Foundation of the arch bridge in the landslide area of Tsakona, Greece

Fondation du pont en arc dans la zone de glissement de terrain de Tsakona, en Grèce

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ABSTRACT

The Tsakona bridge is a two-span (90m and 300m) steel arch structure, which will be part of the National highway connecting Tripolis and Kalamata, in southern Greece. It is planned to pass over and therefore avoid a large landslide, which in 2003 disrupted completely the traffic in the highway. The paper briefly describes the geological and hydro-geological conditions of the area, as well as the triggering instability mechanism of the landslide. Design criteria for the adoption of such a unique bridging solution are reviewed and the approach followed for the foundation design of the bridge, covering both the selection of appropriate locations and suitable types, is presented. Design and construction processes are described with particular reference on the foundation of the middle pier, which consists of a system of four caissons connected with a large and very thick raft. The detailed geological-mapping that was carried out during construction is also presented which verified fundamental design assumptions regarding ground conditions and geotechnical parameters.

RÉSUMÉ

Le pont de Tsakona est un pont en arc en acier qui comprend deux travées (90m et 300m) et fera partie de l'Autoroute Nationale reliant Tripoli et Kalamata, au sud de la Grèce. Le pont est prévu de passer au-dessus et ainsi évitez un glissement important de la région, qui a perturbé complètement le trafic sur l'autoroute en 2003. L'article décrit brièvement les conditions géologiques et hydrogéologiques de la région, ainsi que le mécanisme qui a déclenché l'instabilité du glissement de terrain. Les critères de conception, pour l'adoption d'une telle solution unique de traverser, sont passés en revue et l'approche suivie pour la conception de la fondation du pont est présentée, couvrant à la fois la sélection des emplacements et des types appropriés. Les processus de conception et construction sont décrits avec particulière référence a la fondation de la pile intermédiaire, qui se compose d'un système de quatre caissons liés à un radeau grand et très épais. La cartographie géologique détaillée qui a été réalisée lors de la construction est également présentée. Cette cartographie a vérifié les hypothèses fondamentales de conception en ce qui concerne les conditions du sol et les paramètres géotechniques.

Keywords: Bridge foundation, landslide, caisson, numerical modeling, geological mapping, design verification.

1 INTRODUCTION

The Tsakona landslide took place at the beginning of 2003, disrupting completely a 250m long section of the Tripolis - Kalamata national highway, in the southern Peloponnese (Fig. 1). The landslide involved the mobilization of about 6.000.000m^3 of both flysch colluvium and man-made deposits. The maximum horizontal displacement was of the order of 100m.

Early signs of instability in the area were observed just few months from the completion of

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Figure 1. View of disrupted highway section due to landslide.

the pavement structure in 2000. Geotechnical investigations in 2001 revealed that the highway crosses a zone of an ancient landslide, whereas supplementary ground investigations and geological mapping carried out after reactivation defined precisely the landslide limits and its depth.

The paper briefly describes the geological and hydro-geological conditions of the area, as well as the triggering mechanisms. A detailed review of the landslide can be found in [1] and [2]. Specific focus is made on the design and construction of the foundations of a 300m span bridge, which was selected as the most economical and technically feasible solution in order to avoid the zone of the landslide.

2 GEOLOGY – LANDSLIDE MECHANISM

Geologically the wider area belongs to the Pindos geotectonical zone. At the landslide area the chert series of lower cretaceous age dominates. It consists mainly of radiolarites in alternations with siltstones, sandstones and red marls. The dominating direction of the geological strata is N – NNE. The thickness of the series is estimated to be about 40-50m. The crown of the landslide coincided with an outcrop of an upper cretaceous limestone, while the toe appeared at a river gorge, some 1200m downslope. The contact between the overlaying upslope limestone with the chert series is tectonic (upthrust) and played a significant role in the activation of the landslide phenomenon.

The entire area is covered by talus scree and man-made deposits of various thicknesses. The geological structure is complex and is dominated by numerous tectonic structures such as folds, imprications and thrust with their main axis direction to NW-SE and NE-SW.

A combination of factors including site morphology, geology, human intervention and groundwater played an important role in the landslide reactivation.

The unstable area corresponds to a small basin in which successive accumulations of debris and material from past failures took place progressively with time. The accumulation of such material was a result of the presence of very weak flysch and chert formations.

The groundwater regime, during the very wet winter of 2003 highly influenced the triggering of the instability. A significant geological feature of the area is the presence of the highly permeable limestone, which is a continuous water supply source to the problematic zone since it rests on an aquifuge chert series formation.

Finally, construction activities including large-scale earthworks for the construction of the new highway alignment were an adversely influential factor to the already marginal stability of the slope.

The landslide of 2003 was very extensive with a length of 1200m, average width of 300m and maximum depth, as recorded by inclinometers of the order of 23m. A geological longitudinal section along the axis of the landslide is presented in Fig. 2.

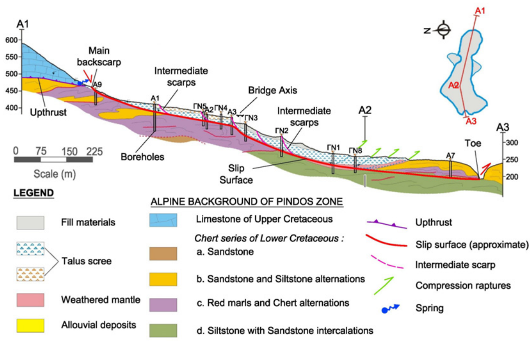

Figure 2. Geological section along landslide axis.

3 DESIGN CRITERIA – SOLUTION

Various stabilization measures were examined, at the early design stages including solutions with earthworks (i.e. excavations and toe weighting), deep drainage, local stabilisation with pile walls or barrettes, generic landslide stabilisation with pile grids etc. Due to the volume and extend of the unstable mass the implementation of an acceptable safety factor was deemed impractical [3]. Moreover, the uncertainties involved during the lifetime of the project by adopting a stabilization solution could not be minimized. A diversion solution by means of a tunnel could have been a technical solution but the cost was much higher in comparison to a large bridge, which also has the advantage that it preserves the existing alignment.

Hence, the solution of a bridge was adopted, which avoids the landslide by passing over it and ensuring that a safe free space is maintained between bridge deck and the excavated ground surface against possible future reactivations (Fig. 3).

A particular feature of the bridge is its large span, 300m long, which will be the second longest ever built in Greece. Due to the size of the span a steel arch was designed, the shape of which introduces also a permanent load component perpendicular to the direction of the landslide movement, creating favourable stability conditions.

A main concern for the feasibility of the bridging solution was the assurance that safe and

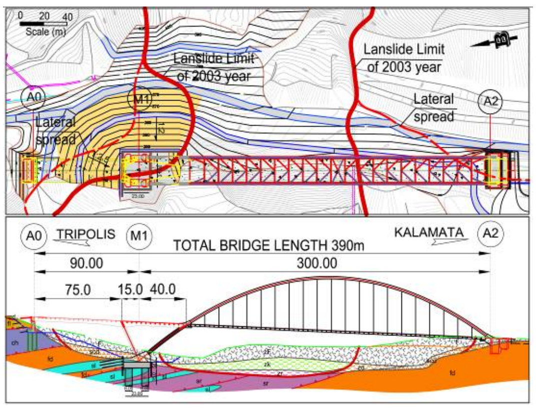

Figure 3. Plan view and longitudinal geological section of the arch bridge

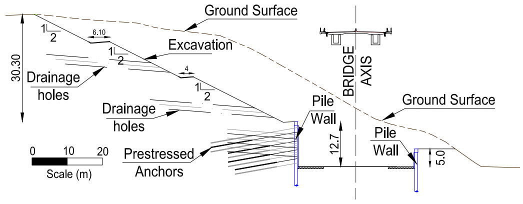

Figure 4. Typical cross-section of open-air excavations with permanent support measures around central pier M1.

stable locations exist for the construction of the foundations. The detailed geotechnical and geological investigations carried out indicated that the landslide limits along the alignment of the highway could be appropriate locations for abutments construction. The foundation of the central pier M1, however, required large-scale earthworks for the local removal of all landslipped soil masses, in order to reach an appropriate elevation where in-situ and stable bedrock exists. Such restriction required extended slope terracing for the provision of safe long-term conditions around the foundation (Fig. 4). For the deeper vertical excavated parts anchored pile retaining walls were used.

4 ARRANGEMENT OF FOUNDATIONS

4.1 *Northern abutment (A0)*

A typical geometrical arrangement was designed for abutment A0, which was founded on a group (3x6) of piles Ø120, 15m deep each (Fig. 5). All piles were connected with a 2.0m thick pile cap (9x21.5m in plan view) and were drilled within mainly the siltstone phase of flysch, classified as

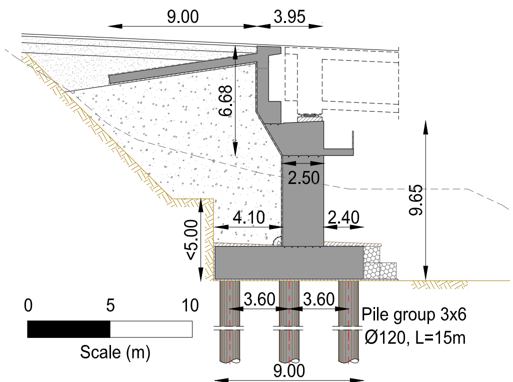

Figure 5. Typical cross-section of abutment A0.

F to E according to the categorization of [4].

Representative geotechnical parameters of Hoek – Brown failure criterion are GSI=20-25, σ_c=2 - 3.5MPa and m_i=8 following proportional corrections, according to the encountered lithological appearances [5]. The equivalent Mohr – Coulomb strength parameters are φ'=25-27°, c'=40-45kPa. The deformation modulus for the rockmass is of the order of E=300-350MPa.

4.2 Central pier (M1)

The foundation of central pier (M1) is the largest ever constructed inland in Greece. It consists of a group of four caissons (shafts) having 6.0m diameter and a depth of 15m each (Fig. 6, 7). They are connected with a 5.0m thick rectangular 23.0x31.0m reinforced concrete raft. The siltstone phase of flysch dominates in the bedrock at the foundation area with similar geotechnical properties as of abutment A0.

The depth of in-situ and stable bedrock at the down-slope edge of the raft determined final excavation level. It was reached following the construction of perimetric pile walls. Extensive application of pre-stressed anchors and long drainage holes was required within an adverse geotechnical environment, where increased water inflows were encountered. The rectangular in shape raft foundation level was also used as a construction platform for the excavation of the caissons. Four deep shafts were constructed using a staged diagonal excavation process. Temporary support measures included shotcrete, steel sets (HEB180) and drainage holes (Fig 7). Prior to their excavation two reinforced concrete base slabs (9.0x23.0m, d=0.6m) were in-situ pure connecting each one a pair of shafts, for the enhancement of the stability of pile walls due to their proximity with shaft locations. Following each excavation completion, all shafts were filled with reinforced concrete and connected monolithically with a base raft creating a rigid foundation for the safe transfer of the very high loads of the superstructure to the ground. Foundation construction required approx. 5500m^3 of concrete.

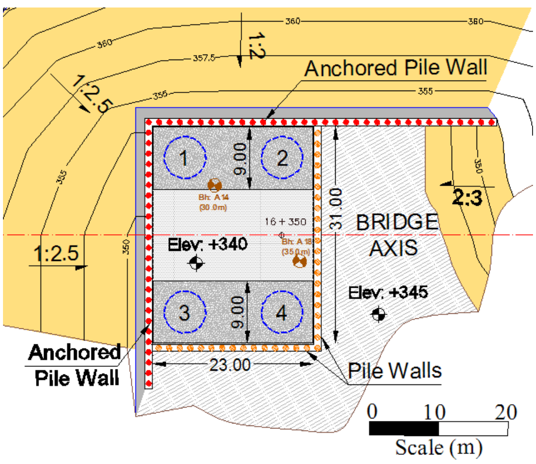

Figure 6. Plan view arrangement of central pier foundation.

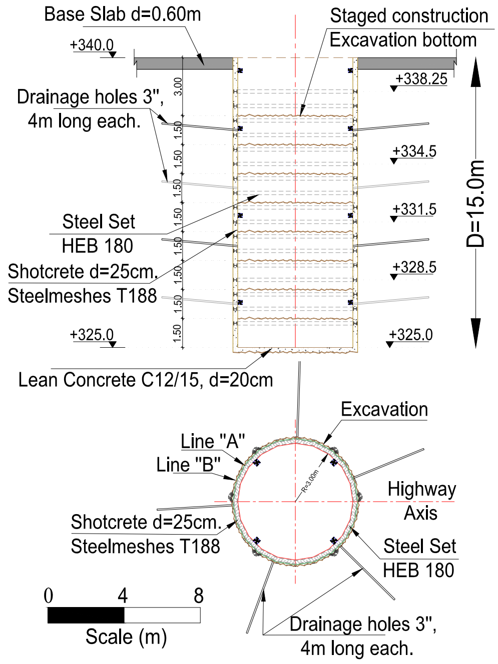

Figure 7. Typical cross-sections of caisson – Temporary support measures.

4.3 Southern abutment (A1)

A shallow stepped foundation was designed for abutment A1 due to the favourable geological conditions that were locally encountered (Fig. 8). The foundation itself, which was comparable in size to a 500m^2 building, was 28.4m in length and 17.0m wide. Along the longitudinal axis of the bridge the footing is founded on two levels

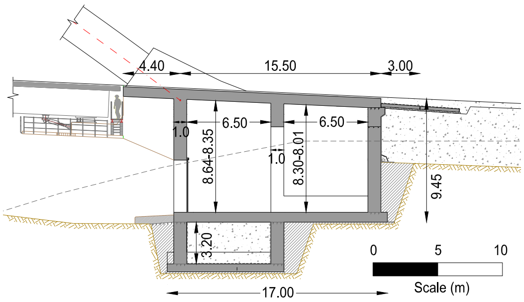

Figure 8. Typical cross-section of abutment A1.

having a vertical distance of 4.0m. The height of the foundation, which consists of thick walls and slabs / plates creating a very rigid reinforced concrete box, was approximately 15m.

At the area of the abutment the sandstone series of flysch dominates (i.e. medium to thick bedded sandstones with thin siltstone intercalations), classified as B to C [4]. The representative geotechnical parameters of Hoek – Brown failure criterion are GSI=35-40, σ_c=30MPa and m_i=12 taking into account the corrections due to the proportion of each lithological appearance [5]. The equivalent Mohr – Coulomb strength parameters for the rockmass are $\varphi'=45^o$ and c'=140kPa, whereas reduced strength parameters are assumed ($\varphi'=30^o$, c'=30kPa) along main discontinuity sets. The deformation modulus of the rockmass is estimated of the order of E=2.3GPa.

5 FOUNDATION DESIGN

Fundamental geotechnical principles and widely accepted methodologies were used for the calculation of the foundation adequacy of both abutments. Hence, for the pile group foundation of abutment A0 as well as for the shallow stepped foundation of abutment A1, DIN1054 specifications as well as other recognized methodologies were applied for bearing capacity and settlement estimation [6], [7], [8], [9], [10].

A particular challenge during design was the selection of the final geometry and the verification of foundation adequacy for the central pier, since very large loads are expected to be applied. The maximum loading components under static conditions were calculated of the order of 280MN vertically and 80MN horizontally, whereas the maximum moment was of the order of 565MNm. Under seismic actions, the maximum loading components were approximately 315MN and 120MN in the vertical and horizontal direction respectively, whereas the maximum design moment exceeded 1.2GNm.

Both analytical and numerical techniques were applied for the dimensioning of the foundation. Analytical solutions involved the comparison of the applied loads on the most heavily loaded caisson with caisson's adequacy under vertical, horizontal loads and moment. Axial load capacity incorporating both side and base resistance was calculated as proposed in [11], [12], [13]. The side shear resistance between the caissons and the rockmass was estimated considering acceptable approaches of literature as reported in [14]. The lateral load capacity assuming conservatively isolated caissons was calculated as in [15] and [16].

Due to the complex geometry involved, advanced numerical modelling was used for the verification of the capacity of the caissons – raft system and for the estimation of deformations. The 3D foundation package PLAXIS was used for all finite element analyses (Fig 9). Precise modeling was carried out including surface morphology, ground stratigraphy, foundation arrangement and all load cases, using representative geotechnical parameters for each ground layer and for all structural elements. The maximum displacements, under service conditions, were calculated less that 2.0cm and 0.6cm in the vertical and horizontal direction respectively.

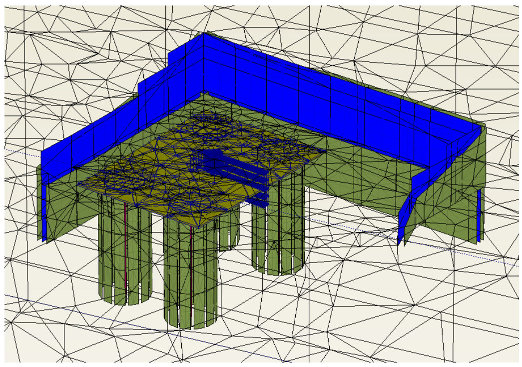
Figure 9. 3D finite elements model of the middle foundation.

6 CONSTRUCTION

Complex engineering projects demand continuous evaluation of the encountering ground conditions as construction progresses.

At the foundation locations of Tsakona bridge, all geotechnical and geological data collected during construction verified basic design assumptions regarding the ground, at an area of an extremely adverse geotechnical environment.

Detailed and continuous geological mapping was an essential part of the construction process for all the excavated surfaces. A typical mapping presenting encountered geological flysch types at the perimeter of the excavated surface of the 15m deep shaft "No.1" is presented in Fig. 10.

All the support measures and the retaining structures that were constructed proved adequate. However, due to the relatively open structure of the main discontinuity sets of the rockmass behind abutment A1, a grouting improvement scheme was considered necessary.

An instrumentation network has been installed and the whole area is under continuous monitoring. The landslide, eight years after its major reactivation, still undergoes creep movements, along the failure surface, with a rate of 2mm per month. Such peculiarity introduces one more engineering challenge that has to be dealt with for the erection of the steel arch bridge over the landslide.

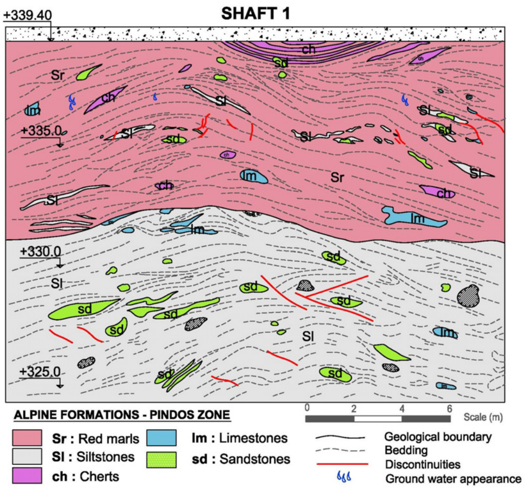

Figure 10. Engineering geological mapping of the excavated surface around the perimeter of shaft No.1 – Unfolded view.

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