Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0825

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

Failure of a massive geosynthetic-reinforced clay dyke for a waste disposal plant: investigation of the causes

Rupture d'un barrage en argile renforcé par géotextile pour une installation d'élimination des déchets: étude des causes

P. Ruggeri
Università Politecnica delle Marche, Ancona, Italy

V.M.E. Fruzzetti, G. Scarpelli
Università Politecnica delle Marche, Ancona, Italy

ABSTRACT: To increase the capacity of a waste disposal plant located in a hilly site in central Italy, a new 25 m high barrage has been built to bound a valley and create a new waste storage over the upstream area. To minimize environmental risks, a geosynthetic-reinforced clay dyke has been designed for the plant. Just after the completion of the works and before entering into operation, clear signs of instability appeared at the upstream side of the dyke. To prevent the growth of such mechanism the dyke was immediately re-profiled by lowering the crest. A thorough ground investigations and monitoring were carried out to understand the causes of the instability; three main causes have been identified: unfavorable local geological conditions; inhomogeneity of the foundation soil; development of high pore pressures in the dam and in the subsoil. This case history turned out to be again an example of the importance of details for design and construction of geotechnical works particularly if of great impact for the environment.

RÉSUMÉ: Dans le cadre d'un projet d'élargissement d'un site de stockage de déchets en Italie, une nouvelle barrière de 25 m de haut a été réalisée. Afin de respecter le cadre réglementaire sur les risques environnementaux ce barrage a été conçu en argile renforcé par géotextiles. De manière inattendue, après l'achèvement des travaux, des signes évidents d'instabilité sont apparus d'un côté de la barrière, indiquant l'apparition d'une rupture de rotation globale. Une enquête approfondie du terrain a été effectuée afin de clarifier les raisons à l'origine de cette instabilité. Trois causes principales ont été examinées: 1) conditions géologiques locales défavorables; 2) manque de substratum sous une partie de la fondation; 3) développement d'un excès de pression interstitielle. Dans le document, chacun de ce scénario sera examiné à la lumière des résultats des enquêtes géotechniques. Ce retour d'expérience s'est avéré être à nouveau un exemple de l'importance des détails dans la conception et la construction des travaux géotechniques, dans un contexte à forts enjeux environnementaux.

Keywords: Earthworks; Waste barriers; Undrained failure; Remedial measures

1 INTRODUCTION

An old urban solid waste disposal plant has been active until recent years on the left side of a little valley, located in the hilly territory of a municipality in the central Italy. Approaching the depletion of the waste storage volume, a project to expand the plant was implemented. The project provided the construction of a new dyke that, crossing the valley, bounds a larger area where a new volume for the storage of waste can be allocated. The dyke is obtained with an embankment, 25 m high and 240 m long, made of clay excavated from the site, compacted and reinforced with structural geosynthetic grids. The design was somehow determined by the following conditions:

- the hydraulic confinement of the stored wastes can be directly provided by the clay;
- the reuse of soil from excavations minimizes costs;
- the use of structural geosynthetic may enhance the strength of the compacted soil as needed for the prospected geometry of the dyke.

After the completion of the work and just before the plant could come into operation, clear signs of instability appeared along the upstream side of the dyke. The upward extension of the phenomenon to the crest of the slope and its evolution, indeed rather fast, suggested that a global failure mechanism was occurring. Such obviously unexpected event at glance appeared unjustified in the light of the large number of geogrids used to reinforce the earth embankment. Movements slowed down only after the dyke crest could be lowered of about 7 m and a berm at the dyke toe was built. Nevertheless, according to inclinometer readings, the soil mass still exhibits, even today, a continuous deformation process. A geotechnical investigation, with boreholes, lab-testing on undisturbed soil samples and cone penetration tests (CPT), was carried out to investigate the collapse. A large number of inclinometers and piezometers were installed to monitor the instability process. Such investigations indicated that failure might have been determined by multiple and concurrent causes. Most significant to interpret the instability process is to consider the use of clayey soil as construction material and some missing but crucial details of the construction procedure.

2 GENERAL SITE CONDITION

The area of interest is the upstream part of a little valley in a countryside, few kilometers away of the Adriatic coast in Central Italy; local morphology is characterized by the gentle hills emerging between the Apennine chain and the Adriatic Sea. The valley is asymmetric, signed by the characteristic river channel that it is shifted towards the left side (facing downstream). The area belongs to the Neogene-Quaternary Periadriatic Basin that developed during the late phases of the Apennine orogeny as the fore deep migrated eastward. The entire Periadriatic Basin was submerged during most of the Pliocene and Pleistocene when it was the bottom boundary for the sediments from of a long phase of marine deposition. The resulting soil deposit so called formation of “Argille Azzurre”, exhibits the joint effects of tectonics and of sea-level changes. Such formation is the bedrock of the area; it is mainly fine grained and strongly overconsolidated as the consequence of its geological history. The content of calcium carbonate make this formation very stiff and, in some cases, close to a Marl, a weak rock with a typical mechanical behavior at failure (Scarpelli et al, 2003). The recent tectonic evolution of the area produced a mild immersion of the bedrock strata towards the Nord-East direction. The bedrock is not outcropping, but it is covered by eluvial and colluvial deposits originated from the weathering of the original formation. Such deposits are heterogeneous, typically fine grained, with poor mechanical properties. The thickness of the soil cover changes from 2-3 m along the sides to over 8 m towards the axis of the valley.

In the area, the groundwater level depends on the climate and the seasonal fluctuation of the rains. No water springs are presents. The bedrock has very low permeability so that any water flow is physically confined keeping into the shallow deposit. The water table raises close to the ground surface during the winter and down to deposit-bedrock interface during the dry season. For this reason the river channel in the valley is dry for most of the year.

The absence of any significant groundwater flow, the geological natural barrier offered by the bedrock and the morphology of the valley are such to make the site ideal for the placement of a waste landfill.

3 THE DYKE

The design of a waste disposal plant shall follow a severe regulatory framework stated to minimize environmental risks. The large part of rules address the hydraulic separation of the leachate produced by the waste degradation from the natural groundwater and tend to minimize gas emissions. To this aim, in the project, a multi-barrier insulation system was employed and a sanitary landfill with waste sprayed in layers and then compacted and covered with soil, was planned. From the environmental point of view, the project followed the best practice indicated in Technical Recommendations (e.g German Geotechnical Society, 1993) and in the scientific literature on the subject.

Besides environmental design, the main object of this paper is to examine the design of the large dyke that, enclosing the upstream area of the valley, produces a new storage volume for urban wastes. The earth embankment rises up to 23 m above the original ground profile. Including the 3 to 5 m deep excavation to reach the bedrock, its maximum height is about 25-26 m; a road along the crest completed the plant. Figure 1 shows the typical cross-section. The slope angle of the upstream face is 45°, while downstream the slope is 40°. Two berms are placed downstream, both 3 m wide. At the end of the construction, with the reshaping of the ground profile around the embankment, the maximum emerging height was about 23.5 m. In the zone of maximum height, the refill after excavation was made of soil improved with lime. The body of the dyke is made of a number of strata of compacted clay, 0.6 m thick, as said enclosed between layers of strong geogrids.

Geogrid reinforcements play an important role for stability of the embankment, as it could never have the adopted steep sides without the geogrids. The designer selected grids knitted from high-tenacity multifilament polyester yarns (PET), coated with black PVC to provide a good resistance to UV radiation and durability. The design is according to the British Standard BS8006-2010. The characteristic strength of the geogrid varies, top to bottom, between 35 kN/m and 400 kN/m.

The International Standards (e.g. USBR: Design Standard n.13: embankment dams, 2012) all highlights how the stability of an earth fill in clay must rely on the effectiveness of the compaction of the soil. In the present case the tender specifications required that cohesive fill materials have to be compacted to at least the 90% of the maximum modified Proctor dry density with moisture content within 2% of optimum. Maximum allowed thickness of each level in the stratum to be compacted was of 0.2 m.

4 THE FAILURE

After the end of construction signs of sufferance of the embankment began to appear. First the unexpected rupture of a drainage pipe crossing the dyke, then the opening of a crack at crest on the downstream face and finally the settlement of an entire segment of the top road were occurring in sequence. The phenomenon became evidently a “mass movement” when a bulge of the ground appeared at the base of the upstream side. Therefore contingency measures were promptly activated: the lowering of the dyke and a ground fill over the bulge. Finally, at the removal of 7 m of soil from the crest for 60 m along the embankment, the paroxysmal movement came to a stop. Figure 2 shows the cross section of the dyke with a sketch of the mobilized mass.

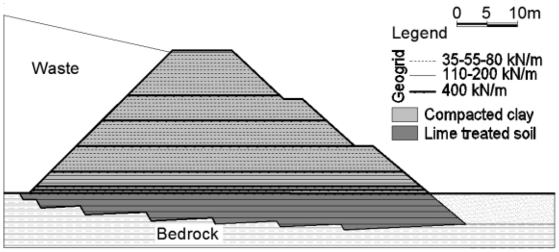

Figure 1. Typical cross section of the dyke

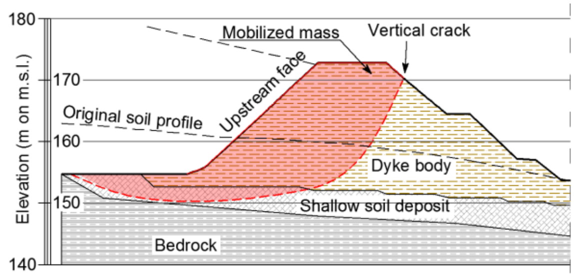

Figure 2. Trace of the observed failure surface

5 RESULTS OF INVESTIGATIONS

The layout of the site with locations of boreholes and probes as installed after the failure event is shown in Figure 3 together with the vertical cross section X-X′ along the dyke crest.

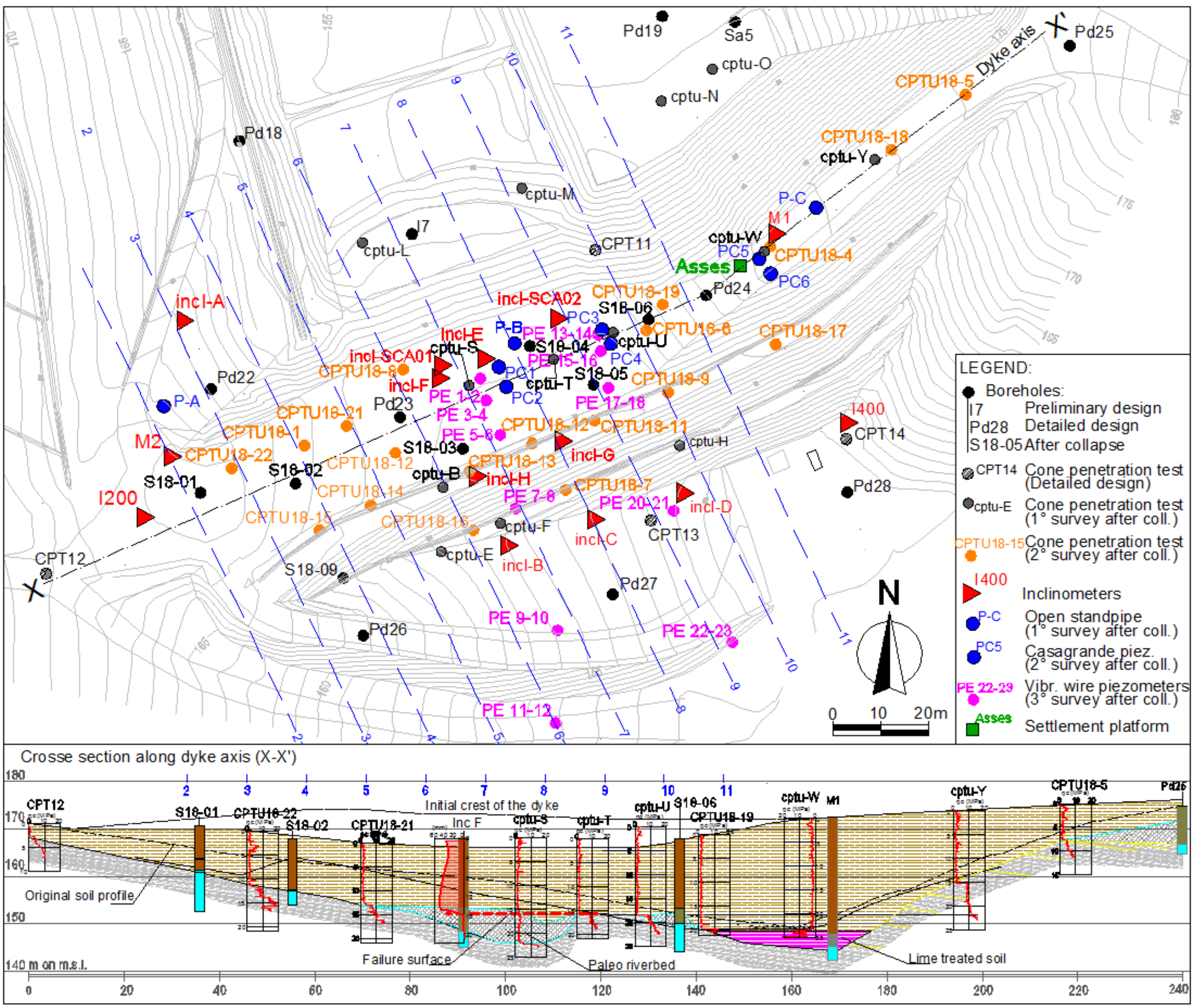

Figure 3. Layout of the site, investigations and resulting geotechnical model along cross section X-X′

The dyke body and the boundaries of the soft soil, of the lime treatment and of the bedrock can be distinguished. Figure 4 shows contours of the altitude of the interface between bedrock and the soft surficial soil.

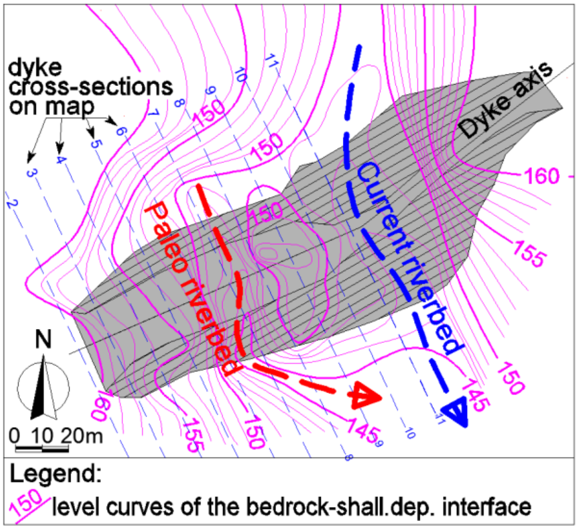

Figure 4. Level contours for the interface between bedrock and the shallow deposits

It is worth noting the displacement of the riverbed towards East and the consequent increase of the thickness of the soft soil filling. Such a displacement of the riverbed is linked to the immersion towards North-East of the bedrock and is the cause of the better geotechnical conditions of the left flank of the valley with respect to the right one. However the presence of a rather thick deposit of soft soil filling the paleo riverbed was not properly considered by designers and this posed the premises to the localization of a failure surface right in this zone of the dyke.

5.1 Kinematic of the failure

The kinematic of the failure emerged clear from both inclinometer profiles and morphological evidences even if the non-homogeneity induced in the embankment by the geogrids influenced somehow the resulting geometry.

Displacements were substantially vertical at the crest. The failure surface appears not regular, being the boundary of a block compound phenomenon, and is formed by a steeper segment in the upper part and a sub-horizontal one in the lower portion. The described geometry of the failure is however a simplification of reality, as the observed displacements at the upper part of the dyke show some rotational components that suggest compatibility with a log-spiral failure surface.

Figure 5 shows the cross section n.7 of the dyke (see Figure 3 for the trace of the sections), just where the instability occurred, with the relevant inclinometer profiles being shown. Note that inclinometer readings refer to the period after the paroxysmal event. The profiles show quite clearly that deformations are localized at the contact between the base of the dyke and the top of the shallow deposit. Moving upstream the failure surface deepens and localizes at the contact between the shallow deposit and the bedrock. Such geometry of the failure is very likely conditioned by the occurrence of residual strength conditions at that interface.

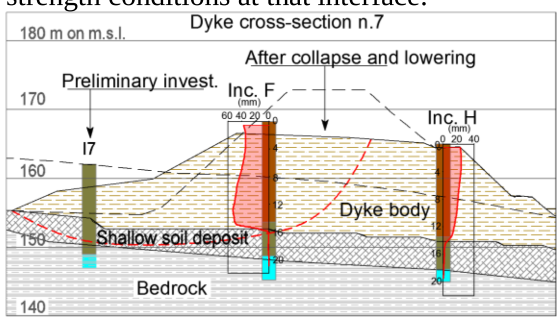

Figure 5. After collapse inclinometer profiles and resulting geometry of the failure surface

5.2 Geotechnical characterization

The geotechnical characterization of the soil deposits was obtained both through site, i.e. CPT's, and laboratory testing of undisturbed samples. Such testing addressed the dyke constituent materials and the foundation soil deposits, the shallow layer and the bedrock. Figure 6a shows the particle size distributions for all soils; a uniform composition made of 50-60% of silt and 40-50% of clay resulted from the tests. Atterberg limits indicate that the soil of the embankment and the soil from the shallow deposit, as expected, have the same index properties: in both cases the clay classifies of high activity (CH). On the other hand, activity of the clay from the bedrock is low (CL). In Figure 6b index properties are compared with values of the natural water content: water content of soil samples taken from the dyke is close to the respective values of plastic limit while, for bedrock samples, water content is lower than the plastic limit.

Shear strengths were then investigated. Undrained shear strength profiles have been estimated from CPT's through well-known correlations checked against undrained unconsolidated triaxial test (TRX-UU in Figure 7).
The samples taken from the body of the dyke show an undrained cohesion ranging from 80 to 100 kPa, independently of depth. Only occasionally, small values as little as 50 kPa have been registered. The same figure holds for the soil from the shallow deposit. Bedrock samples instead show undrained cohesion values ranging from 300 to 600 kPa.

Effective shear strength have been evaluated by drained (TRX-CD) and undrained (TRX-CU) triaxial tests. Figure 8 shows the results of the tests in the plane of invariants. The estimated failure envelope for both the soils from the dyke and the shallow deposit can be described with a negligible effective cohesion and a friction angle of 26°. For the bedrock, the failure envelope gives an effective cohesion of 80 kPa and a friction angle of 26°. Such values well compare with the typical figures of this soil formation (e.g. Segato et al., 2015; Ruggeri et al., 2016).
The residual shear strength was estimated by shear testing, either direct or ring shear. Samples from shallow deposit exhibit a failure envelope with a negligible effective cohesion and a friction angle ranging from 14 to 18°. Failure envelope for bedrock shows again negligible cohesion but a friction angle of 18°. This difference reflects those of the index properties between the two soils.

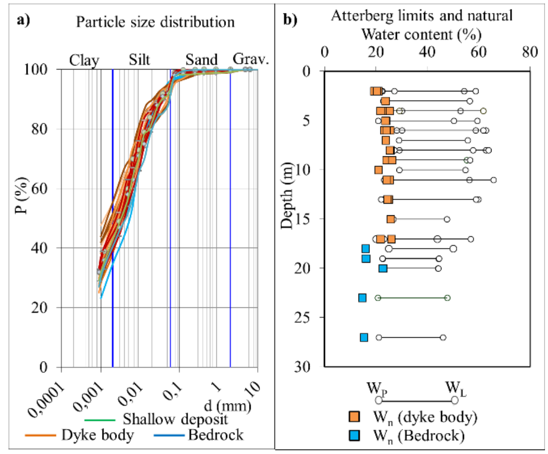

Figure 6. a) Particle size distributions; b) Soil index properties and natural water content

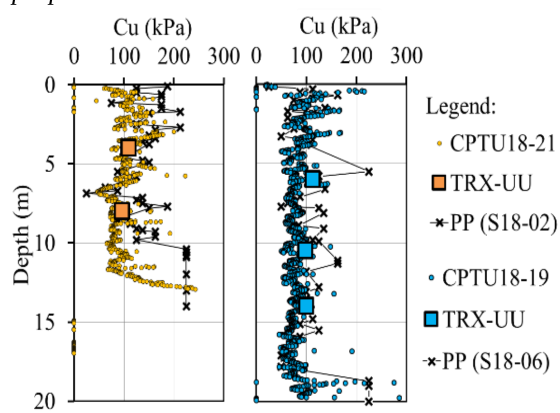

Figure 7. Undrained shear strength profiles: derived values from CPT's, pocket penetrometer (PP) and Undrained Triaxial tests (UU)

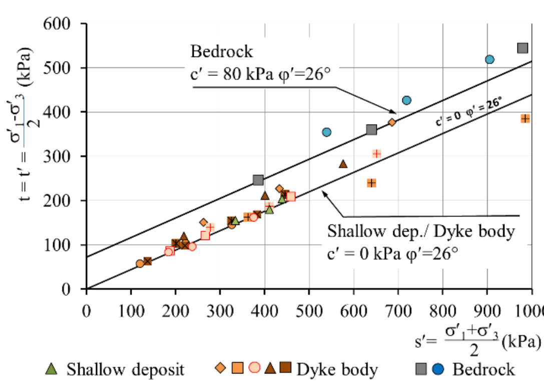

Figure 8. Failure envelopes from Triaxial Tests

5.3 Groundwater measurements

Site investigations before the construction of the dyke indicated the groundwater table rises up to the ground surface in winter and is at the bedrock - shallow soil interface during the dry season. The presence of the formation of "Argille Azzurre", always characterized at the scale of the soil element by very small permeability and in this case fairly homogeneous, justifies the assumption of impervious boundary at the bedrock. Piezometric heads were not monitored during construction, neither in the embankment nor in the foundation soil; instead, some piezometers, initially few open standpipes, have been installed only after collapse. As very high water level were detected, a more careful geotechnical investigation was planned to understand the possible role of pore pressures in the collapse. 6 Casagrande piezometers with filters placed at different depth, have been installed together with 21 vibrating wire piezometers with continuous reading and data recording. Pore-water pressures are more or less hydrostatic in the center of the dyke, from the crest down to the foundation soil. Figure 9 show the piezometric heads resulting from the measurements along the cross sections n.7 and n.9 of the dyke.

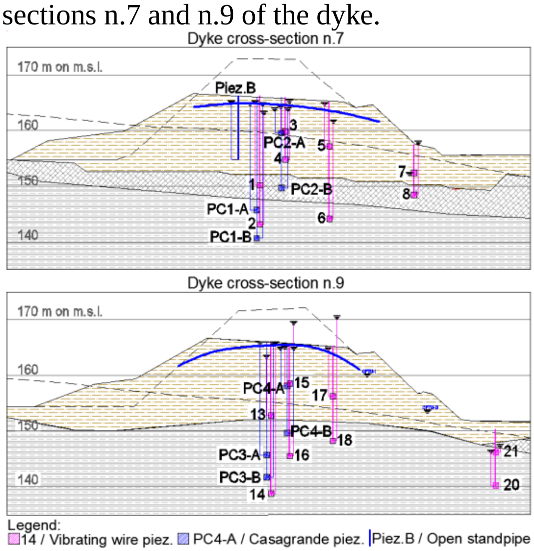

Figure 9. After collapse piezometric heads along cross sections n.7 and n.9 in Figure 4

The embankment is today fully saturated, and pore-pressures are significant both in the lower part of the dyke and in the ground. It is worth noting that two piezometers indicate that the piezometric level is well above the current crest.

6 DISCUSSION

After collapse geotechnical investigations suggest that the following three issues were not properly considered for the design of the earth structure:

- the presence of a paleo riverbed filled with soft soil that was not detected by the preliminary site investigations with the consequence of a dyke resting on fairly non-homogeneous foundation soils, partly bedrock, partly not;
- the interface between the shallow soft soil deposit and the bedrock, and characterized only by a very low friction, could become the ideal surface for sliding; this circumstance is related to the geological evolution of the valley which was not fully investigated for design;
- the onset of overpressures induced by the undrained loading of the soil in the dyke and in the foundation, resulting fully saturated in both cases, were not anticipated; high pore pressure values are measured even today, months after the collapse event.

Each of the above issues may have largely influenced the geotechnical safety of the dyke. However a back-analysis of the failure has shown that only their joint influence could be able to effectively reduce the high safety margin of the initial design to cause the failure. In other words, each factor having a negative impact on safety may have been well covered by the margins prescribed in technical recommendations. This case history support the convincement that failure of geotechnical works is very often of multi-causal origin as shown in the literature by Ruggeri et al. (2013) for a retaining wall and by Segato et al. (2015) for trench and tunnel.

From a general point of view it is useful to investigate when in design and during construction, the followed procedures have been favored the occurrence of a failure. For sure in the design phase, preliminary site investigations were unable to identify the paleo riverbed, even if their coverage of the site fulfilled typical technical recommendations. During construction, when excavations were carried out for the foundation of the embankment, field engineers underestimated the risk of the presence of weak interfaces, possibly at residual for geological reasons. During compaction, some modest deviation from optimal water content have been probably underestimated, with the consequence of a high saturation of the compacted strata.

7 CONCLUSION

The failure of a dyke built with compacted clay and reinforced by geogrids has been analyzed. The result of investigations allowed to assess the multi-causal origin of the collapse. Beyond the specific causes, probably a general underestimation of the real impact of the prospected earth structure has mined from the beginning the success of the initiative. For example, compaction procedures does not seem adequately designed and the effectiveness of the checks and monitoring during construction were not indeed appropriate to the complexity of the geotechnical work. Underestimation of critical details resulted in the development of overpressures in the groundwater due to the undrained response of the compacted clay and of the foundation soils. Such overpressures favor the instability along a failure surface where residual strength conditions hold for geological reasons. Remedial measures have planned as a direct consequence of the interpretations herewith presented. Layer of horizontal drains are presently drilled through the dam; more drains from a deep well to be excavated at the toe of the dam downstream will intercept overpressures in the foundation soil, where the failure surface is localized.

The analysis of this case history is however still preliminary and a final judgement on the failure as well on the given interpretations could be formulated only when the prospected remedial measures will be fully operational and above all will result effective to return the dam to its original objectives.

8 AKNOWLEDGMENTS

The financial contribution of the Italian Ministry of University and Research, project PRIN 201572YTLA_005, is greatly acknowledged.

9 REFERENCES

German Geotechnical Society (ed.) 1993. *Geotechnics of Landfill Design and Remedial Works*. Technical Recommendations GLR, ed. for the ISSMFE, Ernst & Sohn, Berlin

Ramke, H.-G. 2001. Appropriate Design and Operation of Sanitary Landfills. – *Int. Conf. on Sustainable Economic Development and Sound Resource Management in Central Asia*, Tashkent, Uzbekistan, 30 pages.

Ruggeri, P., Segato, D., Scarpelli, G., 2013. *Sheet pile quay wall safety: investigation of posttensionede anchor failures*. J. Geotech. and Geoenv. Eng., 139(9): 1567-1574.

Ruggeri P., Fruzzetti V.M.E., Vita A., Paternesi A., Scarpelli G., 2016. Deep-seated landslide triggered by tunnel excavation. *Proc. 12th Int. Symp. on Landslides (Naples, June 2016), S. Aversa et al. Ed., CRC Press 2016, 1759–1766*. DOI: 10.1201/b21520-219

Scarpelli, G., Sakellariadi, E., Fruzzetti, V.M.E. 2003, The dilatant behaviour of overconsolidated clays. *Proc. of Int. Symp. on Deformation Characteristic of Geomaterials. ISLYON '03. H. Di Benedetto et al Ed., Balkema, 451-460.*

Segato, D., Scarpelli, G., Fruzzetti, V.M.E., Ruggeri, P., Vita, A., Paternesi, A. 2015. Excavation works in stiff jointed clay material: examples from the Trubi formation, southern Italy. *Landslides*, 12:721–730.

US Bureau of Reclamation (USBR) 2012. *Design Standard n.13: Embankment Dams - Chapter 10: Embankment Construction*, Denver