Design of temporary deep foundation and monitoring for the erection of an arched bridge over an active landslide

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Abstract: The geotechnical and structural design of the infrastructure works required for the erection of an arched bridge is presented in this paper. The project was designed to overcome a major landslide, occurred in 2003 in Tsakona, Southern Peloponnese, Greece, which remains active until today. Although, a large part of these projects were founded in the sliding mass, they were designed to ensure the safe erection of the steel arch and minimum movements for its assembly and welding on the air. A real-time monitoring system with geotechnical instruments was decided to be installed, measuring and recording constantly the soil movements in the area, aimed to support the design and construction. The main assumptions of the deep temporary foundation design are underlined in this work, incorporating the monitoring results and the profiles of the ground movement. The results of the geotechnical and structural analyses, which led to the final design of the towers foundation, are also included.

Keywords: deep foundation of temporary works, monitoring of displacements, active landslide.

Introduction

In February 2003, a large landslide occurred in southern Peloponnese, Greece, with a dramatic cost on the economic and social life of the area. It was one of the largest landslide events in the country, leading to large soil mass movement of about 6.000.000m^3 and disrupting the National Highway Korinthos – Kalamata (Figure 1). In order to rehabilitate the Highway, it was decided to build an arched steel bridge over the sliding slope, named as the Tsakona bridge, having a total length of 390m (Figure 2). The arch was designed to bridge over the active landslide, which moved downslope at an average rate of about 1.5-2.0mm/month. The rate of movement increased significantly during the periods of intense rainfalls exceeding for short periods of time rates of about 30mm/month. Although the adversities, the great uncertainties and risks that a landslide phenomenon conceals, the project was successfully completed in March 2016.

Figure 1. The Tsakona landslide (2003)

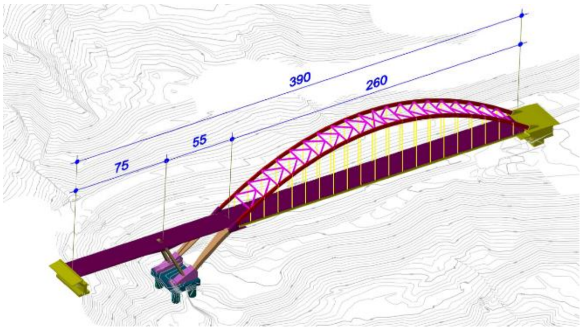


Figure 2. The Tsakona bridge

Fourteen (14) twin steel temporary towers, up to about 60m height, were required for the erection of the steel arch of the Tsakona bridge. The towers aimed at heavy lifting and final assembly at height of the steel arch parts. Their foundations consisted of pile groups, as shown in Figure 3. Thus, seven piers named as Π1Α-Π7Α were constructed for the left (east) branch of the Highway with pile lengths varying between 5m and 8m and another seven, denoted as Π1Δ-Π7Δ, for the right (west) branch with pile lengths varying from 8m for the pier Π7Δ to 32m for Π1Δ. The later included piles with the maximum free height of 8m. Piers Π1Δ to Π6Δ of the right branch and Π2Α to Π6Α of the left one were sited within the sliding mass. Among them, the central four pairs (Π2 – Π5) were the most crucial as they were totally founded in the central sliding mass of the active landslide where the most intense movements were observed. Between these piers, along the longitudinal axis of the bridge arches, slabs on the ground for the left branch and small temporary bridges on pile systems for the right branch were also built, which were used for the assembly and welding of the arch and deck segments.

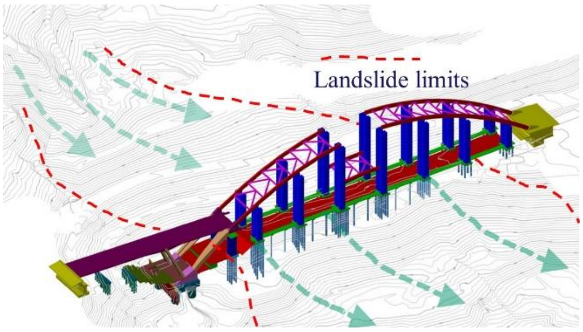


Figure 3. The towers for the erection of the Tsakona bridge

Two heavy duty towers were also placed beneath the transverse beam at the top of the curved strut of the V shaped bridge pier, named as MTA for the left branch and MTΔ for the right one, as shown in Figure 4. Moreover, the heavy highway traffic was deviated to pass on top of the slide body adding more risks during the construction. A pile wall was also designed to retain this temporary deviation of the highway. All these structural projects were combined with earth works, which were limited to the minimum possible in order to avoid additional loading to the sliding body of the landslide. It was clear form the geotechnical assessment that adding earth masses on the existing slope could lead to perturbation of the stability conditions with catastrophic results. A general view with all these works is illustrated in Figure 5.

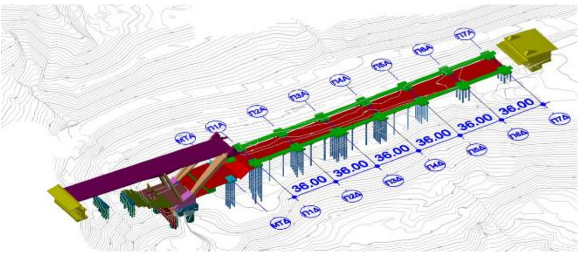


Figure 4. The foundation of the towers

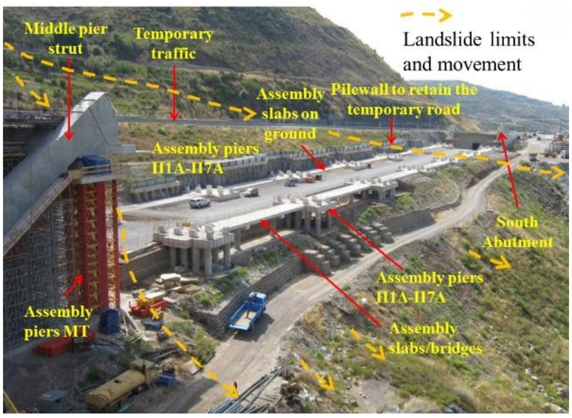


Figure 5. General view of the worksite

It was decided that the whole project was fully monitored with a network of geotechnical instrumentation, which consisted of optical targets, inclinometers and piezometers, initially installed to monitor the landslide during the period 2001-2011. In January 2013, an automated real-time system of inclinometers, piezometers, and rain gauge (pluviometer) was added, continuously recording, monitoring and updating a database. The system was linked simultaneously with a software program which automatically was providing the basic parameters of the slide movements. The evolving movements on the landslide surface were used (a) to assess the risk associated with the slide, (b) to facilitate the design of the deep foundation, (c) to inform the assembling engineers of any excess movements of the temporary piers and (d) to protect the temporary traffic from excessive movements.

Many two and three-dimensional finite element models were set up to calculate the behavior of the piers under static and seismic load combinations, taking into account the imposed movements caused by the active landslide. Numerical geotechnical analyses were conducted, in order to decide the parameters that would describe the soil-structure interaction due to the landslide movements, by means of earth pressure coefficient and equivalent spring constants for the piles. Those parameters would be further used to study the response of the piers founded in the sliding mass, and finalize their structural design.

In this work the monitoring system is initially presented, focusing on the system design, the process of continuous recording and the evaluation of the measurements, which, incorporating the monitoring results and the profiles of the ground movement, provided the main assumptions of the deep temporary foundation design. The results of the geotechnical and structural analyses, which led to the final design of the towers foundation, are also discussed.

During the final design of this project, the designers had to deal with several engineering issues, in the effort to optimize the cost since all the works were considered temporary. Most important issues were the foundation design, the depth of the piles with respect to the steep underlying bedrock, the estimation of the piles bearing capacity, bored in an extremely non-uniform soil material, comprising of the land slide debris, manmade fill and randomly dumped material from past excavations.

The landslide chronicle

At the area where the landslide took place, the old National Highway was opened in 2000. Soon after the inauguration, a sliding on the pavement was noticed and a geotechnical investigation was ordered in 2001, which concluded that the landslide pre-existed with a maximum depth of the slip surface reaching 20m-35m. The specific section of the project was under the risk of a greater slope instability that could jeopardize the safety of the Highway.

In January 2003, following a long period of heavy rainfalls, a large settlement and cracks were noticed on the pavement, continuously increasing. The installed instruments indicated the slope was unstable. The traffic was deviated to a local network. A month later, in February 2003, the devastating landslide was activated, causing a horizontal displacement of the sliding mass up to 100m, while the vertical one was of the order of 40m. The catastrophic phenomena affected a large area of 400m upslope and 700m downslope (Belokas 2013; Belokas and Dounias 2016).

The Tsakona bridge

Among the solutions that were studied to rehabilitate the National Highway, which included (a) a partial stabilization of the landslide with large excavations and toe weighting, piles, caissons, prestressed anchors and deep drainage, (b) a diversion of the Highway with a tunnel, and (c) a bridge, the latter was the most cost-efficient and technically feasible solution (Fikiris *et al*. 2011).

The bridge project consisted of an access part of about 130m long, made of prestressed concrete and a 260m long steel arch with a steel suspended deck. The abutments and the pier were made of reinforced concrete. Deep foundation with three series of 6 piles connected with pile-cap was designed for the northern abutment A0, while a shallow foundation was selected for the southern abutment A2 towards Kalamata. The foundation of the bridge middle pier M1 consisted of four caissons connected with a large and very thick raft, made of reinforced concrete. The caissons had a diameter of 6.0m and a depth of 15m each, while the raft was 5.0m thick, with a rectangular plan-view having the dimensions of 23.0mx31.0m (Fikiris *et al*. 2011).

The V-shaped supporting struts of the pier were constructed first. For the south strut, which was curved, scaffolding towers anchored on slabs on the ground were used. For the north one, a self-climbing formwork system was implemented, while special temporary tendons and steel stiffeners were designed to provide stability between the two struts (Figure 6). Two towers MT, with bearing capacity 12000kN and 16500kN each, under static and seismic load combination, respectively, were placed below the head of the curved strut to ensure the stability of the incomplete pier system during its construction (Figure 7). These towers remained until the completion of the whole bridge. The sensitive structure of these steel towers should firmly support the V-shaped struts, while contingency measures should take place to compensate any movement within the loose soil sited on the bedrock as well as any deformations of the towers themselves.

Figure 6. Construction of the pier struts

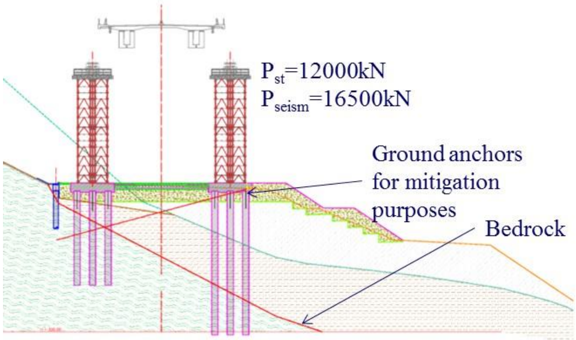


Figure 7. Arrangement of the MT towers

The construction of the 55m prestressed part of the deck followed, ensuring the stability of the V-shaped pier. The 130m access concrete bridge was completed with the 75m prestressed deck between the pier and the abutment A0, which was cast-in-place using heavy duty PERI bridge-type scaffolding, founded with piles on adverse surface morphology and soil conditions (Figure 8). The bridge finished with the 260m twin steel arch and the suspended deck erection. The arch system of the bridge consisted of two separate arches, one for each branch of the highway and K-shaped bracings, all fabricated in the factory. Segments of 36m long were assembled and welded on the ground, before they were lifted on their position with the heavy lifting equipment mounted on the towers. The various 36m segments were then welded at their final elevation. Shelter boxes were placed at the top of the towers to protect the welding procedure from the wind and the rain (Figure 9).

Figure 8. Construction of the prestressed concrete part of the bridge deck

Figure 9. Assembly, erection and welding of the arch segments

Special attention was required in three phases of the construction. The first one was the measuring, cutting and welding of the last piece (key) of the arch. Due to the temperature difference, the gap between the two parts of the arch, already constructed, differed during the day and this difference reached almost 4cm. The gap was fixed to a firm distance, in order to measure, cut and weld the key, using steel plates and pins, designed to sustain a total tensile force between the constructed parts of the arch equal to 10000kN, caused by a temperature uniform difference of 15^{o}C (Figure 10). The second crucial phase of the construction was the gradual removal of the arch towers, without causing bending moments at the arch and additional compressive forces to the remaining towers (Figure 11). A detailed model was set up by the designers of the bridge, defining step by step the sequence of the removal, ensuring the safe activation of the static function of the arch and avoiding undesirable effects. The third and final crucial phase of the construction was the removal of the MT towers, in order to totally liberate the bridge from the supporting structures. The displacements measured on site reached the 10cm settlement and 7cm horizontal displacement at the top of the curved strut, confirming the corresponding displacements calculated by the analysis model.

Figure 10. Fixing the gap between the arch parts to weld its last piece

Figure 11. Removal of the towers

Geological and geotechnical conditions

In the area of the landslide, the subsurface consists of flysch (sandstones and siltstones prevail) while in deeper layers the presence of limestone prevails. The products of weathering of these formations cover the bedrock up to a depth of 40m in some cases. For the design of the infrastructure projects, the crucial characteristics of the area were, (a) the depth the weathered material (10-35m), (b) the steep surface of the bedrock, with an inclination angle ranging from 12^{o} to 30^{o}, which locally in the central part of the slide reached up to 40^{o} and (c) the presence of the ground water mainly within the permeable limestones, which at the border with the overlying flysch was flowing in the form of springs feeding the weathered soil type material and thus creating conditions of low shear strength. The described adverse and variable morphology enabled a lot of uncertainties on the proper characteristic geotechnical section for the design and on the other hand created a high risk of sudden local slides within the area of the worksite. The above uncertainties and risks, combined with (a) the observed fact that the activity of the slide from 2000 to 2010 changed from periods of very low to very intense movements, (b) the aforementioned geotechnical regime, (c) the importance and sensitivity of the erection activities of the bridge arch and (d) the safety requirements of the highway traffic which although temporary would cross the body of the landslide, called for the reliable assessment of the sliding mass behavior and the close monitoring during construction.

Instrumentation – Monitoring – Early warning system

The fact that the landslide was active with the margins of safety against failure being very small led to the implementation of an instrumentation and monitoring network, consisting of conventional inclinometers, piezometers and optical targets as well as fully automated inclinometric arrays, combined with electrical piezometers and rain gauges located at key depths within the sliding zone. Optical targets were also installed on structural elements. In Figure 12 the location of all instruments is shown in plan-view, while in Figure 13 a characteristic section is illustrated presenting the position of the instruments with respect to the bedrock surface. The automated instruments provided real-time records of the movements and the piezometric levels. They continuously updated a software application (developed on excel), which performed real-time evaluation of the readings and the trends of the phenomena. The system was accessible by the responsible engineers of the monitoring and construction team and was used to assess at any point the potential risk of the landslide. In addition to the continuous evaluation of the readings, alert and alarm limits were introduced in the data logger, which, in such a case, was programmed to send SMS messages to the engineers in charge, in order to take immediately actions preventing any accidents due to potential excessive movements or slides (Seferoglou and Chrysohoidis 2016). The schematic function of the automated monitoring is shown in Figure 14, while the flow chart is given in Figure 15.

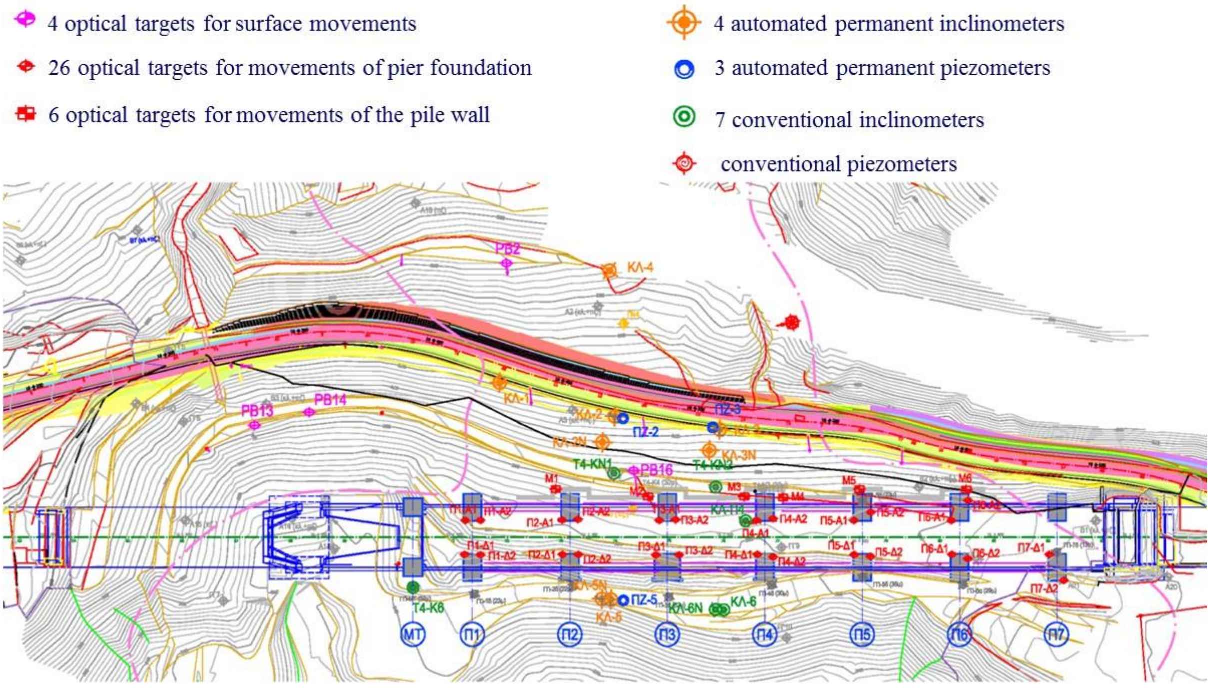


Figure 12. Position of the intruments in plan-view

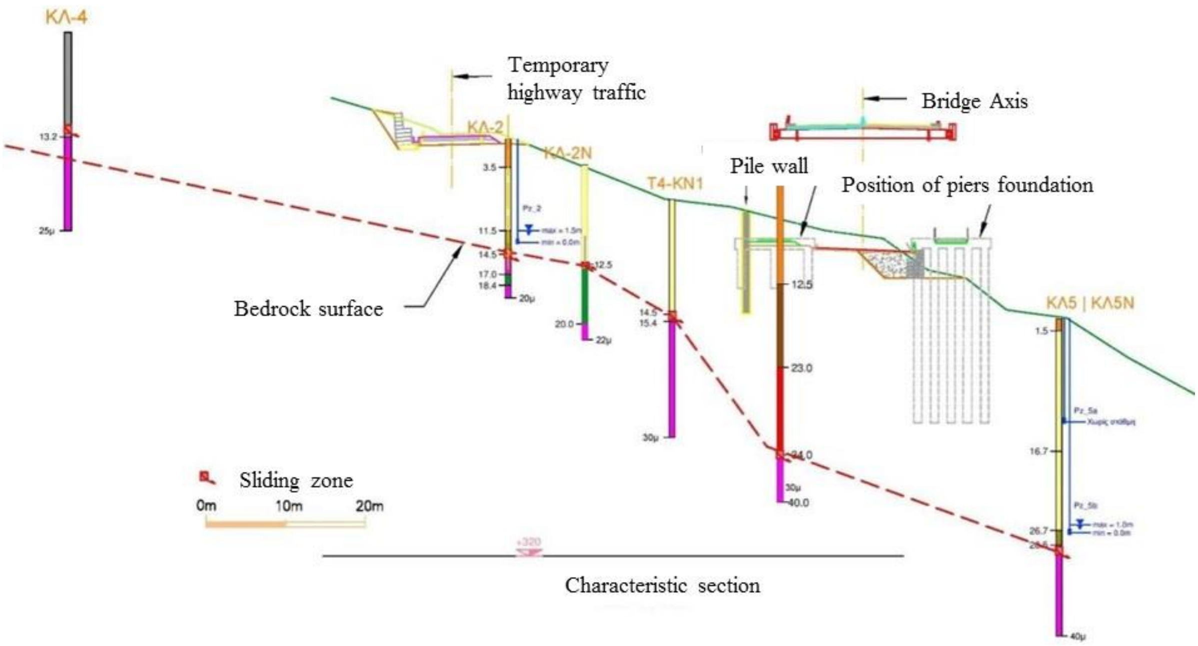


Figure 13. Typical section showing the intruments and the bedrock surface

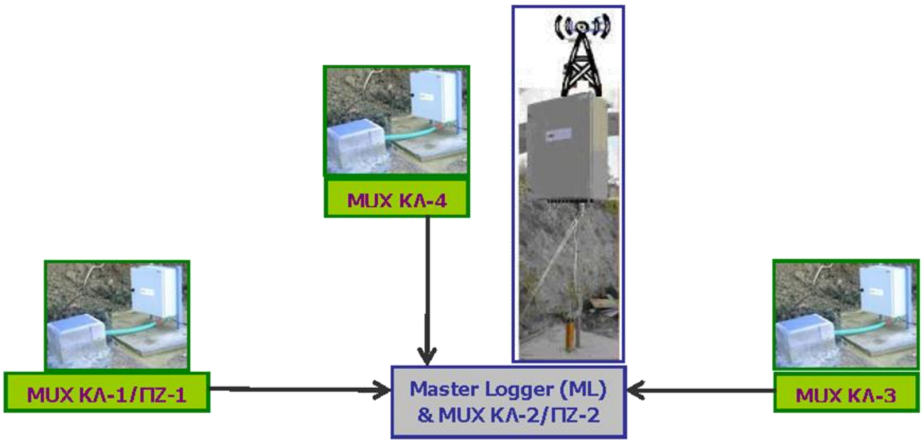


Figure 14. Automated Monitoring – Schematic function

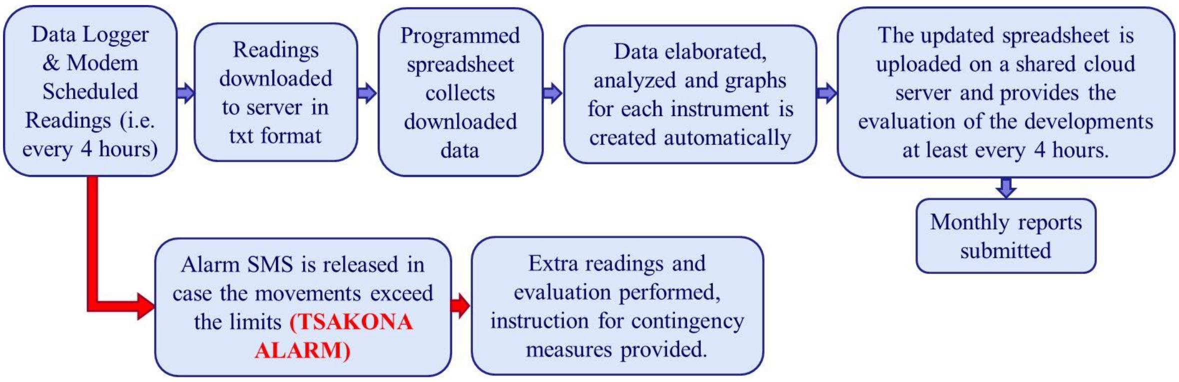


Figure 15. Automated Monitoring – Flow chart

The instrumentation and monitoring project started early 2012 and was completed on January 2016. During that period, several incidents of excessive movements were recorded and special instructions were given to the construction team. It should be noted that several seismic events in the region were also monitored during the construction period. The early collected data were also used to optimize the design of the deep foundations within the sliding mass, aiming to support the assembly towers and the retaining wall of the temporary traffic. The main conclusions drawn from monitoring results were the following:

- Deep movements (inclinometric readings) and surface movements (optical targets movements) were in good agreement, as shown from the data of Table 1.
- A seasonal variation of the rate of movement of the slide was noted related to the rainfall of the previous months, as shown in the diagram of Figure 16.
- Differences of the rate of movement were observed among the various monitoring locations (13 - 40mm/year) (Figure 17). Factors influencing this rate were the depth of the sliding surface and the inclination of the bedrock, which differed significantly along a transverse section, as shown in Figure 13.
- For the design of the projects a construction period of two (2) years was assumed and the proposed design movements were: (a) 30mm/year transverse to the bridge axis, (b) a total differential movement between temporary piers in the longitudinal direction of about 15mm/year and (c) a differential settlement equal to 25mm/year between the piles of adjacent piers of the right branch.

Table 1. Comparison of deep vs surface movements (28/03/13 ÷ 7/03/14)

Location of readings	Depth of sliding zone	Mean total annual movement
Sliding zone T4-KN1 and T4-KN2	15-20m	27mm
Optical targets on the pile wall	18-22m	25mm
Optical targets on pile caps Π2A, Π3A, Π4A	18-20m	29mm
Optical targets on pile caps Π2Δ, Π3Δ, Π4Δ	30-34m	34mm
Sliding zone ΚΛ5 and ΚΛ6	35-40m	32mm

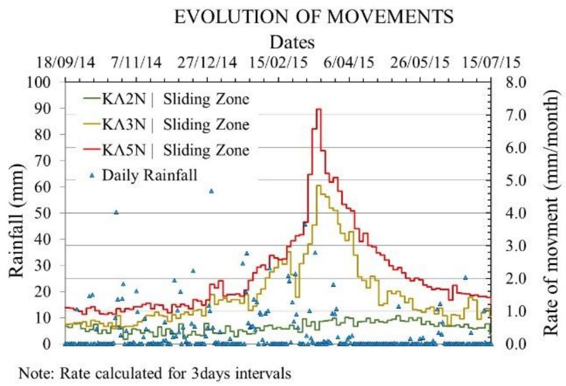


Figure 16. Diagram of movement evolution

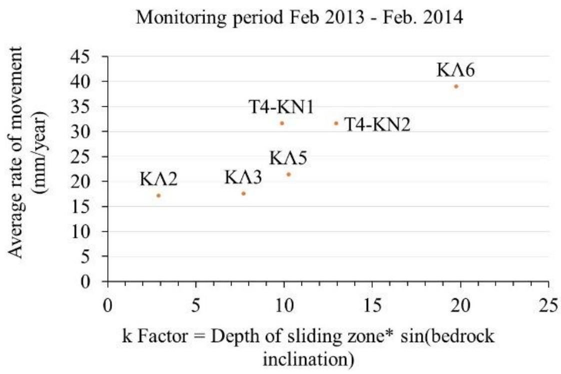


Figure 17. Rate of movement for different locations

Design of the piers

Design criteria

The most challenging of the required temporary infrastructure was the design of the foundation of the 14 steel towers used for the heavy lifting and final assembly at height of the 36m parts of the arch pair. The demanding stability within a moving mass, combined with the very tight accuracy requirements of the arch welding activities and the significant loads transferred to the ground comprised the technical framework for the design of these works.

The selection of the most appropriate solution among the two options of shallow or deep foundation was defined initially by the geotechnical conditions of the specific area, characterized by various materials (i.e. debris of landslide, weathered flysch bedrock, colluvia of limestone, recent embankment and fill materials), sitting on the firm bedrock surface. Another important criterion of the design was the morphology of the construction area, which was very steep presenting intense transverse slopes near the piers Π1 – Π5 of the right branch (Figure 18). Moreover, the intense inclination of the bedrock surface, in combination with the continuously evolving sliding movement downslope, constituted another influential factor for the design.

Figure 18. Piers of the right branch

The solution of the shallow foundation was excluded immediately. The assembling line of the right branch was very close to the edge of the slope within the sliding mass and the danger of local failures and large settlements could not be neglected. Moreover, in order to create slabs on the ground, additional soil masses should be placed on the sliding body, which would increase the risk of local failures due to the additional weight. It should be mentioned that extreme settlements or local failures would have been catastrophic if occurred during the erection and welding of the arch segments.

Hence, the solution of pile groups was decided, independent for each pier. At first, the case of long piles, which would cross the sliding surface penetrating into the bedrock, was studied. Nevertheless, it was soon abandoned because such a solution could not stop the evolving sliding movement or prevent an eventual new activation of the landslide. The piles would have reached very soon shear failure at points where the interface between the sliding mass and the bedrock surface was crossed. It would only increase the bearing capacity of the piles and the cost of their construction would exceed the permissible limits of a temporary project. Thus, taking into account the expected loads on the top of the piles (maxP=2000kN), the solution of friction piles, floating into the sliding mass was selected for the piers of the right branch, where the thickness of the soil materials was large. For the left branch, where the bedrock surface was encountered at less than 5m, the piles entered in the rock mass, in order to ensure adequate bearing capacity.

It should be clarified that the scope of these interventions was not the stabilization of the landslide, or the ensuring of satisfactory safety factors for the sliding surface of 2003, that was impossible to obtain, but the safe erection of the arch, under the assumptions that the construction period of the arch would not exceed two years, the movements of the sliding mass would continue at the same rhythm as the one evolved the last decade, and the landslide would not be reactivated during the works.

Design methodology

In order to assess the influence of the sliding movements to the response of the piles and pile-caps of the piers, a characteristic transverse section was selected between the pier Π2 and Π3, which was considered the most critical one, because of the depth of the bedrock surface and the free height of the piles of the right branch. This characteristic section is illustrated in Figure 19.

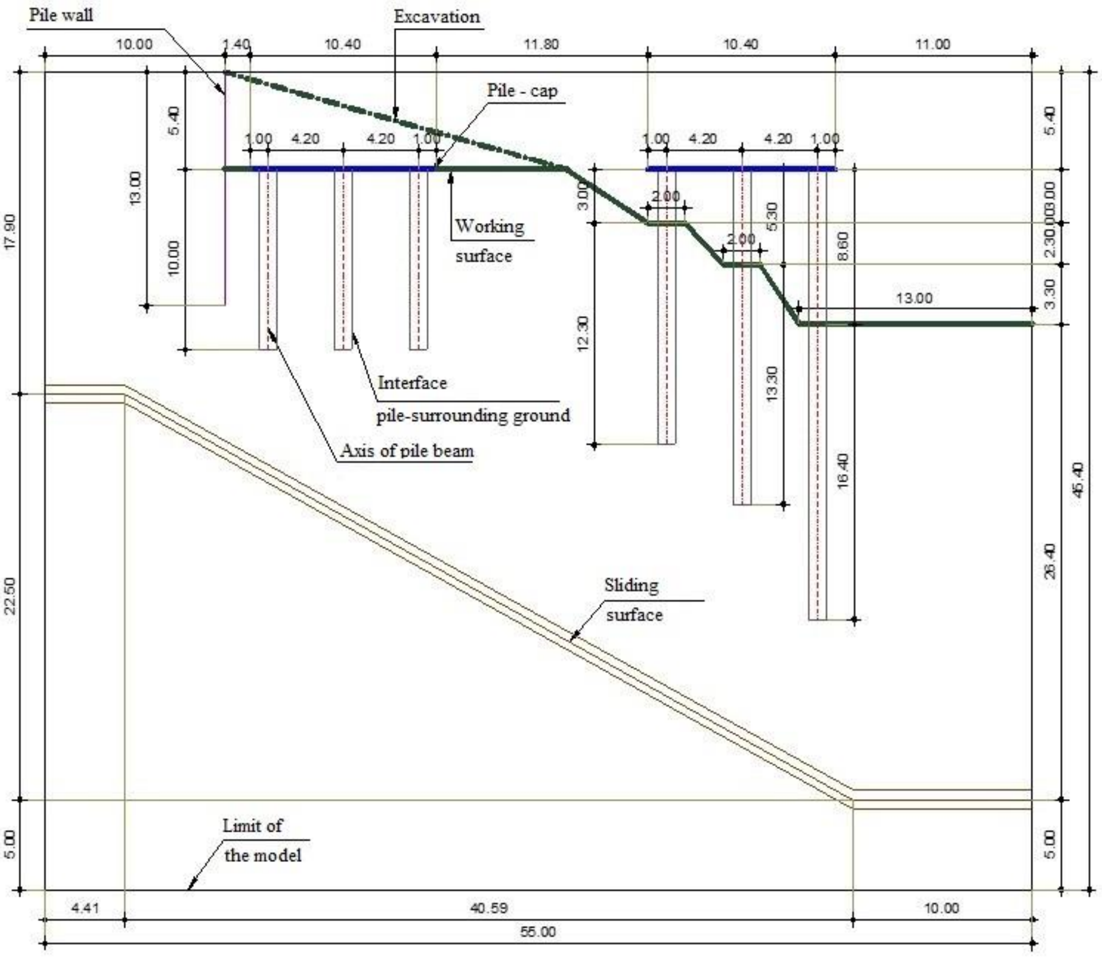


Figure 19. Characteristic transverse section

The design methodology that was followed consisted of five (5) phases as described in the flowing chart of Figure 20, adopting the requirements of the Eurocodes and other current codes like DIN4014. The models used for the geotechnical and structural analyses performed were the following (Seferoglou and Vassilopoulou 2016):

- As the major concern was the influence of the landslide movements on the bending moments and shear forces on each pile group, it was decided to use initially a 3D model (ABAQUS) as a guide for the geotechnical assessments. The excitation due to soil mass movements was introduced using a real profile of the movements as it was depicted from the monitoring data. External loading from the superstructure was also added.
- In order to avoid the complex and time consuming procedure of analyzing each pair of piers with the above method, the 3D «guide model» was then used to calibrate 2D geotechnical models (PLAXIS) for each pair of the 14 piers, which led to the assessment of bending and shear increase due to the soil movement, considering in each case the local morphology of the sliding surface and the local profile of the soil mass movement.
- Finally, the results of the geotechnical analyses provided the required data to feed more detailed 3D structural analysis (SOFiSTiK) used for the optimization of the dimensioning and required reinforcement of each structural element of the foundation.

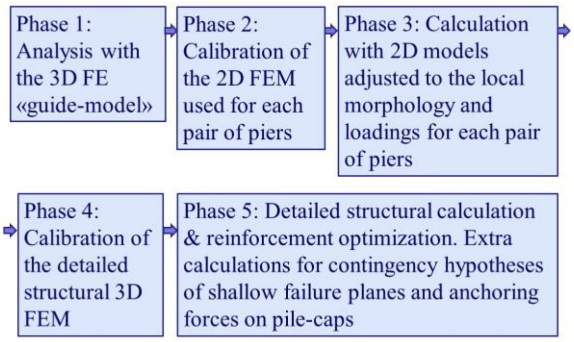


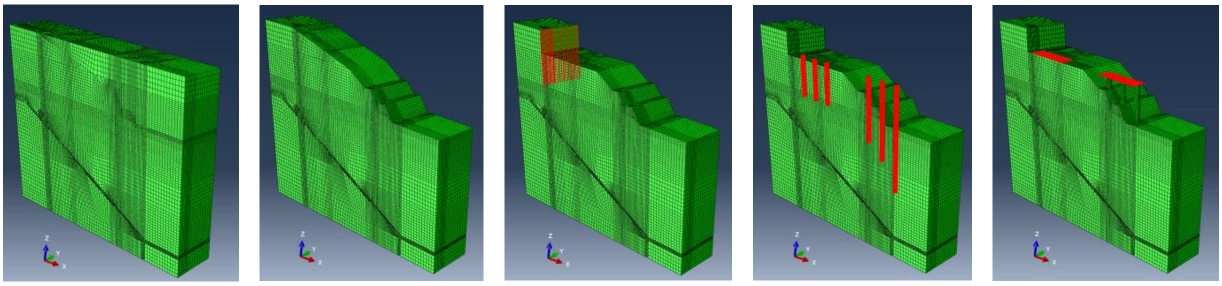
Figure 20. Design methodology – Schematic form

Analysis of the 3D finite element «guide model» with ABAQUS

For the «guide model» solid (eight-nodal hexahedron) finite elements were used to simulate the surrounding ground following the Mohr-Coulomb failure criterion, shell (four node quadrilateral) finite elements for the pile-caps and beam elements for the piles. The retaining wall supporting the temporary deviation of the Highway is also simulated with beam elements. The holes at the ground created by the piles were filled with solid finite elements with zero elastic modulus. In order to reduce the time of analysis, a symmetry plane was considered, perpendicular to the longitudinal axis of the bridge, passing from the transverse axis of the pier pair. Hence, only half of the model is set up. The selection of the model dimensions and boundary conditions, as well as the procedure of the analysis was based on the methodology proposed by Kourkoulis *et al*. (2011, 2012). Seven steps are considered for the numerical analyses accounting for the actual construction stages of structure, which were the following:

- **Step 1:** Geostatic state with horizontal ground surface.
- **Step 2:** Remove of the ground material forming the existing morphology.
- **Step 3:** Simulations of the excavation at the level of the piles head. Activation of the shell elements of the retaining wall.
- **Step 4:** Deactivation of the finite solid elements into the holes of the piles, simulating the ground. Activation of the beam elements of the piles and the solid elements with zero stiffness.
- **Step 5:** Activation of the shell elements of the pile-caps.
- **Step 6:** Loads from the towers.
- **Step 7:** Imposed horizontal movement of 6cm, accounting for two years of construction (3cm/year).

Figures from the model at different steps of analyses are given in Figure 21, while Figure 22 shows the calculated soil displacement due to the imposed horizontal movement at the end of Step 7. Due to the morphology of the ground and the free height, the most critical internal forces are noted at the piles of the right branch. Based on the analyses conducted, the slope movement practically led to a uniform movement of the soil above the failure plane, without differential displacements, causing small increase of bending, shear and axial internal forces of the piles, (ΔN=+3.4% (Figure 23), ΔQ=4.2% (Figure 24), ΔM=37.5% (Figure 25)). These results were used to calibrate the other two approaches with PLAXIS and SOFISTIK and their assumptions. The diagrams of the horizontal displacements of the piles of the right branch pier, given in Figure 26, shows that at the end of step 7 the displacements are almost the same along the piles.



Step 1 Step 2 Step 3 Step 4 Step 5

Figure 21. Different steps of analysis for the «guide model»

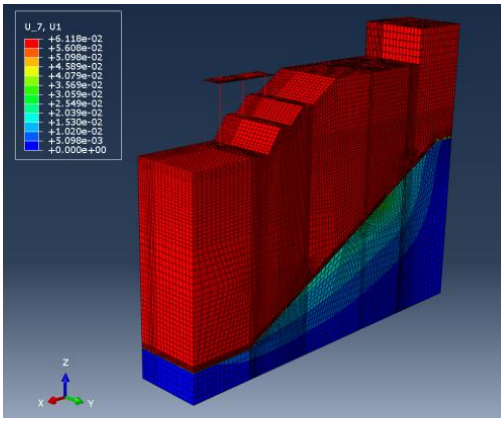
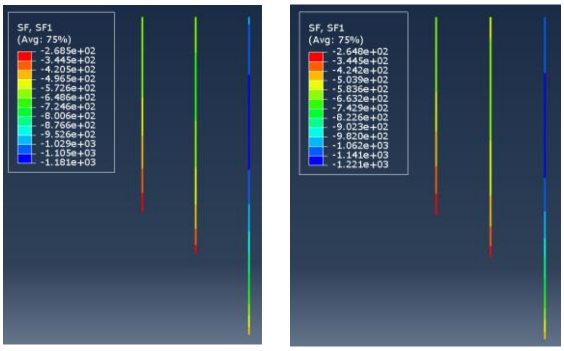
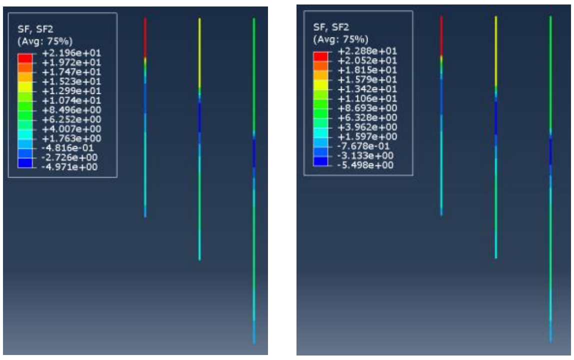


Figure 22. Transverse slope displacement due to the imposed movement



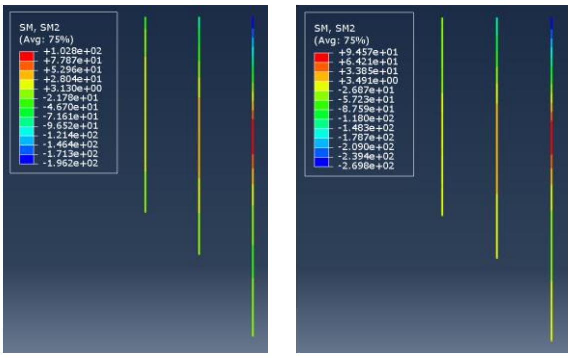
(a) (b)

Figure 23. Axial forces of the piles of the right branch: (a) step 6, (b) step 7



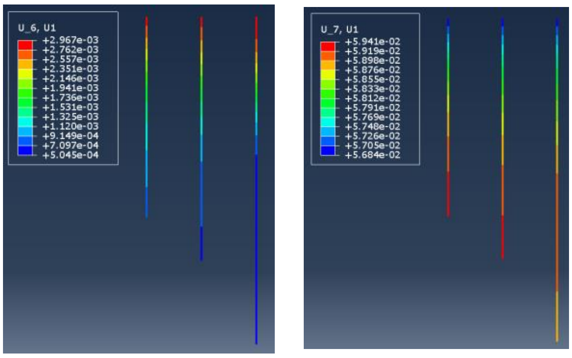
(a) (b)

Figure 24. Shear forces of the piles of the right branch: (a) step 6, (b) step 7



(a) (b)

Figure 25. Bending moment with respect to the longitudinal axis of the bridge of the piles of the right branch: (a) step 6, (b) step 7



(a) (b)

Figure 26. Horizontal displacement in the transverse direction of the bridge of the piles of the right branch: (a) step 6, (b) step 7

Analysis of 2D finite element models with PLAXIS

Each pier should be studied separately due to the different morphology of the ground and the geometry of the bedrock surface, presenting intense slopes. Hence geotechnical models should be set up for each pair of piers. The 3D geotechnical model, which was firstly used for the general conclusions and the delineation of the basic guidelines for the selection of the foundation type (shallow or deep), was very time consuming regarding the configuration of the model, the calculation and the evaluation of the results. Thus, conducting analyses of each pier with similar 3D finite element models was not efficient. For this reason, the software PLAXIS was used, performing 2D analyses for each section of the project, after calibrating the results by tuning the response of the 2D model representing the geometry of the characteristic transverse section of Figure 19 with the one of the 3D «guide model». The model used for the calibration of PLAXIS is illustrated in Figure 27.

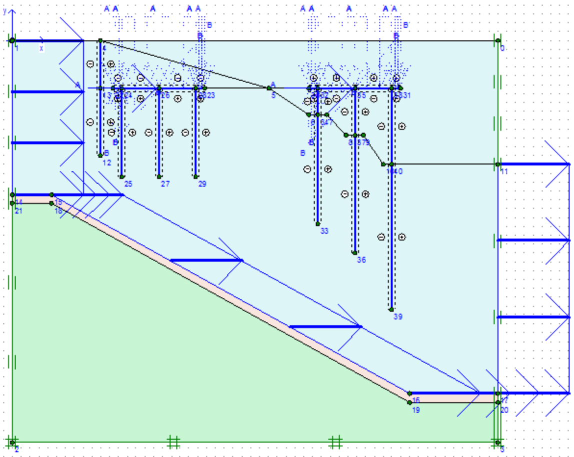


Figure 27. 2D finite element model used for the calibration of PLAXIS with the «guide model»

The piles were simulated with element stiffness which corresponded to 2 piles in series. The geometry of the excavations, the geotechnical parameters and assumptions as well as the construction stages considered for the 3D analyses were adopted for the 2D analyses, too. At the same time, an imposed movement equal to 6cm was taken into account, applied at the border of the model, similar to the one considered for the «guide model». The analyses steps were the following:

- **Step 1:** Geostatic state
- **Step 2:** Excavation and construction of the pile wall for the temporary deviation of the Highway.
- **Step 3:** Construction of the foundation (piles and pile-caps).
- **Step 4:** External loads from the towers, applied on the pile-caps.
- **Step 7:** Imposed horizontal movement of 6cm.

The internal forces of the piles and the calculated transverse slope displacement due to the imposed movement were compared with the corresponding ones of the «guide model», reaching a satisfactory agreement.

The final design of the towers foundation (final dimensioning and required reinforcement) was implemented using the software SOFiSTiK, for which vertical springs at the bottom of the piles were needed to be used, simulating the soil-structure interaction, as there was no possibility to directly simulate the ground surface geometry and the inclined bedrock interface. The calculation of the constants of the equivalent springs that would be used for the structural analysis of the piers was achieved by using a 2D finite element model for each pair of piers, imposing a load equal to P=1000kN on each pile head and calculating the settlement vz. The equivalent spring stiffness was expressed as K=P/vz. As an example of this approach the model of the analysis for pier Π5 is illustrated in Figure 28. Moreover, the results of these 2D analyses were used to define the earth pressure evolved at the piles of the piers at the right branch founded in the sliding mass. Calculating the horizontal and vertical stresses, an equivalent pressure coefficient yielded, which resulted in a value varying between 0.33 and 0.47. Thus, for the safety side a pressure coefficient equal to 0.50 was assumed. This value was adopted for the static analyses. The same procedure was also followed for the calculation of the earth pressure due to the imposed movement, but the equivalent pressure coefficient reached the same value, since the whole sliding mass and the floating piles were moved uniformly.

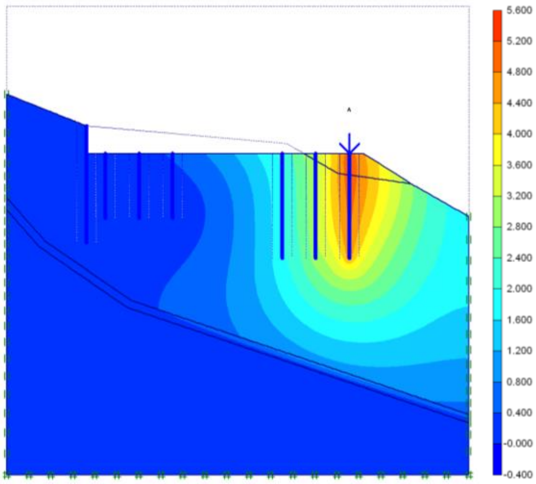
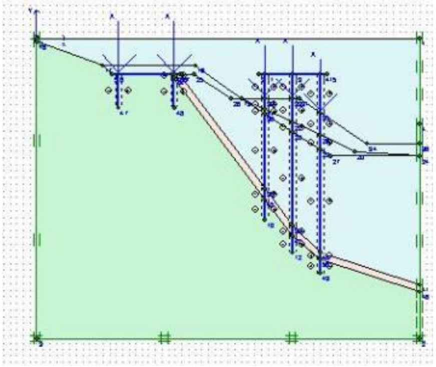
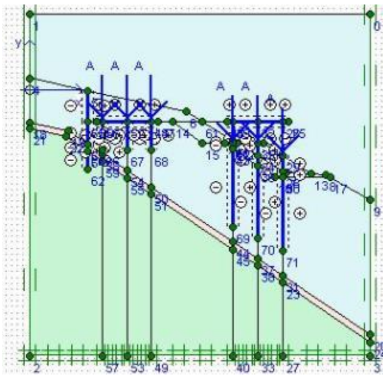


Figure 28. Total vertical displacement (in mm) of the outer pile of pier Π5

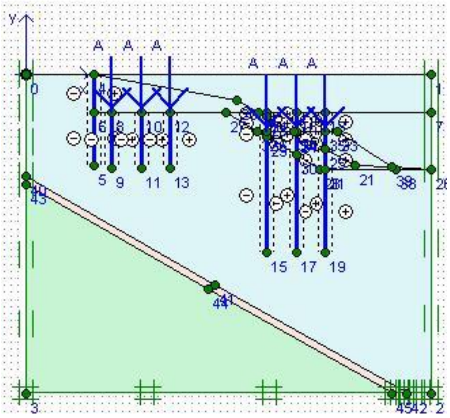
For all analyses performed, the exact geometry of the ground at each position was taken into account and the same steps considered for the calibration of the program were followed. The seven models created for each pair of piers are illustrated in Figure 29, where the different morphology and geometry of the bedrock surface can also be seen.



П1



П2



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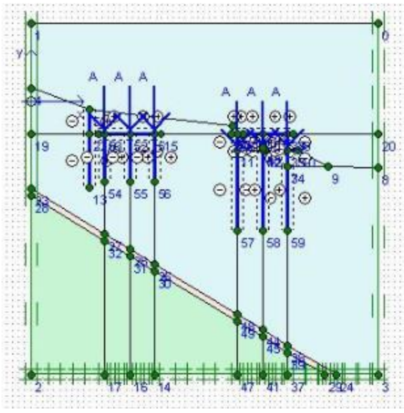
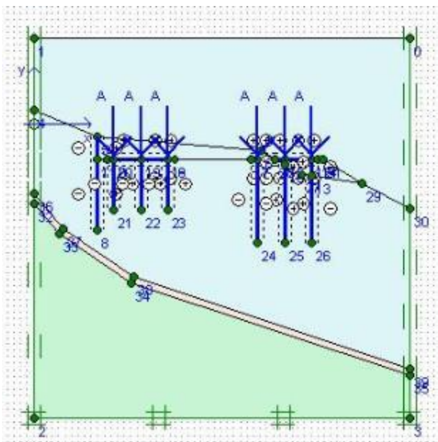
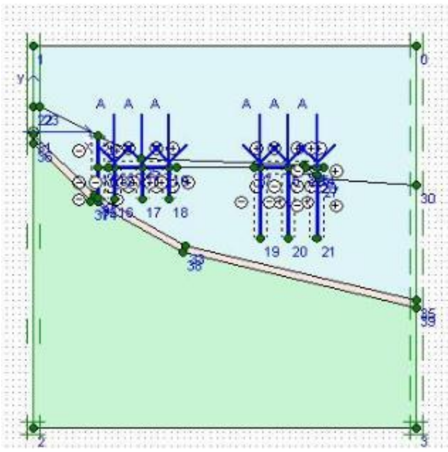


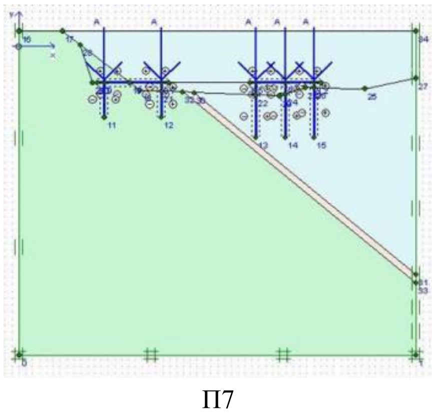
П4



П5



П6



П7

Figure 29. Models for the 2D geotechnical analyses

Analysis of 3D finite element models with SOFiSTiK

In order to calibrate the static results with the ones derived by the previous analyses, first, the 3D finite element model of Figure 30 was set up based on the characteristic transverse section of Figure 19, in order to compare the response of the model with the one of the «guide model». The soil-structure interaction was taken into account with horizontal springs, perpendicular to the pile and vertical ones applied at the bottom of the piles, extracted from the PLAXIS, as described in the previous section. The self-weight, the towers loads and the earth pressure, with coefficient K_o=0.50 were the only load cases. Comparing the internal forces of the piles with the ones calculated for the «guide model» small differences were noted, concluding satisfactorily the calibration between the two approaches. Hence, further static analyses followed for each pier, in order to take into consideration the seismic action (0.08g), reduced due to the temporary character of the project, with important factor γ_I=1.30, accounting for the increased seismic hazard conditions and the public safety of the whole project. Contingency working hypotheses are also assumed, such as local shallow failure planes developed at height 8m-10m from the ground surface, introducing a 15mm transverse movement (Figure 31) and anchoring forces applied on the pile-caps (Figure 32) in case the movements exceeded the predicted values or the tolerance of the superstructure during the erection of the steel arches.

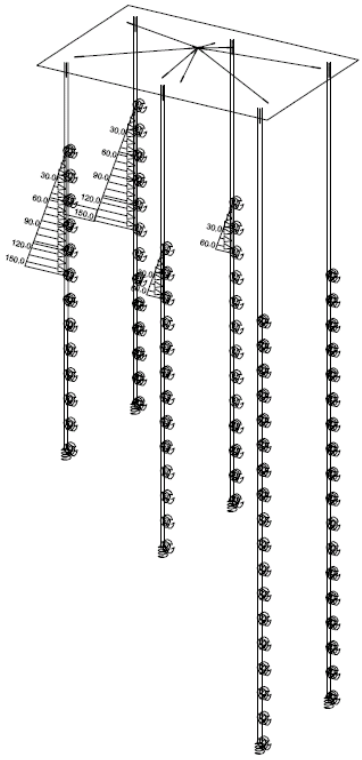


Figure 30. 3D finite element model for the calibration of SOFiSTiK

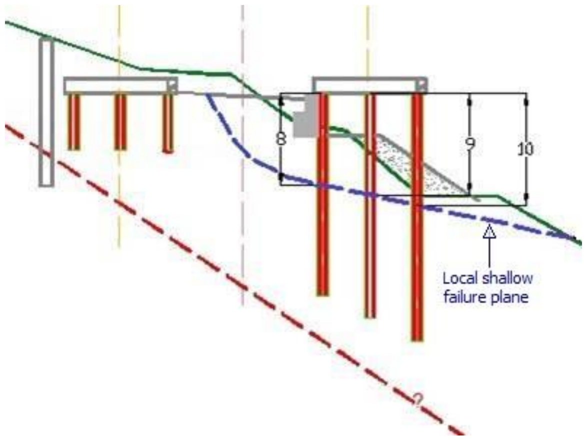


Figure 31. Contingency local shallow failure planes

Figure 32. Prediction of anchoring of the pile-caps

The internal forces developed at the top of the piles of the most crucial piers Π2Δ – Π5Δ, where the maximum required longitudinal reinforcement were noted, were compared with the corresponding ones calculated by PLAXIS considering the imposed movement of 6cm due to the sliding of the soil mass. The comparison showed that the soil movement did not increase the internal forces significantly. More specifically, the axial forces were increased by 2.5% and the bending moments by 6%. In order to take this into account, it was decided to increase the longitudinal reinforcement of the piles of those piers by 10%.

Summary and conclusions

In this article the methodology of the geotechnical and structural design of the temporary towers foundations was described, which were used for the erection of the arch of the Tsakona bridge in Greece. These towers were founded in an active landslide, where an instrumentation and monitoring system was installed, analyzing and evaluating continuously the evolving movements of the sliding mass during the construction works of the bridge. The peculiarity of the project determined the approach of the design, which consisted on variable phases of analyses, with several 2D and 3D geotechnical and structural models, in order to calculate the behavior of the temporary deep foundation under the imposed displacement induced by the active landslide. Deep foundation with piles that would not penetrate the bedrock surface was selected as the most efficient and cost-effective solution, under the assumption that the duration of the construction works would not exceed two years. The results of the analyses showed that the sliding mass moved uniformly, without causing differential displacement to the piles of the towers foundation, thus, without increasing significantly their internal forces. The project was successfully completed in March 2016 (Figure 33) and the imposing bridge is standing safely in the landscape of the southern Peloponnese, rendering proud those involved in the project.

Figure 33. The completed Tsakona bridge

Acknowledgements

The design and construction of the project has been supervised by the Greek Authorities (DMEO, D1, EYDE/DDP) of the Ministry of Infrastructure, Transport and Networks. The contractor of the project was initially the J/V TERNA S.A. – ALPINE BAU and then TERNA S.A. The design and construction team consisted of the following:

- Structural design of the Tsakona bridge: DOMI S.A.
- Geotechnical and foundation design of the Tsakona bridge: EDAFOS S.A.
- Design of infrastructure projects and construction consulting: ODOTECHNIKI Ltd
- Design of heavy lifting scaffolding: PERI Formwork Scaffolding Engineering
- Steel fabrication and erection: EMEK S.A.
- Consulting support on optimization of ABAQUS by Prof. I. Anastasopoulos and K. Tzivakos, Civil Engineers.

The whole project was characterized by many uncertainties, adversities and risks for catastrophic events, such as a new activation of the landslide during the construction or the erection of the arch. The excellent collaboration between the geotechnical and structural teams, the construction team and the consultants was the fundamental element for the success of the project.

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