The "Strange" Case of the Scorciavacche Failure

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Abstract. Just after the completion of the upgrading works of the SS 121 national road connecting Palermo to Agrigento, in Sicily, the brand-new embankment linked to the northern abutment of the Scorciavacche viaduct suffered of two instabilities along the downhill side slope, causing the loss of two separate road segments of around 250 metres of total length. Early bird interpretation of the collapses ascribes the failures to a bearing capacity problem, consequence of the inadequacy of the embankment foundation. Surprisingly for the many actors on the scene, the subsequent forensic investigations have shown that the causes of the failure were different and not at all easily readable from the non-exhaustive observations of the initial scrutiny. Thorough geological and geotechnical investigations together with the monitoring of the site over a wide area made instead possible to discover clear signs of a quiescent instability which was bounded by a pre-existing failure surface that undoubtedly played a role in fixing the geometrical features of the collapses. The different picture of the scene that emerged after a careful geomorphological survey, a sound geotechnical investigation and a sufficiently long monitoring, convinced the technical experts of the defence that a wider scale model was needed to interpret this failure case that could not so simply be reconducted to a classical bearing capacity problem.

Keywords: Forensic geotechnical engineering · Slope instability · Embankment failure

1 Introduction

In the scientific literature, many papers analyse examples of failure of road embankments. According to Chen et al. (2004) and Briggs et al. (2017), fundamental causes of these failures can be ascribed to:

– bearing capacity problem due to poor geotechnical properties of the foundation soil;
– bad quality execution and insufficient workmanship supervision during construction causing poor geotechnical properties of the embankment soil.

In condition of scarce strength resources, the typical triggering events are snowmelt, heavy rainfall or earthquakes. Water infiltration due to snowmelt and rainfalls

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produce soil saturation and the raising of groundwater in the foundation soil likewise a decrease of the matrix suction within a fine-grained embankment body (Alonso 1999; Rahardjo et al. 2007). In both cases the increase of the pore water pressures reduces the soil shear strengths. Sometimes, with low-permeability embankment, during heavy rainfalls, the natural water flow may be obstructed so causing new actions either directly or as the consequence of the filling of tension cracks; in both cases the increase of the loading may bring the embankment to collapse (Shibuya et al. 2004). On the other hand, earthquakes determine unfavourable inertial forces on soil mass.

When failure happens in the presence of a variety of predisposing conditions, many interpretative assumptions may be realistic, so that a thorough investigation over a wide range of possibilities has to be set up to get to the most plausible explanation of the phenomenon.

The present paper moves from the two failure episodes occurred to a newly built segment of a high road embankment, along the SS 121 national road connecting Palermo to Agrigento, close to the town of Villafrati, in Sicily. Soon after the construction of the embankment, close to the North abutment of the new Scorciavacche viaduct, the road platform was damaged by the appearance of two instabilities events along the downhill side slope of the embankment. Over these episodes, there has been much public outcry since the road segment was opened to traffic just few days before the appearance of signs of instability that made obligatory its immediate closure. A preliminary investigation was initiated by the Public Prosecutor ("*Pubblico Ministero*") with his panel of experts questioned all the possible professional figures involved in the events, in particular the Designer ("*Progettista*"), the Engineer (or Supervisor – "*Direttore dei Lavori*"), the Project Manager ("*Direttore di cantiere*") and some technical figures from the Client (or Employer - "*Stazione Appaltante*") that in this case was the Italian National Road Public Authority. The definitions of professional figures are in according to the most common international families of contract templates in the field of engineering and construction: FIDIC – International Federation of Consulting Engineers and NEC- New Engineering Contract.

The very ample and long-lasting investigation set for the controversy is herewith presented and discussed, not only to show some interesting technical aspects of this particular case history, but also to highlight the risks that in forensic engineering can derive from possible misinterpretations and hasty judgement, often the consequence of the enormous resonance that such events may have in the public opinion. In the following, the case of the Scorciavacche failure is presented under the perspectives of the technical experts and of the lawyer appointed by the Designers.

2 Sequence of the Events

Along the old road segment crossing the Scorciavacche district in the province of Palermo, due to the intense aging suffered by the concrete structural elements, the "old Scorciavacche 2 viaduct" (see Fig. 1) needed to be completely substituted with a new construction, partly with a short viaduct (180 m, named "new Scorciavacche 2 viaduct") and partly with a long embankment (380 m), attached to the north-side abutment of the new viaduct (Fig. 1).

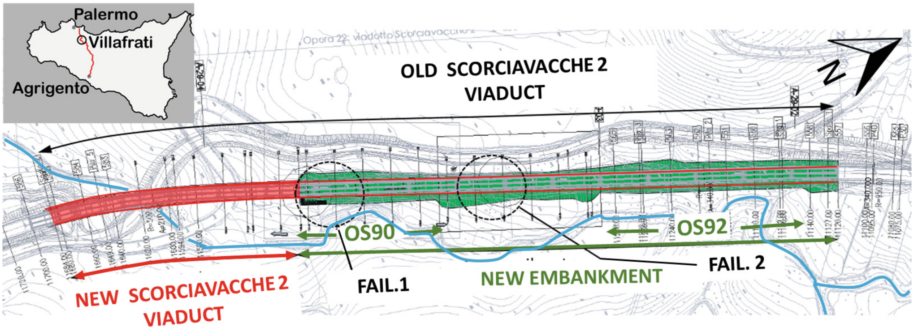

Fig. 1. Plan view of the embankment and positions of the two failure events

Due to the presence of a tiny stream, the cross section of the embankment at its toe had to be profiled downhill by a reinforced earth wall. More precisely, starting from the abutment, the new embankment was bounded by a very steep reinforced earth wall (with a dip of 70°) up to 100 m chainage (named OS90) and from chainage 200 m to 355 m (named OS92). Where the stream was sufficiently far, the embankment was simply shaped at 2:3 slope.

The construction works of the new road segment started in September 2014: the embankment was raised in stages, with layers of compacted crushed limestone; at the end of November 2014 the road pavement was completed. The road was opened to traffic on 23 December 2014. After six days, the first pavement sags were noticed in the area of OS90 and the damage worsened so rapidly that the following day, that is the 30 December, the road had to be closed. Three days later an evident scarp (about 1 m high) appeared on the pavement together with a bulge at the toe of the reinforced earth. The Failure 1 phenomenon, occurred where the reinforced earth reached its maximum height (about 9 m) and it was confined southward by the viaduct abutment.

One month later, signs of a new failure phenomenon appeared on the pavement at chainage 110 m towards north; such new event fully developed in three days, so becoming the Failure 2. Figure 2 shows the main kinematic features of the two events on an aerial image of the area, that can be summarised as follows:

Failure 1:

- appearance of a scarp trench at the West side of the failure suggesting that the prevailing soil movements were towards East that is towards the stream at the toe of the embankment;
- planar rigid rotation of the reinforced earth close to the viaduct abutment;
- appearance of a traction fissure on the pavement, some tens of metres North side of the main failure, indicating a slight movement towards South-East of the embankment.

Failure 2:

- scarp at the crest of the mechanism very close to the upper limit of the embankment slope, thus suggesting that the failure surface is very shallow;

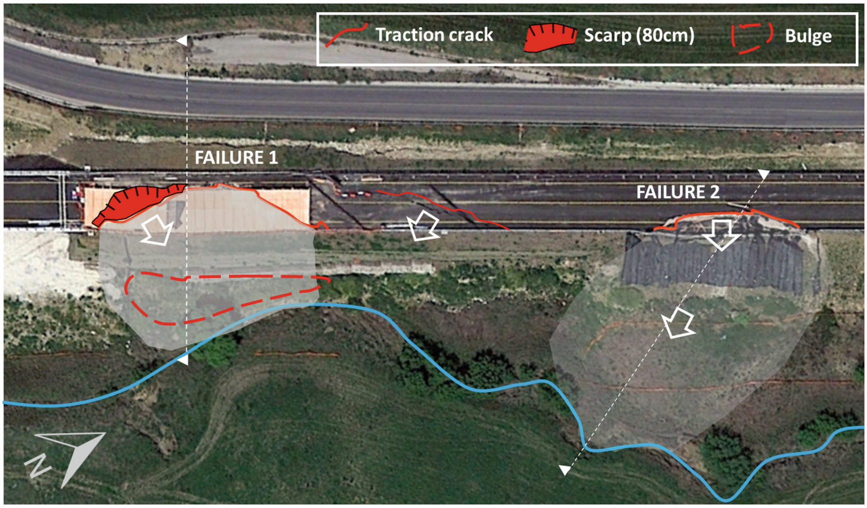

Fig. 2. Schematics of the instabilities drawn over the aerial image of the site.

– morphology of the ground, compatible with a movement towards South-East.

The monthly rainfalls recorded at the Ciminnà weather station, close to the area of interest, in the time interval from construction up to the occurrence of the second failure, indicate that a relatively severe raining season with intense rainfalls and snowfalls preceded the two failures.

3 The Geotechnical Investigation During the Pre-trial Phase

Soon after the closure of the road, a preliminary investigation was initiated by the Public Prosecutor's Office to investigate over the causes of the accident. To freeze the evidences of the site condition subjected to modification with time, the Prosecutor requested the judge for preliminary investigations ("*Giudice delle indagini preliminari – GIP*") to admit a special enquiry procedure for recording evidences ("*Incidente probatorio*") during the pre-trial phase. The judge admitted this procedure identifying some Court-appointed experts ("*Periti*") that with the contribution of the experts appointed by the suspects ("*Consulenti tecnici degli indagati - CTP*") and by the Prosecutor ("*Consulenti tecnici del Pubblico Ministero - CT*") planned a comprehensive on-site recognition and a ground investigation. As imposed by the procedures of the Penal Code the main lines of the investigations were selected by the Court-appointed experts; their intention was to verify three possible causes of the failures:

(a) insufficient bearing capacity due to poor geotechnical characteristics of the foundation soil;
(b) poor mechanical/hydraulic characteristics of the embankment;
(c) unforeseen hydraulic boundary conditions.

The geotechnical investigation plans consisted of several boreholes with undisturbed sampling, cone penetration tests (CPTu), exploratory trenches, plate load tests and geophysical surveys, as showed in Fig. 3. A monitoring of the site was also implemented with inclinometers, Casagrande piezometers and topographic benchmarks. Moreover, data from a series of topographic targets placed at the end of November 2014, soon after the embankment completion, on the reinforced earth close to the viaduct abutment were available during two time periods: one preceding and one following the first failure event (Fig. 3). As it will be clear in the following, the pre-failure topographic measurements resulted particularly significant to discover the key mechanism for Failure 1.

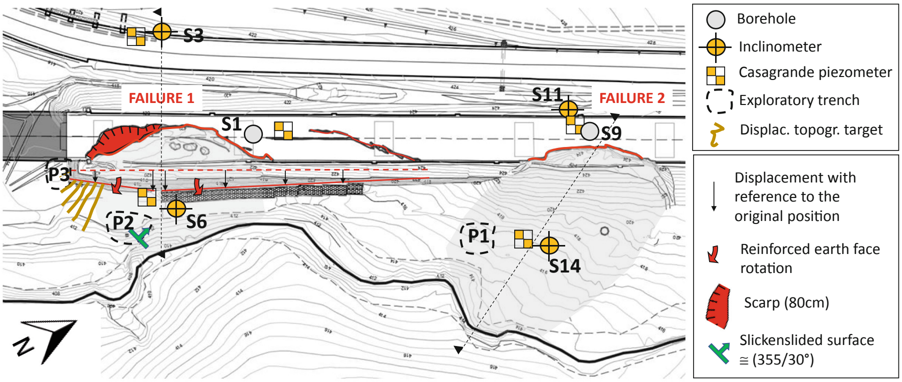

Fig. 3. Arrangement of the field investigation, displacement vectors and schematics of the two mechanisms

Soil stratigraphy is characterized by the presence of a base formation (FB) made of a stiff, sometimes scaly, clay and marl, with rare inclusions of conglomerates; the base formation is overlaid by a thin (0.5 ÷ 2.5 m) layer of clay, consequence of the alteration of the base material (FBa). Above the altered formation a deposit of dark and yellow sandy clayey silts (LSA) can be found, with variable thickness, from 3.7 m close to the stream up to 15 m uphill.

A schematic picture of the soil profile with groundwater heads from piezometers and deformation profiles from inclinometers are presented in Fig. 4 for the two failures. It is worth to note that the critical surface for both the collapses lies near the contact between the silty-sand deposit and the base formation.

Laboratory tests were run on undisturbed samples either Shelby or block type from the base formation (FB) when close to the slickensided surface discovered inside the P2 exploratory trench.

The grain size distribution of the soil from LSA deposit presents a gravel fractions up to 20% in weight and a fine content ranging from 35 to 65%. The Formation (FB and FBa) presents a negligible sand fraction (less than 5%) and fine content ranging from 75 to 96% with clay fraction always greater than 50%. Void index, natural water content (w_n), liquid limit (w_L) and plasticity index (PI) results as it follows:

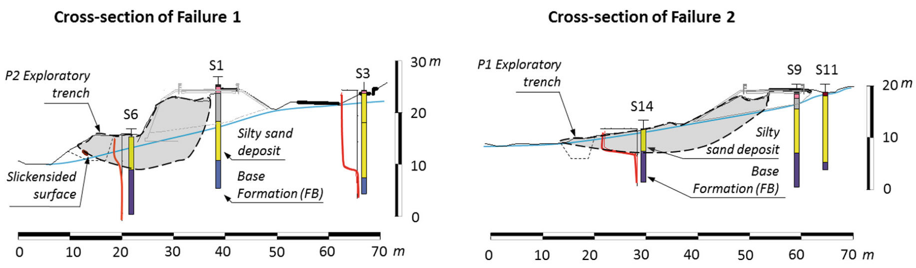

Fig. 4. Cross section of Failure 1 (left) and Failure 2 (right).

$$\text{LAS} \quad e_0 = 0,45 \div 0,70; \;\; w_n = 8 \div 20\%; \;\; w_L = 27 \div 50; \;\; PI = 12 \div 30;$$
$$\text{FB} \quad e_0 = 0,30 \div 0,58; \;\; w_n = 11 \div 22\%; \;\; w_L = 44 \div 55; \;\; PI = 31 \div 35.$$

A band with higher natural water content was detected at the toe of the embankment across the contact between LSA and FBa where sliding is most probably localized. The fine fraction for both soils, LSA and FB, is of medium to low activity (IA = 0.5–0.1). To determine the effective strength parameters, direct shear and triaxial compression tests were carried out on undisturbed samples. Figure 5 shows the failure envelopes for the samples of LSA and FBa recovered during pre-trial investigation. The unit peak shear resistance shown by LSA samples appears sparse with samples from LSA-FB interface giving the lower values; it is however possible to draw upper and lower limit envelopes and observe that the design envelope falls in between those limits. The observed range of shear strength envelopes fit well with those obtained for design: the higher number of failure points close the lower envelope is justified because most of the samples were taken close to the slipping zone or to observed discontinuities.

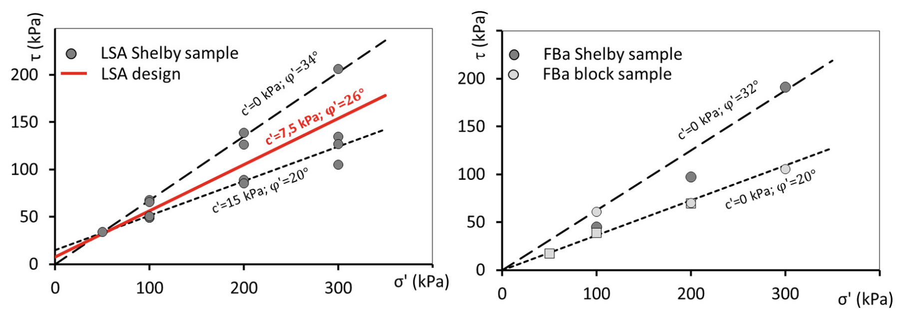

Fig. 5. Effective strength envelopes for LSA and FBa samples recovered during pre-trial investigation

Only few samples from the Base Formation were available for direct shear testing, due to the limited interest for the investigated problem. Samples, belonging to FBa, gave ambiguous results: despite high stiffness and low void index resulting from the tests, shear strength parameters resulted closer to those expected for critical state or residual conditions, especially for samples recovered from the slickensided surface in P2. A plausible interpretation for this mechanical behaviour refers to the presence of pre-existing fissures in the samples: such planes may influence the strength according to their prevalent orientation, as documented in Calabresi e Manfredini (1973) for stiff jointed clays, in the same phenomenological framework of structurally complex formations.

A review of the stability analysis of the earthwork in relation to the laboratory test results, suggested that the average strength characteristics of LSA were adequate to guarantee the embankment safety. With reference to the embankment body, plate load tests certified strength characteristics higher than expected in the design phase and a pervious grain arrangement was demonstrated through the Matsuo-Akai field test (Matsuo et al. 1953).

In conclusion, in contrast with what it was expected in the early stages of the investigation, the laboratory test results didn't allow to identify a general lack of geotechnical resources to be pointed at as the main suspect for the observed failures.

More interesting results were given by the monitoring activity. A map of the area with the polar representation of the magnified deformation measured by the inclinometers is shown in Fig. 6. Despite a relatively short monitoring period (4 to 5 months), which started ten months after the failure, three of the four inclinometers, S3, S6 and S14, gave a clear indication of the existence of a large movement, oriented in the direction 150–170°N, that included both the observed failures. Particularly meaningful were the readings of the inclinometer S3, located far away from the observed phenomena, showing clear signs of a sliding 16 m below the G.L. at the interface between LSA deposit and the base formation (FB).

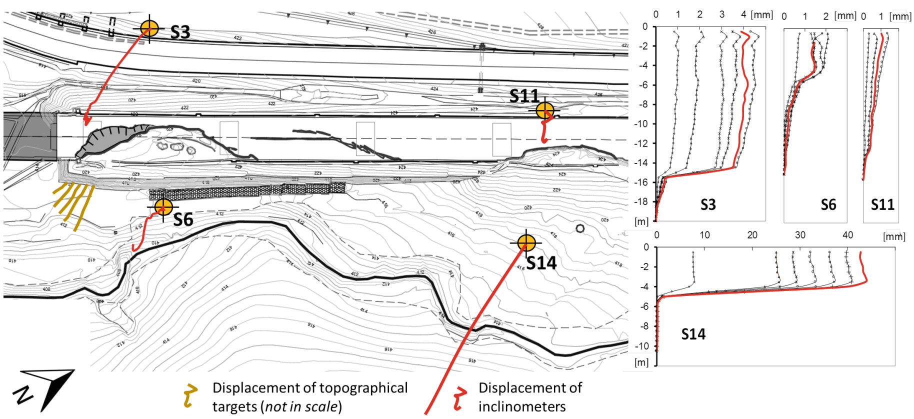

Fig. 6. Monitoring of the displacements. Displacement vectors (left) and vertical profiles (right)

This finding greatly enlarged the scale of the problem and was interpreted as the indication of the presence of a specific weak layer, that is of a failure plane, along which a slide may have developed. From now on, the investigation perspectives were changed towards new interpretations of the observed mechanisms.

4 Technical Interpretations

Results from the monitoring allowed to recognize the existence of a large mass movement with direction South-East in the site under investigation. This mass movement, possibly an old reactivated landslide, has at least interfered with, but very likely was the necessary precondition for the observed failure events. In facts this large movement easily explains the Failure 2 mechanism, whose kinematics is consistent with the geometrical features of the landslide. On the other hand, interpretation of Failure 1 is not straightforward, being the paroxysmal movement directed not accordingly to the landslide movement but orthogonally, towards the stream at the toe of the embankment, as it is demonstrated by the shape of the scarp at the top of the mechanism, that is close to the junction with the abutment. What makes awkward the interpretation of Failure 1, is the rotation of the azimuth of the displacement vector towards South-East which was observed after failure. Such direction of the displacements is coherent with the pattern of fissures appeared on the road pavement and above all with the displacements of the targets placed on the earth-reinforced wall that gave signs of an incipient failure accordingly to the kinematics features of the old landslide. All the above evidences suggest that Failure 1 was favoured by the pre-existence of a weak surface associated with the general hillside movement, but apparent movements were initially different, as they were conditioned by the local morphology, that is the presence of the stream at the toe of the embankment.

According to these findings, a full revision of the geological and geotechnical model was proposed including the presence of a pre-existing failure surface between the shallow deposit and base formation. In this new framework, back analysis of the two collapses demonstrated that failures could be justified only if the failure mechanism, at least partially, lays over a pre-existing failure surface for which residual values of the shear strength parameters hold. In facts, failures were made possible by the large increase of the loads over the ground and triggered by the very intense rainfalls and the consequent rising of groundwater pressures.

It is well known that weak layers in structured clayey formations are always difficult to identify through ordinary geological surveys (Ruggeri et al. 2016), but their influence on the stability conditions of slopes, where new infrastructures are constructed, can be very significant because of the very low shear strength attained (Calabresi e Manfredini 1973). Such weak surfaces can be part of the network of discontinuities affecting the natural soil formations (Alonso and Gens 2006a, b; Scarpelli et al. 2013; Segato et al. 2015; Ruggeri et al. 2019) or the consequence of old slope instabilities as in the present case.

5 Legal Aspects

The present case study resulted particularly significant because it shows how it is important for an optimal defence of the suspects to keep and guarantee a close relationship between technical findings and legal strategies. Very honestly, a reliable reconstruction of the facts could be achieved in this case only because an open-minded technical investigation was executed as part of the judicial inquiry. What favours such way of proceeding is the recent trend of the Supreme Court of Cassation that imposes to the judge to justify his decisions specifically when are dissenting from the opinion of his technical experts. Differently, the judge in the past was free to consider or not consider the conclusions of his experts, in agreement with the Latin aphorism “*iudex peritus peritorum*” (*i.e.* judge is the expert above the experts). For these reasons the technical assessment should cover all the possible aspects of the problem to give the judge elements and suggestions that may end up to be favourable for the defence. It should be remarked that the Italian Law allows, and even obliges, the defence lawyer to use only the arguments that are favourable for his part and it is therefore very important to let the technical investigations moving freely in every direction to establish the best explanation of the facts, so allowing a progressive adjustment of the defensive strategy to optimal, without excessive risks when adverse findings result from the investigations.

6 Conclusions

In the presented case history only an exhaustive site investigation made possible to establish a convincing interpretation of the occurred failures. Decisive for reaching this result was the monitoring from one inclinometer (namely S3) located well out of the embankment significant soil volume. Its unforeseen outcomes have completely changed the initial picture of the scene suggesting that the somehow “strange” and to many investigators unexplained failure phenomena could be at the end justified by the presence of an existing deep failure surface of an old landslide which was partially included in the new formed embankment failure mechanism. In other terms, a critical analysis of the outcomes from field investigations, soil characterization and monitoring brought to a complete review of the geological and geotechnical models initially assumed. What at first glance had to be interpreted as a bearing capacity problem for the embankment foundation, that is clearly a flaw in the geotechnical design, turned out to be the more complex effect of the existence of unforeseen adverse geological conditions. In this respect is appropriate to recall the phrase of Conan Doyle (1890): “*once you have eliminated the impossible, whatever remains, however improbable, must be the truth*”. Thanks to the good work of the Prosecutor’s experts (*CT*) and to the open-minded investigation allowed by the Court-appointed experts, the strategy opposed by defence lawyer resulted successful in convincing the Prosecutor to drop off the charges from a large part of suspects and specifically from Designers.

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