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The stabilisation of a slope-viaduct system without closing traffic

La stabilisation d'un système pente-viaduc sans arrêter le trafic

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ABSTRACT A case history concerned with the stabilisation of a slope-viaduct system is reported. The two-carriageway viaduct was built 39 years ago on a rather gentle slope that did not show any sign of instability. In 2010 a landslide took place and damaged the viaduct. As the relocation of the motorway was deemed unpracticable, the alternative solution of stabilising the slope and repairing the viaduct was chosen. The owner asked to design remedial measures that did not require the closure of the viaduct. This was possible since the movements of the slided mass were slow and regular. The slope-viaduct system was stabilised by improving, firstly, the bearing conditions of the decks, then strengthening the existing foundations of the involved piers, and finally stabilising the slope by reducing the pore water pressure by means of subhorizontal drains. The remedial works were successfully carried out and resulted to be effective.

RÉSUMÉ Un exposé d'un cas concret concernant la stabilisation d'un système pente viaduc est rapporté. Le viaduc a été construit il ya 39 ans sur une pente douce qui n'a montré aucune indication de instabilité. En 2010, un glissement de terrain a eu lieu et a endommagé le viaduc. Comme le déplacement de l'autoroute a été jugé impraticable, la solution alternative de la stabilisation de la pente et la réparation du viaduc a été choisi. Le propriétaire a demandé de concevoir des mesures correctives qui ne nécessitent pas la fermeture du viaduc. Cela a été possible, car les mouvements de la masse glissé étaient lent et régulier. Le système pente-viaduc a été stabilisé en améliorant, d'une part, les conditions d'appui du pont, puis de renforcer les fondations existantes des piles concernées, et finalement de stabiliser la pente par la réduction de la pression de l'eau interstitielle à l'aide de drains subhorizontaux. Les travaux de consolidation ont été réalisés avec succès et ont été efficaces.

1 INTRODUCTION

Viaducts may happen to be built on slopes that, at the design stage, did not show signs of instability. If unexpected landslides involve and damage later such constructions, two courses of action open to the engineer, namely relocating or stabilising the slope-viaduct system. The latter option demands that: a) the movements of the sliding mass be slow and regular; b) the viaduct be capable of withstanding possible adverse effects of ground movements induced by the execution of the stabilising works; c) the remedial works be feasible, i.e. that it is possible to find a sequence of construction phases that do not jeopardise the slope stability and the viaduct; d) a decision be made on the possibility of carrying out safely the stabilising works without closing traffic; e) the remedial activities be closely monitored and contingency plans be prepared.

The latter solution was adopted for stabilising a stretch of a dual carriageway viaduct, which was involved in a landslide in March 2010. The geotechnical engineering aspects of this case history are dealt with in the present paper.

2 CARACTERISTICS OF THE VIADUCT AND FOUNDATIONS

The viaduct is located near Alcamo, in Sicily. It was built 39 years ago. Its planimetric layout is slightly curvilinear (Figure 1). The reinforced concrete piers are hollow and capped with a pulvino; each of them stems from a plinth which is founded on six 29m long piles (Figure 2). The deck consists of four prefabricated prestressed concrete simply supported beams, spanning 35 m, bearing on pulvinos. The beams are connected at the top by a reinforced concrete slab, poured in place. The height of piers ranges from 11.32m to 15.83m.

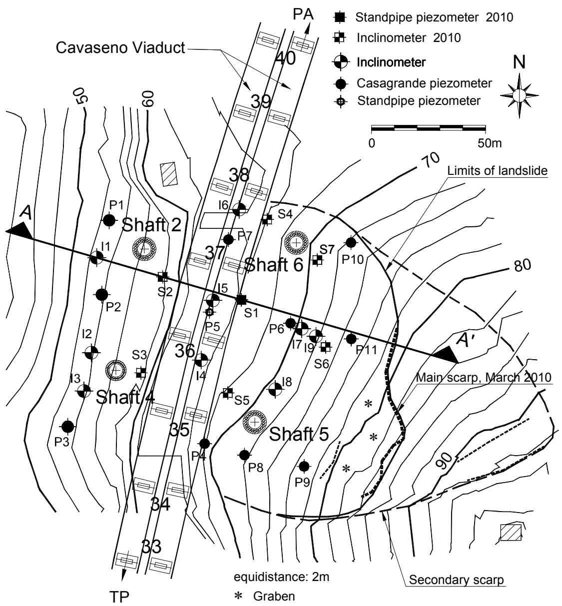

Figure 1. Schematic plan with location of viaduct, boreholes drilled soon after the occurrence of the landslide (S1-S7), other piezometers and inclinometers installed subsequently, and drainage shafts. Elevation: m above mean sea level.

3 INVESTIGATIONS AND MONITORING

3.1 *Survey of the slope*

The area of the slide and the plinths were surveyed a few days after the occurrence of the landslide. As a result, a contour map with the limits and the headscarp of the slide were drawn, Figure 1.

3.2 *Geotechnical investigations and instrumentation*

The landslide was investigated in March-April 2010 by means of seven vertical boreholes S1-S7 located as shown in Figure 1. Their depth below ground surface ranged from 25m up to 38m.

Undisturbed as well as disturbed samples representative of the soil natural water content were taken. Subsequently many other boreholes were drilled.

In 2010 an open standpipe piezometer was installed in borehole S1 while inclinometric tubes were placed in the remaining six ones (S2-S7). In 2012 and 2013, 10 Casagrande and one open pipe piezometers, 7 inclinometric tubes were installed. Unfortunately some of these instruments underwent damage or vandalism. The location of the surviving instruments is shown in Figure 1.

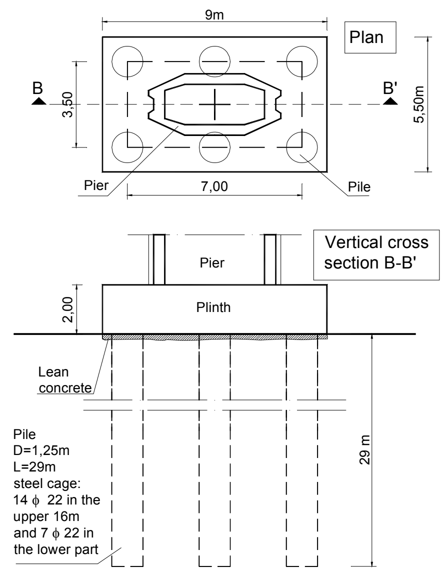

Figure 2. Schematic plan and vertical cross section B-B′ of the existing foundations of piers.

3.3 *Coring of existing piles and plinth-pile connection*

Two piles of the piers 36 and 37 have been cored up to depths of 7.70m and 6.50m respectively. In 2010 the pile-plinth connection of the above piers was uncovered on the downhill side. It was found that a layer of lean concrete, about 0.2m thick was interposed between the plinth and the pile head, see Figure 2, and that the piles were fissured just near the head; fissures extended all over the cross section of the pile; there the pile steel bars were partially corroded.

3.4 *Monitoring of the movements of piers and decks*

The monitoring of horizontal displacements of the decks and the pulvinos started in March 2010 and is still going on. Clinometers were placed on piers for detecting possible tilt especially during the implementation of remedial works.

4 GROUND CONDITIONS

The surface of the slope is regular and dips 8-11° towards West-North-West. The motorway runs approximately North-South, Figure 1.

Within the area relevant to the viaduct, the ground is made prevailingly of uncemented soils with the occurrence of lenses and pockets of highly permeable coarse-grained materials (cobbles mixed with sandy gravel and soft conglomerates). These soils belong to the "marly-clayey" facies of Terravecchia Formation that dates back to Tortonian-Lower Messinian (Basilone, 2012). The sediments of the marly-clayey facies were deposited by turbiditic currents that impressed to the soil mass a lenticular structure and interdigitations; consequently the ground conditions are variable and the structural pattern of the soil mass is complex.

The borehole logs and the visual inspection of the material excavated in the drainage shafts permitted to identify the following soil units, from top to bottom:

- TA Top soil consisting of remoulded brown sandy silt with organic matter; natural water content during the rainy periods reaches 40%; thickness: 0.50-2.00m.
- T1 Brown clayey sandy silt; natural water content 9-37%, generally decreasing with depth; thickness: 1.70-14.80m. Water bearing lenses and pockets of sandy gravel and cobbles and soft conglomerates are included within this unit. The thickness of the inclusions varies from 0.20 up to 2.60m.
- T2 Water bearing layer made of sand and gravel mixed with cobbles. This is a highly permeable transition layer separating units T1 and T3; it appears to be continuous; its thickness varies from 0.30 to 2.50m.
- T3 Very thick substratum comprising stiff grey clay, grey marls, and very dense sandy clayey silts. The silts are sometimes laminated and dissected by a three-dimensional network of white gypsum veins spaced from 15 to 25 cm. The silts have been uncovered at small depth in correspondence of the plinth of the uphill pier 35.

A schematic section of the slope is reported in Figure 3 from which the relationships between the soils and the foundations of the piers are apparent.

5 GEOTECHNICAL PROPERTIES OF SOILS

Laboratory data are summarized below.

Fine-grained soils of T1 unit: inactive and low-medium plasticity; specific weight γ_s=26-27 kN/m^3; γ_{sat}=18-20 kN/m^3; clay fraction: 22-42%; silt fraction: 45-68%; sand fraction: 7-18%. The shear strength parameters were determined by CD direct shear test: c'_p=2-3 kPa; φ'_p=26-27°; φ'_r =19-22°; coefficient of permeability k=10^{-6}-10^{-7} cm/s.

Clayey sandy silts of the T3 unit: inactive; low plasticity. γ_{sat}=22 kN/m^3; c'_p =10 kPa; φ'_p =36°; φ'_r =28°; k=10^{-8} cm/s.

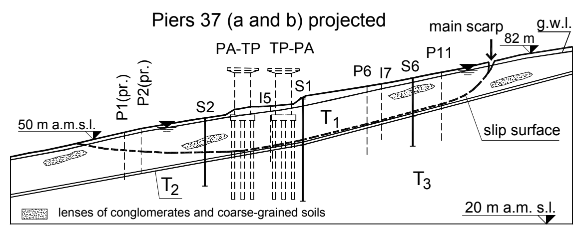

Figure 3. Schematic cross section A-A′. T_1: brown clayey sandy silt; T_2: water bearing layer made of sand and gravel mixed with cobbles; T_3: stiff grey clay, grey marls, and very dense sandy clayey silts. Top soil not shown.

6 GROUNDWATER

The groundwater condition was not known at the onset of the slide. Immediately after the slide was discovered by roadmen during a routinary check of the viaduct, ponds were found near the main scarp and elsewhere on the slope. Besides, the watertable was found at the base of the plinths of the involved piers; water was also noted in boreholes that started to be drilled two weeks after the event. In some instances water abruptly rised when the borehole went through conglomerate and sandy gravel lenses. Similar observations were made subsequently during the drilling of new piles. In the piezometer S1, installed in 2010, water stabilised at 1.80m below ground surface. Other piezometers, installed in the period 2012-2013, show that the groundwater table was located at depths ranging from 1.10 to 3.50 m. This information points out that the pore water pressures in the slope were certainly high, and that an artesian condition may exist, at least temporarily, in sandy gravel and conglomeratic lenses.

7 KINEMATIC CHARACTERISTICS OF THE SLIDE

The headscarp was clearly marked on the left side (as viewed from the crown) by a steep 1-1.5m high scarp; on the right side the scarp was 0.10-0.20m high (Figure 1). Near the main scarp the rows of vines were distorted indicating that the displacements of the slided mass had a prevailing downhill component towards the West and a much smaller transversal component towards the North. Only a small extent of the slide limits on both flanks was evident on the upper reach of the slide. The length and the width of the slide are 100m and 135m respectively. The slip surface was clearly pointed out by inclinometers S2 and S4 (cfr. Figure 1) at a depth of 12m and 18m respectively; the horizontal downhill displacements of the ground surface were 38 mm and 24mm 8 months after the installation of the inclinometers. The other inclinometers did not evidence any shear strain localisation. Based on these limited data the slip surface shown in Figure 3 was inferred for section A-A′; it passes through the fine grained soils of T1 unit and always overlies the top of T3 substratum. The slide involves a substantial part of the soil volume relevant to the piers' foundations. The estimated volume of the slide is 215×10^3 m^3. The maximum recorded velocity of the sliding mass did not exceed 5 mm/month; it became almost negligible six months after the onset of movement.

8 MOBILISED SHEAR STRENGTH AND CAUSES OF THE SLIDE

A backanalysis of the slide was carried out with reference to the sliding mechanism and to the pertinent estimated piezometric surface shown in Figure 3. 2D and drained conditions were considered. For $c'_m=0$ and a safety factor FS=1, a value of the mobilised angle of shear strength φ'_m = 24.7° was backcalculated by the Morgenstern & Price method. The influence of the foundations of piers was disregarded in the analyses. This value of φ'_m is close to the residual (φ'_r=19°-22°) as well as to the peak strength (φ'_p =26°-27°) of the fine-grained soils of the T1 unit; it is therefore almost undecidable if the slide is first-time or reactivated, since documented evidence of past failures is lacking. In any case, the main triggering factor appears to be the increase of the pore water pressure induced by prolonged and exceptionally intense rainfall during the six months preceding the slide. In fact, the cumulated rainfall from September 2009 to March 2010 reached the record value of 997.2 mm, which exceeded by 63% the average rainfall and by 153% the minimum rainfall in the previous decade. It is worth mentioning that this record cumulative rainfall was approached only two times during the period 1921-2010, namely in 1951-52 (805mm) and in 2004-2005 (884.8mm) when the slope might have been on the verge of failure.

9 REMEDIAL MEASURES

The above analysis pointed out that the slope-viaduct system needed interventions aiming at stabilising the slope, strengthening the piers' foundations and improving the bearings of the deck. It was soon evident that slope stabilising interventions only were not sufficient to return the viaduct to safe conditions as the existing piles were clearly unable to withstand vertical and horizontal actions. It was also clear that the slope stabilising works could not be carried out if the foundations and the superstructure were not strengthened beforehand. Moreover the owner asked to find a solution that did not require the closure of the motorway to traffic. The selected solution comprises the following phases. 1) The improvement of the bearings of the decks; 2) The strengthening of the foundations of the piers by encircling them by means of a "curtain" of concrete piles (reinforced by HEB240 steel beams) capped by a "ring" beam connected, in turn, to the existing plinth. The installation of the steel beams and the pouring of the concrete ought to take place immediately after drilling each pile; 3) The slope stabilisation by subhorizontal tubular drains installed radially from drainage shafts (Hutchinson, 1977). The latter were excavated after the realisation of a "curtain" of almost contiguous reinforced concrete piles, diam. ϕ 1.00 m. The installation of the reinforcing steel cage and the pouring of the concrete took place just after the completion of the perforation of each pile. The piles were capped by a rigid annular beam before the start of the excavation of the shaft. The interventions are schematically shown in figures 4-6.

The above succession of constructive phases as well as the close monitoring of the movements of the slope and of the structure were considered mandatory. Due to the complexity of the slope-viaduct system the remedial works were designed with a substantial degree of redundancy (Osterberg, 1989).

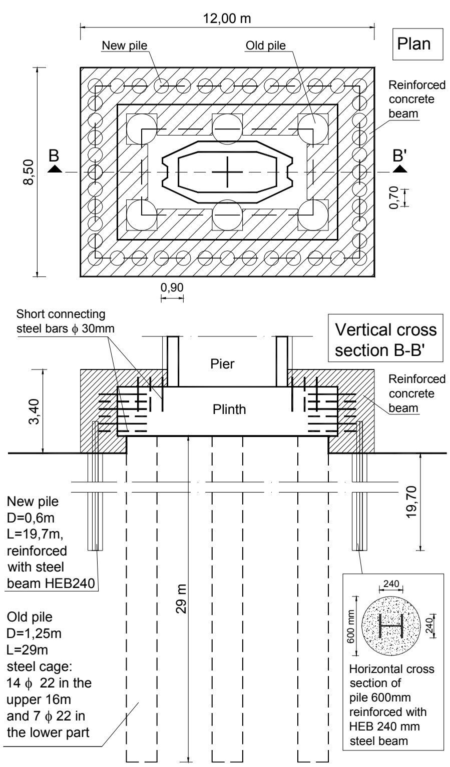

Figure 4. Measures for strengthening the existing foundations of the piers. Schematic plan and vertical cross section B-B′.

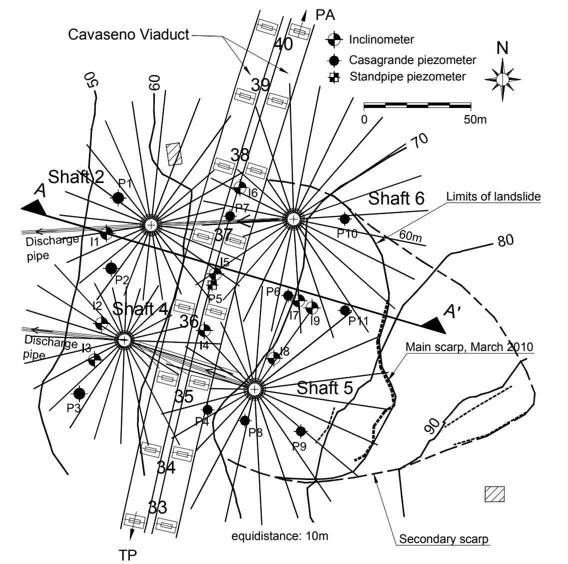

Figure 5. Layout of drainage shafts, radial subhorizontal drains, conduits connecting the shafts, and gravity discharge pipes.

10 EFFECTIVENESS OF CORRECTIVE MEASURES

The strengthening works of the foundations of piers 35 (a and b), 36 (a and b), 37 (a and b), 38 (a and b) were carried out in 2012-2013. During the construction of the ϕ 600mm piles around the existing foundations the pulvinos experienced cumulative horizontal displacements lower than 2 cm except on one occasion when the pouring of concrete for two piles was delayed some hours; as a consequence the lateral movement of the pulvino of the involved pier increased rapidly by about 1 cm with a corresponding tilt of about 1/1200; however the rate of increase slowed down as soon as the concrete was poured. The construction of the subhorizontal tubular drains started in September 2013 and was completed in June 2014. The effectiveness of the stabilising measures can be assessed on the base of both the decrease of pore water pressure, Δu_w, and the movements of the slope and of the viaduct. As yet horizontal displacements and settlements of the viaduct are negligible. Δu_w on or near the slip surface up to now ranges from 30 kPa to 101 kPa with a mean value of 59.5 kPa; typical trends of Δu_w vs time are shown in Figure 7. The decrease of pore water pressure brought about an increase of the safety factor by 28%. The flow rate of individual tubular drains was extremely variable and was higher when the drain passed through coarse materials. The maximum flow rate from the drainage system amounted to 105 litres per minute and was recorded in July 2014; it came prevailingly from drains installed in the North-East side of drainage shaft P2. At the end of September 2014 the flow rate almost halved.

11 CONCLUSIONS

The case history discussed in the present paper proves that the design philosophy adopted for the stabilisation of the slope-viaduct system was effective. Remedial works were successfully carried out without putting the viaduct out of service. Essential preconditions for the application of the approach were the regularity and the slowness of the sliding process and the capability of the structure to withstand the displacements originating from ground deformations induced by the execution of the remedial works, that have been carried out according to a proper sequence of the construction phases and under close monitoring.

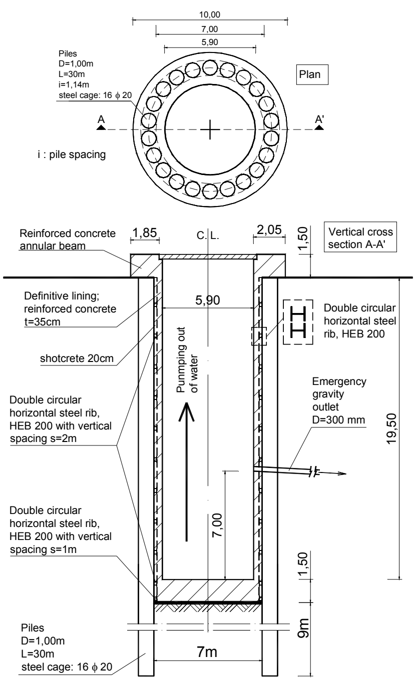

Figure 6. Schematic plan and vertical cross section A-A′ of drainage shafts.

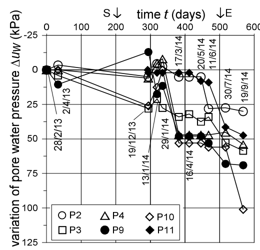

Figure 7. Typical trends of change of pore water pressure on or near the slip surface. Δu_w positive: reduction. The installation of the drainage system was started on September 20 2013 (arrow S) in shaft P6 and was concluded in July 15 2014 (shaft P4), arrow E.

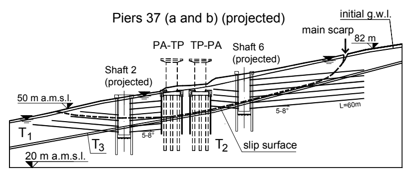

Figure 8. Schematic cross section A-A′ showing drainage shafts, subhorizontal drains, and old foundation piles and plinths encircled by a "curtain" of new piles.

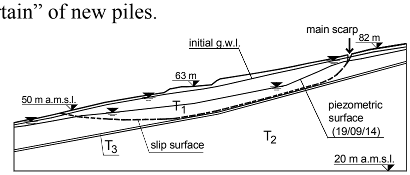

Figure 9. Schematic cross section A-A′ showing the slip surface used for stability calculations, the initial g.w.l. and the piezometric surface in September 2014.

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REFERENCES

Basilone, L. 2012. Litostratigrafia della Sicilia. *Dip. di Scienze della Terra e del Mare* – Università degli Studi di Palermo*,* 75-78.

Hutchinson, J.N. 1977. Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movements. *Bull. IAEG,* **16**, 131-155.

Osterberg, J.O. 1989. Necessary Redundancy in Geotechnical Engineering. *Journ. Geotech. Eng. ASCE,* **115**, 11, 1513-1531.