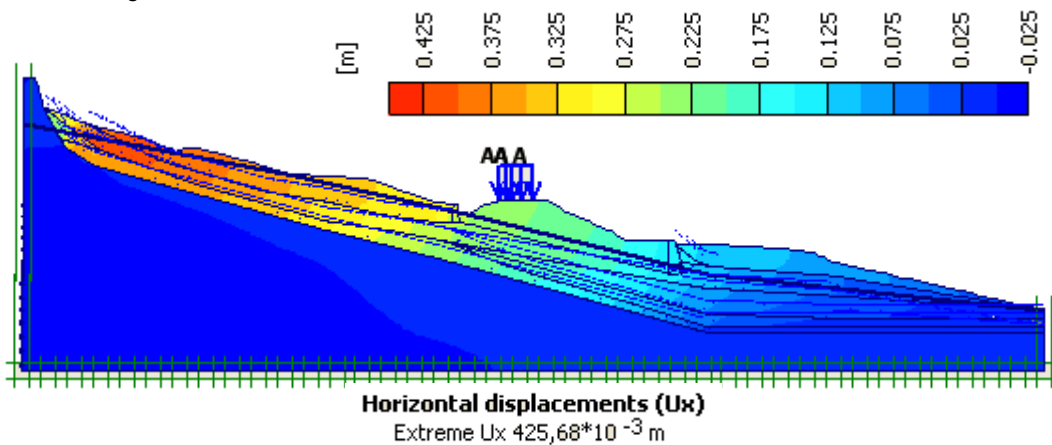


Figure 5. Horizontal deformations, active sliding planes  $\phi_R = 15^\circ$ , high WT.

From the FE analyses it is evaluated that for the sliding zone to be in equilibrium the residual friction angle should be minimum  $\varphi_R = 12^\circ$  for low WT and minimum  $\varphi_R = 15^\circ$  for high WT, both characteristic values. For WT at the terrain an even larger value of the minimum residual friction angle is required. The measured residual friction angles vary from  $\varphi_R = (10 - 14)^\circ$ . This means that depending on the WT variation during the year, the sliding zone changes from stable to unstable. When unstable, excessive deformations may occur. This can explain the deformations observed at the road in this area. It can be concluded that the sliding section along the considered road in Albania is unstable.

## 5 Design of the new engineering measures

To stabilise the sliding zone additional engineering measures are necessary. Lowering the WT by means of drainage system and building river barrages at the sliding toe to protect the slope from the erosion of the river is very important. Interfering in the existing measures consisting of two retaining walls, constructing them differently is probably an alternative. An investigation (FE analysis) has shown that the stability of the sliding zone is almost the same with or without the presence of the concrete retaining walls.

Other methods, (geometrical, hydrological, chemical, mechanical) can be used to correct the landslide. Mechanical method, in which the shear strength of the sliding mass is increased, is currently chosen. Stabilising cast-in-place concrete piles, which have a number of advantages over driven piles, are particularly investigated. For these pile types problems associated with vibration and the remoulding of the soil are greatly reduced. The piles applied in such cases are typically (0.3 – 1.5) m in diameter and the spacing varies from 1D to 3D ((Broms & Wong 1991).

Four rows of concrete cast-in-place inclined piles of  $D=1.0$  m and (2-3)D spacing, long enough to reach Layer 7 and in-caster minimum 1.0 m at this layer, are applied as shown in Fig. 6 in combination with the drainage system, which maintains the WT at the measured level. Plate elements are employed to model the row of piles in the out-of-plane direction taking into account the pile dimensions, material and pile spacing. The piles behave elastically. The 3D effect will be investigated in a further investigation considering 3D FE modelling.

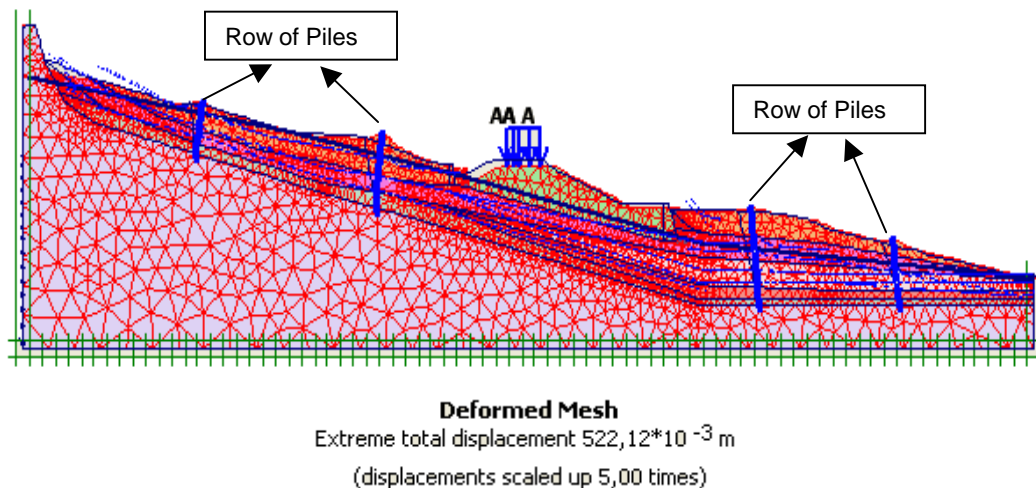


Figure 6. Deformed mesh. Landslide stabilized by 4 rows of cast-in-place piles.

The results of the landslide deformation and stability analysis including the stabilising cast-in-place

piles are shown in Fig. 6, deformed model, and Fig. 7 failure mechanism. The analysis is carried out with design parameters (including partial coefficients) for soil layers and the interface or sliding planes. The measured WT is maintained due to drainage system. Safety factor  $F_s > 1.0$  is calculated for the stabilized landslide.

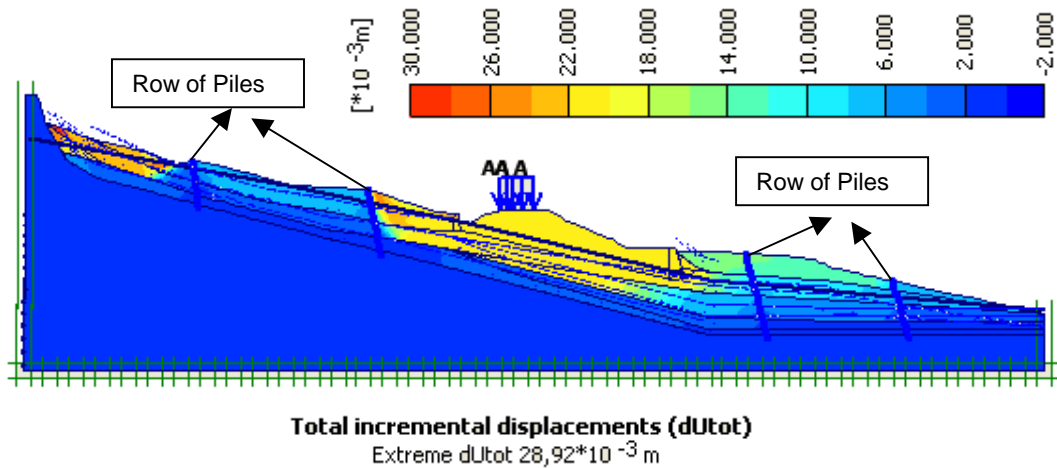


Figure 7. Failure mechanism. Landslide stabilized by 4 rows of cast-in-place piles.

## 6 Conclusions

Stability analysis of the sliding section/landslide along a road in Albania is carried out. The soil profile consists of firm to stiff silty clays, mudstone and sandstones. The active sliding planes are identified from the boreholes, and the residual parameters are determined for soils and rock masses faced in this zone. Stability of the landslide is evaluated based on the FE modelling. PLAXIS FE program is used and a 2D modeling is applied. The sliding planes are modeled employing interface elements. The material set for these elements (and the soil layers), considering the available parameters, consist of Mohr Coulomb type with friction equal to the residual friction angle. On the basis of FE stability analysis the sliding section is evaluated to be unstable. Engineering measures consisting of cast-in-place piles maintaining the measured WT, are analysed to stabilise the landslide.

## 7 References

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