Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering– Rahman and Jaksa (Eds)

The failure of an earth retaining wall: investigation and assessment of the causes

La rupture d'un mur de soutènement : enquête et évaluation des causes

Giuseppe Scarpelli, Paolo Ruggeri & Viviene M.E. Fruzzetti
Department of Materials, Environmental Sciences and Urban Planning, Università Politecnica delle Marche, Italy, g.scarpelli@staff.univpm.it

Alessandro Vita, David Segato
GES-Geotechnical Engineering Services, Italy

ABSTRACT: After seven years from the end of construction, a reinforced concrete cantilever wall sustaining a 9 m high embankment along a State Road in Italy, showed a sudden and unexpected outwards displacement at its top. This event caused the immediate closure of the traffic over the road and promoted an investigation by the Contractor to design and implement suitable remedial measures. Early interpretations suggested that a rigid rotation of the entire wall may have occurred due to a bearing capacity failure, so that the underpinning of the wall base through bored piles was planned. However, during the repairing works, the wall stem rotated and collapsed by full overturning. In Italy every collapse involving public works has to be prosecuted, irrespectively of the severity of the consequences and a judicial investigation was then open implying new analyses and studies even by the Contractor. In the paper, the results of these latter studies are presented and the factors playing a role in the collapse are analyzed.

RÉSUMÉ : Sept ans après la fin de la construction, le long d'une route nationale en Italie, un mur cantilever en béton armé de 9 m de haut a montré en tête un déplacement soudain et inattendu vers l'extérieur. Cela a engendré la fermeture immédiate de la circulation et activé, de la part du constructeur, une étude sur les causes afin de concevoir et mettre en œuvre les mesures correctives plus adaptées. D'après les premières interprétations une rotation rigide de tout le mur avait été supposée à cause d'une capacité portante des terrains de fondation insuffisante. En conséquence, le renforcement de la fondation par pieux forés avait été envisagée. Lors des travaux de réparation, le mur vertical s'est effondrée par retournement complet. En Italie, tout accident impliquant des travaux publics, quelle que soit la gravité des conséquences, doit faire l'objet d'enquête judiciaire qui a donc été ouverte. Cela a engendré des nouvelles analyses et études, même de la part du constructeur. L'article présente les résultats de ces dernières études ainsi que les analyses des facteurs qui ont joué un rôle clé dans le collapse de l'ouvrage.

KEYWORDS: Failure; Cantilever wall; Forensic Geotechnical Engineering; Safety; Investigation.

1 INTRODUCTION

Reinforced cantilever walls are very common geotechnical structures able to provide lateral support for steep, often vertical, cuts in soils. The stability of such walls is assured by self-weight and the weight of the backfill over the base; such structures are then typically classified as gravity walls (see, for example Eurocode prEN1997-3).

The design of such kind of structures is based on simplified assumptions (e.g. reaching the active limit state conditions within the backfill) that rarely capture the essence of the soil-structure interaction. Compaction procedures of the backfill, differential settlements at the wall base, deflection of the stem, ratio between stem height and heel slab length are, among the others, the main factors that determine the value of the lateral pressures exerted by soil on the wall (Ingold, 1979; Bentler and Labuz, 2006). Despite these non-trivial issues, the wall proportions recommended in the literature and the magnitude of the safety factors prescribed by design standards usually enable the design of safe walls able to tolerate the adoption of oversimplified hypothesis (Clayton et al., 1993; Bowles, 1996).

However, some cases of poorly performing cantilever retaining walls were also recorded. The analysis of the distresses and failures observed in such structures are always of great interest to improve the quality and safety of the geotechnical design. Marsh et al. (1996), Day (1997), Abdullahi (2009), by presenting some failure cases of cantilever walls, try to outline the main causes determining such a dangerous event. In particular, the efficiency of drainage systems, the mechanical properties of the backfill, the compaction procedure for the backfill and earthquake loadings, have been pointed out as the leading causes for the majority of the failure cases.

In this paper the failure of a reinforced cantilever wall sustaining a 9 m high road embankment is presented and analyzed. Two aspects of the case study appear particularly interesting: the time elapsed between the construction and the failure, that was 7 years, and the severe corrosion process that have affected the rebars of the stem at the junction with the foundation base.

2 PROJECT DESCRIPTION

The reinforced cantilever wall of interest has been built in the first semester of 2010 as part of a two-level intersection between two double-lane roadways, in Italy. As shown in the cross-section of Figure 1, the wall supported the embankment of one ramp of the intersection that, for reason of intersection geometry, is placed very close to the first level main road. The total height of the wall is about 9.00 m, 8.00 m of stem height and 1.00 of footing thickness. The concreting of the footing has been made over a lean concrete layer able to make clean and uniform the bottom of the excavation. The footing is 5.40 m wide, with a toe length of 1.00 m on the front face and a heel length of 3.40 m on the back face of the wall. The ratio between footing width and total height, equal to 0.6, is usual for such kind of wall in seismic areas of Italy. The first half of the stem is 1.00 m thick; thickness then reduces to 0.80 m up to the top. Flexural reinforcements were calculated at three critical sections: at the base of the stem, at the toe and heel face of the stem. As a result, the stem base was reinforced with 1 bar d=20 mm every 0.20 m on the tension side and with 1 bar d=12 mm every 0.20 m on the compression side.

The wall retains a 9 m high embankment over the footing with the ground slightly sloping towards the top of the stem and is finished with 2 m of made ground over its base at the front.

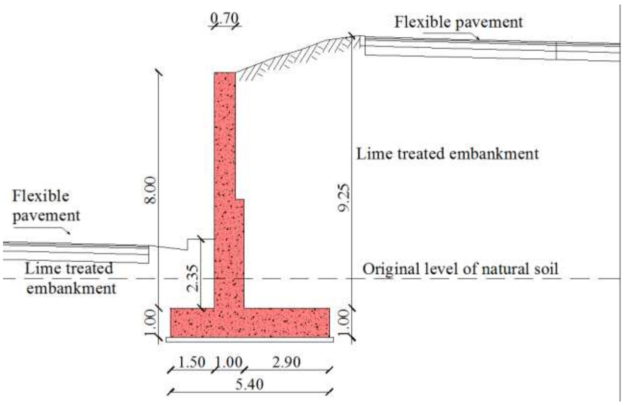

Figure 1. Cross-section of the reinforced cantilever wall (segment of maximum height)

The construction site is a flat area originated by the alluvial sedimentation of a river now flowing about 300 m far away. Consequently, the foundation of the wall lays over a thick alluvial deposit (thickness > 13 m) made of alternate layers of silty sand and sandy silt with clay, typically with the finest layers found in the upper portion of the deposit. Beneath the alluvial deposit, overconsolidated marly clay, the typical bedrock of the area, is encountered (Blue Clay formation). Reference values of the geotechnical properties in terms of soil unit weight (γ), effective cohesion (c'), effective friction angle (φ') and Young's modulus (E'_{25}, - i.e. the secant modulus at 25% of the maximum deviatoric stress) are the following: γ=18.0-20.0 kN/m^3, c'=10-20 kPa; φ'=27-32°, E'_{25}=10-25 MPa. The variability of ground properties reflects the typical heterogeneity of alluvial deposits. Ground investigations in the zone of influence of the wall footing indicated the prevalence of fine soils with fine content (clay and silt) between 50 and 80%, void index between 0.6 and 0.7 and that the soil strength can be represented by an effective stress envelope with c'=12 kPa and φ'=30°.

The road embankment that constitutes the backfill of the wall was built with compacted layers of silty clay stabilized with lime. Such operation was performed by using a large compactor with vibratory padfoot drum, except for a 1.5 m strip right behind the wall where a lightweight vibrating roller was used. The soil for the backfill originates from the excavation of tunnels carried out along the mega lot under construction in the area (Ruggeri et al., 2020). To analyze the collapse of the wall, a geotechnical investigation specially focused on the ground forming the embankment; relevant results will be presented and discussed in the paper.

Finally, the groundwater level was found at about 4 m below the original level of the ground, slightly above the extrados of the wall base. Seasonal variations of the groundwater level are however expected due to the presence of the river not far from the construction site.

3 INVESTIGATION OVERVIEW

3.1 *Signs of structural distress*

Signs of structural distress were not observed until September 2016, as proven by an image showing a correct alignment of the wall stem over the whole length of the collapsed segment of the wall. However, a forward rotation of the stem clearly appeared in May 2017. The horizontal displacement at the top was estimated about 200-250 mm. Fortunately, the access to the area was immediately inhibited, traffic lanes were diverted, and an extensive high-precision topographical monitoring was quickly established. In the following days, no progression of the displacements could be observed. This circumstance with the lack of any evident damage of the structure justifies an early interpretation of the kinematics of the event as the rigid rotation of the whole wall at its base, indicating a potential bearing capacity problem. The contractor, that for Italian law is still responsible for any repairing works within ten years from end of construction, planned to improve the wall foundation by underpinning. Precisely, the designer planned to align a row of bored piles in front of the foundation to be connected to the existing structure through a concrete beam and several concrete counterforts of the wall stem. To make construction works easy and safe, the piles were bored without removing the backfill on the front side of the wall.

3.2 *Failure event*

With the piles being completed, the remotion of the ground above the slab initiated to allow the construction of the counterforts. It was during such excavation phase that the forward rotation of the stem accelerated ending with a full collapse by toppling; the collapse included the ground wedge which was retained by the wall as sketched in Figure 2. The impact of the stem on the ground as profiled for temporary excavation works together with the weight of the collapsed soil caused the failure of the stem at the weakest cross section, that is where thickness changes. Again fortunately, no one was injured in the event, thanks to the ductile mode of failure that occurred.

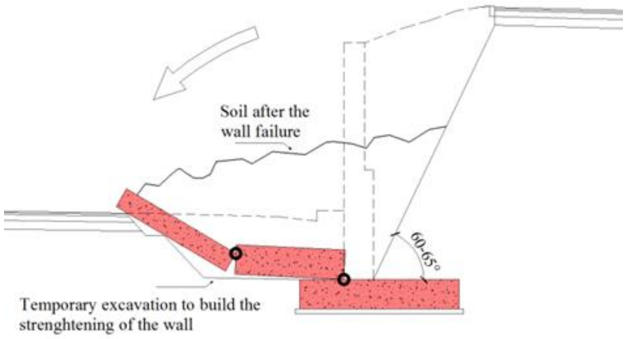

Figure 2. Scheme of the wall collapse

3.3 *Collection and interpretation of data*

3.3.1 *Early evaluations*

After removal of the materials burying the broken parts of the concrete structure it was evident that the foundation slab was still intact and surprisingly not at all displaced from its original position. On the other hand, the stem resulted overturned with a clear separation from the base of the wall at the joint between the stem and the base. Indeed, all the rebars crossing the joint were truncated at the level of the joint, without any cracks or damage of the concrete around the bars.

These findings suggested that a thorough investigation on the resistance of the foundation-stem joint and about the earth pressures exerted by the backfill was necessary to clarify the causes of the collapse.

3.3.2 *Properties of rebars and concrete*

Design specifications of the wall provided reinforcement steel of class B450C (yield strength f_{yk} = 450 MPa; tensile strength f_t = 540 MPa; elongation at failure ε_{uk} =≥ 7.5%) and a concrete belonging to strength class C30/37 (i.e. characteristic 5%

cylinder/cubic strength determined at 28 days equal to $f_{ck}/f_{ck,cube} = 30/37$ MPa) according to Eurocode 2 (EN 1992-1-1). To verify the compliance between design specifications and actual properties of the reinforcing steel, several specimens have been taken from the bars and a number of concrete specimens were extracted for tensile and compressive strength tests, respectively.

The rebar specimens test results were compliant with nominal values, showing a yield strength (f_y) between 500 and 565 MPa, tensile strength (f_t) between 604 and 668 MPa and elongation at failure between 12.0 and 14.7%.

Uniaxial compression test on the concrete samples gave strength values much higher than expected, showing a compressive cylinder strength (f_c) between 50 and 69 MPa, more than 1.5 times the values assumed for design.

In conclusion, the result of the tests on the structural materials, the steel and the concrete, clearly indicated that failure could not be explained by poor material properties.

3.3.3 *Geotechnical characterization of the backfill*

A second line of investigation was directed towards the geotechnical characterization of the backfill, forming the body of the road embankment.

The uniformity of the construction process allowed to assume that soil properties could be determined by investigating the embankment in zones not directly involved in the collapse, although very closely. Boreholes were drilled in the embankment and several undisturbed samples were taken for laboratory testing. However, several samples were also taken in the collapse area, from the exposure after removal of the collapsed soil. The investigation was completed by some field flat dilatometer (DMT) testing.

Figure 3 presents some results from laboratory testing on the backfill soil. The backfill soil resulted fine graded, made of 20% of clay, 50% of silt, 25% of sand and 5% of gravel. The grading indicates that most of the particles fall between coarse silt and fine sand, consistently with the effect of the lime treatment of the original soil. The soil resulted of low plasticity, with Atterberg limits falling along the A-line. The natural water content was almost constant with depth as typical for a layered embankment and lower than the Plasticity Limit, indicating a brittle soil state. The undrained shear strength c_u from pocket penetrometer sounding on the laboratory samples ranges between 100 and 300 kPa, in agreement with the range c_u=152±89 kPa suggested by Haigh et al. (2012) for soil at plastic limit. Direct shear testing provides a range for peak effective cohesion c′ and friction φ′ by means of a lower failure envelope for which c′ = 15 kPa and φ′ = 26° and an upper envelope for which c′ = 30 kPa and φ′ = 30°.

Flat dilatometer tests (DMT) were carried out from the top of the embankment, in the area of the collapse. Field measurements, interpreted according to TC16 DMT Report (2001), are shown in Figure 4 for three investigation locations. The material index (I_D) correctly identifies the soil as a sandy silt. The constrained modulus (M) derived from DMT testing gives representative values between 15 and 30 MPa, with some higher values probably consequence of the "crust" of the layers compacted by the roller. The undrained shear strength, evaluated for strata recognized as fine graded (i.e. with $I_D < 1.2$), ranges between 30 and 100 kPa, lower than the values from the pocket penetrometer on laboratory samples. For the objective of the investigation on the causes of the collapse, of particular interest is the horizontal stress index (K_D), a parameter linked to the at rest earth pressure coefficient (K_0) amplified by the blade penetration effect. In NC clays K_D assumes the typical value of 2. When $K_D > 2$ a fine graded soil is cemented, aged or overconsolidated.

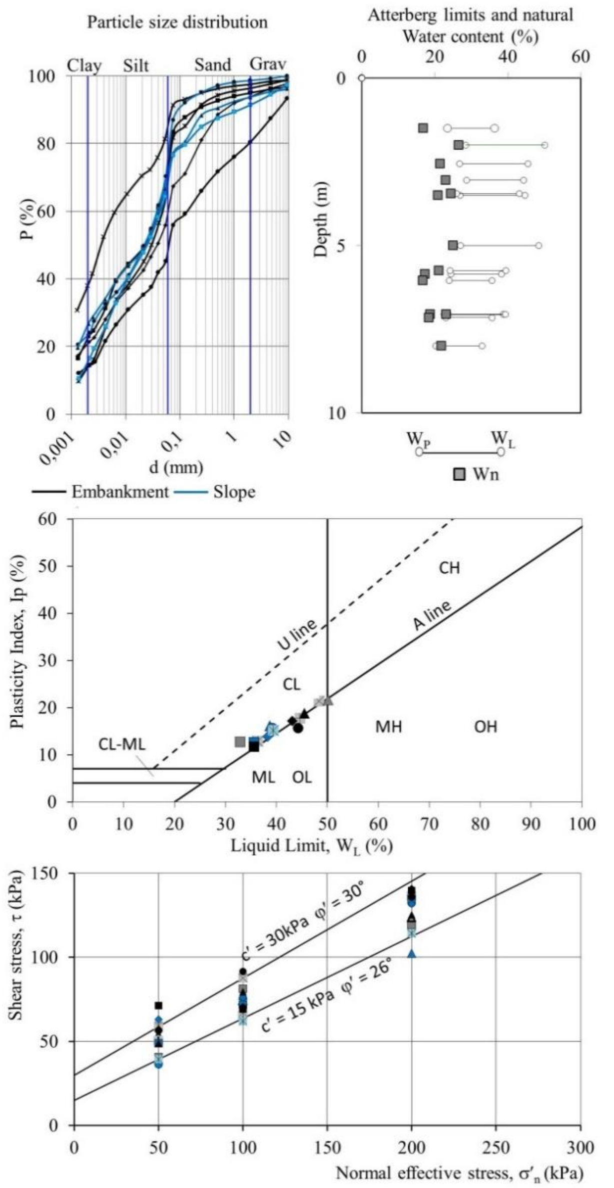

Figure 3. Laboratory test results on undisturbed samples of the embankment

As shown in Figure 4, the first 6 m of the embankment present very high values of K_D. Having no relevant indication of cementation from laboratory samples and being the embankment manmade (i.e. no ageing), it is possible to infer that such high values of K_D are justified by heavily overconsolidation of the tested soils. Empirical correlations available in the literature allows to estimate the value of K_0 from K_D for natural soils; such correlations are not useful for lime-treated and roller compacted materials. However, former data indicate unequivocally that the horizontal stresses in the embankment are very high, well above those expected for NC deposit.

4 DISCUSSION

So far, the results of the investigations have excluded any lack of capacity of the structural materials and of bearing capacity of foundation but pointed out the occurrence of very high values of the earth pressures on the wall stem that needs to be specifically analyzed. Another aspect to be explained is why the wall collapsed only years after its construction.

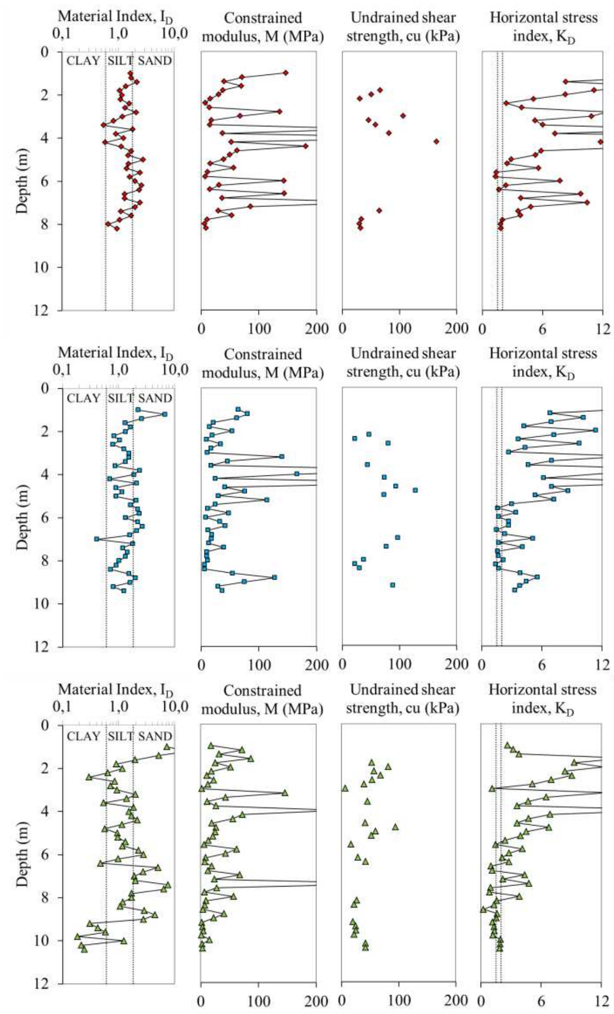

Figure 4. DMT results

4.1 *Evaluation of the earth pressure against the wall*

A cautious estimate of the strength parameters c′=26 kPa; φ′=26° has been assumed from laboratory testing on the backfill. With this assumption, the five earth pressure distributions and bending moments presented in Figure 5 have been computed:

1. backfill reached an active limit state; earth pressures are according to the Rankine's theoretical distribution;
2. backfill reached an active limit state; earth pressures are plotted according the theoretical distribution but the plot is forced to neglect the "tension zone" (see Bowles, 1988);
3. backfill applies the at-rest earth pressures to the wall;
4. backfill applies the earth pressure resulting from compaction through the use of a lightweight roller;
5. backfill applies the earth pressure resulting from compaction through the use of a heavy roller.

Condition 1 assumes the existence of a "tension zone" in the soil consistent with a non-zero cohesion. This assumption is clearly not safe but has been considered because many practitioners refer to it.

Condition 3 assumes that the wall cannot move horizontally when backfilled.

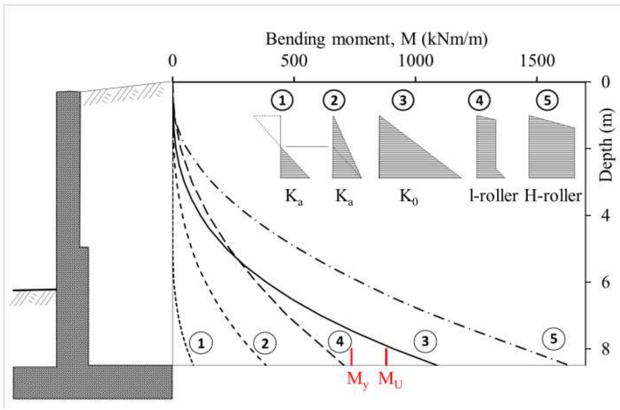

Figure 5. Assumed earth pressures and consequent bending moment distributions

Conditions 4 and 5 consider the effect of compaction. It has been mentioned that two different rollers have been used, a large roller for the embankment body and a little roller in the vicinity of the wall. Both possibilities were considered.

To evaluate the lateral earth pressure due to compaction, the simple analytical method proposed by Ingold (1979) is considered. This method assumes that the running of the roller determines a temporary concentrated surcharge on the ground whose effect decrease with depth due to diffusion of the induced stress. The increase of vertical stresses causes the increase of horizontal stresses. After compaction, vertical stresses reduce to equilibrate soil weight while the horizontal stresses do not reduce. However, the value of the horizontal stresses cannot exceed the passive limit values that depend on the shear strength which is, in turn, related to vertical stresses and friction. So, the final effect on horizontal stresses of compaction can be represented by a plot in which its lower part is determined by the induced stress distribution with depth and its upper part is limited by the passive pressure values. The maximum value of the horizontal stress increment is reached at the intersection between these two diagrams, whose position is defined as critical depth (z_c) and can be evaluated as:

$$z_c = K_a\sqrt{2p/\pi\gamma} \quad (1)$$

where p is line load equivalent to the roller action, γ is the soil unit weight and K_a is the active lateral earth pressure coefficient. The value (σ'_{hm}) of the horizontal pressure at the critical depth z_c according to Ingold is:

$$\sigma'_{hm} = \sqrt{2p\gamma/\pi} \quad (2)$$

Being the embankment compacted in subsequent strata, the final plot of the horizontal pressure is obtained by combining several plots progressively shifted upward, so that a uniform distribution of the pressures of magnitude σ'_{hm} results. For depth $z < z_c$ the pressure assumes the passive values $K_p\gamma z$. With the increasing of the height of the embankment, a further critical depth (d) can be reached, when the horizontal pressure induced by compaction is equal to the conventional active pressure $K_a\gamma d$. This depth is given by the expression:

$$d = K_p\sqrt{2p/\pi\gamma} \quad (3)$$

Note that the method of Ingold requires to convert the effect of the roller in a line load of magnitude p. Ingold suggests to make this transformation by dividing the weight of the drum for its width, neglecting the difference on the induced stress between a point load and a line load. For vibratory roller the equivalent line load is evaluated by adding to the deadweight of the roller the centrifugal force generated by the roller vibrating mechanism. According to the information collected from the contractor, the line load of the two used rollers have been evaluated: the heavy roller (H-roller) and the lightweight roller (l-roller) produced a line load of 207 kN/m and 33 kN/m, respectively.

The bending moment distributions shown in Figure 5 refer to a stem 1 m wide. The corresponding bending resistance of the stem at the base (M_U) is equal to 880 kNm/m and this value is also indicated in the figure. Note that this resistance was calculated by considering the measured values of the strength of materials, that is no safety factor was considered. Such value decreases to $M_y = 736$ kNm/m if the nominal and not the measured strength at yielding of the steel is considered.

The comparison between the bending moments at the stem base for the 5 earth pressure distributions and the structural resistance of the stem cross section allows to infer that the K_0 and H-roller moment distributions (diagram n.3 and n.5) cannot be sustained by the wall. In other words, actions from such lateral stress distributions would have been sufficient to determine the collapse of the wall since the construction of the backfill. On the contrary, the fact that the wall operated for several years before collapse, indicated that the initial earth pressure had to be consistent with the structural strength resources of the wall itself. Hence, earth pressure diagrams n.1, 2 and 4 fulfill this requirement. Taking into account the construction phases, the distribution 4 appears to be a good approximation for the soil pressures acting on the wall at the end of construction.

Measurements of stresses in the concrete for intact, geometrically similar, segments of the wall confirmed values of lateral pressures consistent with those of distribution 4. Such measurements have been done by using flat jack testing, that is a test method developed for the evaluation of the compressive stress in masonry wall (ASTM Standard C 1196-91) and by the over-coring technique using a doorstopper (Leeman, 1969), that is a method to investigate the stress state in rock mass.

4.2 *The delayed failure of the wall*

An aspect of particular relevance in the analyzed failure is the delay of the collapse after the construction of the wall. The construction of the wall ended in 2010 and the supported embankment was completed in 2011. In November 2011 the road was opened to traffic and nothing odd was reported until May 2017, when a misalignment of the wall at the top led to the closure of the road. So, for about 7 years after its construction and for more than 5 years of operative life the wall performed appropriately.

The first hypothesis made to explain this behaviour was the fatigue effect related to the passage of heavy load on the road sustained by the wall. Luckily, the Italian regulations require specific authorization for every passage of heavy good vehicle (HGV), so it has been possible to evaluate that about 100 HGV were authorized to use the road between 2011 and 2017. However, the distance of the carriageway from the wall and the localized force exerted by the tires of these vehicles makes the influence of HGV to the lateral pressure against the wall very little.

A second possible explanation resulted from the accurate analysis of the reinforcement bars in the zone of the joint between stem and foundation, exactly where they broke due to the tensile force. As shown in Figure 6, a close examination of the two sides of the broken bar, demonstrates that a severe localised corrosion occurred before failure. This phenomenon is made evident by the thick rust layer that determined a narrowing of the resistant area of the bar. Such loss in steel was relevant because it involves the entire surface of a "TempCore" reinforcement bar. This is because the "TempCore" treatment, very common for concrete rebars (see Noville, 2015), determines different properties of the steel in the cross-section of the bar: a thin thickness of steel along the perimeter acquires very high strength, while the core of the bar remains mild. Therefore, the loss of the external area of the bar determines a more than proportional reduction of its total tensile strength.

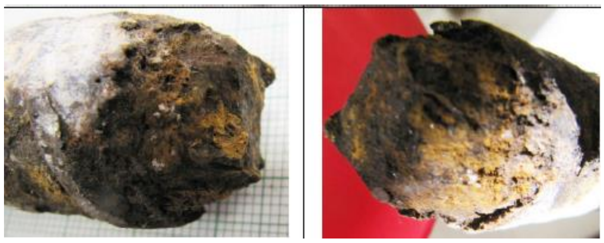

Figure 6. Picture of the two sides of a steel bar ø20 picked up from the zone of failure between the stem base and the foundation of the collapsed wall (courtesy of Fratesi 2018)

But why a so relevant localized corrosion process affected several rebars at the stem base?

Some measurements demonstrated the existence of a "macrocell corrosion" between actively corroding areas of rebars and large passive areas of the wall. In Jaggi et al. (2001) is possible to find detail of this dangerous phenomenon. In summary, rebars in concrete structures, thanks to the high pH environment that take place in concrete, are generally protected by the spontaneous formation of a thin protective oxide film (passive film). Besides, all the bars in a concrete structure result electrically connected. If a little portion of the rebars network loses its protection it is likely that the exposed area of steel will suffer a heavy corrosion process. This happens because there is a little area where the corrosion is able to operate (local anode) and a large area of steels where oxygen is reduced to hydroxyl ion (large cathode). Anode and cathode form a short circuited corrosion cell, with the flow of electrons in the steel and of ions in the pore solution of the concrete. This process is schematically represented in Figure 7.

Measurements of the corrosion potentials of the anode and cathode supported the existence of this process in the considered case study. Therefore, the delayed failure of the wall can be explained by the time required to corrosion to proceed so much to reduce the rebars cross section enough to produce their tensile failure.

But why a crack opened at the stem base and why a good quality concrete was unable to protect the bars?

The investigation suggested the following combined causes: a high value of the bending moment at the base of the stem; the existence of a "cold joint" at the contact between stem and foundation; the chloride ions contained in the pore water of the embankment.

As showed before, the rolling process of the embankment determined a high value of the earth pressures and a corresponding bending moment distribution with the maximum at the stem base was originated. As a consequence, the opening of a crack at the stem-foundation joint was possible.

Moreover, the interruption in concreting between the foundation and the stem determined the presence of a weak plane in the concrete mass. This occurrence is related to the construction phases of the wall and cannot be totally avoided. It is possible to treat the surface of the hardened concrete to allow a good adhesion of the new fresh concrete, but this does not represent the standard in Italy, except for special watertight structures (caissons, concrete water cisterns).

Finally, the presence of chloride ions in the rust taken from the bars where ascertained by chemical analysis. Such presence was unexpected because the wall is located far from the sea and in a low-altitude and temperate climatic region where the use of de-icing road products is unlikely. However, chemical analysis of the water after immersion of the soil showed the existence of a little contents of chloride ions. So, the most probable origin of this ions was related to the marine origin of the Blue clay, the soil coming from the tunnel excavation that, thanks to its very low permeability, has probably preserved part of the marine salt contained during the sedimentation process that originated this formation, some millions of years ago.

The contemporary preexistence of unfavorable conditions and details of apparently minor importance led to the activation of a serious corrosion process. The relevance of a corrosion process on the bad behaviour of geotechnical structures was observed in other circumstances by the Authors (Ruggeri et al., 2013), when dealing with structures in marine environment where this phenomenon play an important role in the design process.

4.3 *Summary remarks*

The extensive and multidisciplinary approach adopted to investigate the causes of the wall failure led to a valid explanation of the event. The following process has very likely happened:

1. the rolling of the embankment during construction determined the develop of high values of earth pressures on the wall;
2. the soil in front of the wall exerted an effective constraint of the foundation so that an active state of stress in the sustained soil could not be reached;
3. the maximum value of the bending moments, at the base of the stem, favored the opening of a tension crack;
4. the presence of a “cold joint” between the stem and the foundation and the high strength of the concrete favored the opening of a single tension crack of large width;
5. the clayey soil used to build the embankment contained some “fossil” chloride ions that concentrate on the bar (attracted by the anode potential) determining the localized destruction of the passive protective film;
6. the activation of a “macro-cell corrosion” with a very limited anode and a large cathode areas determined the rapid corrosion of the rebars in the zone of the joint;
7. when the strength reduction of the bars was enough, the wall rotated to find a new precarious equilibrium by mobilizing the passive pressure in the soil in front of the wall;
8. the removal of the soil in front of the wall caused the collapse by toppling of the stem.

5 CONCLUSIONS

The comprehensive investigation carried out to explain the failure of a cantilever retaining wall located along a new road has been presented. The findings highlight that a geotechnical collapse is always the result of the combination of many unfavorable factors, among others the constraints imposed to the wall, the effects of the compaction of the earth fill, the corrosion of the steel bars. This case study pointed out once more the importance of the care in the details to guarantee a good behaviour of the geotechnical structures. It is worth noting that only the use of a multidisciplinary approach made possible to achieve a credible explanation of the collapse event.

6 ACKNOWLEDGEMENTS

The authors wish to thank the Contractor for the availability of technical data at the base of this paper. A special thanks to prof. Romeo Fratesi in the understanding the tricky aspects of the corrosion phenomena. This research benefited from the project 2020 “*Analisi di fenomeni di collasso di opere geotecniche*” funded by the Università Politecnica delle Marche.

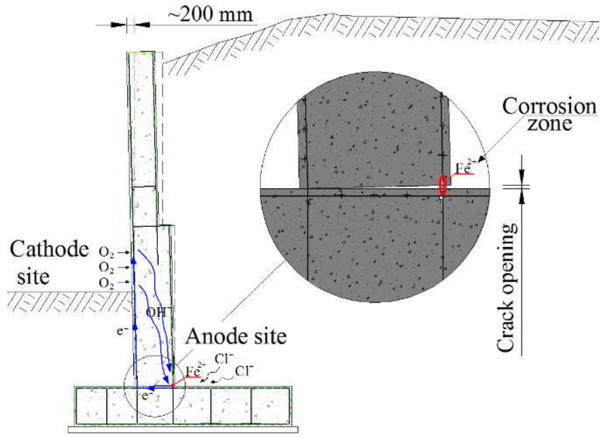

Figure 7. Simplified scheme of the “macrocell corrosion” activated on the analysed wall

7 REFERENCES

Abdullahi M. M. 2009. Evaluation of causes of retaining wall failure. *Leonardo Electronic Journal of Practices and Technologies*. 14:11-18.

ASTM (1991) In-situ compressive stress within solid unit masonry estimated using flat-jack measurements, ASTM Standard C 1196-91, 1991

Bentler J.G., Labuz, J. F. 2006. Performance of a cantilever retaining wall. *Journal of geotechnical and geoenvironmental engineering*. 132: 1062-1070.

Bowles, J. E. (1988). *Foundation analysis and design*. New York: McGraw-Hill.

Bowles, J.E. 1996. *Foundation analysis and design*. McGraw Hill, New York.

Clayton C.R.I., Milititsky J., Woods R.I. 1993. *Earth Pressure and Earth Retaining Structures*, Spon Press, Taylor and Francis Group, 2nd Edition 1993

Day, W.R. 1997. Design and Construction of Cantilevered Retaining Walls. Practice *Periodical on Structural Design and Construction*, 2 (1), 16-21.

Fratesi R. (2018) Personal communication

Haigh SK, Vardanega PJ and Bolton MD (2012) The plastic limit of clays. *Géotechnique*, http://dx.doi.org/10.1680/geot. 11.P.123.

Ingold, T. S. 1979. The effects of compaction on retaining walls. *Géotechnique*, 29(3): 265–283.

Leeman, E. R. (1969) The 'Doorstopper' and triaxial rock stress measuring instruments developed by CSIR. *Journal of the South African Institution of Mining and Metallurgy*, Vol. 69, pp. 305-339.

Marsh, E. T. and Walsh, R. K. 1996. Common Causes of retaining-Wall Distress: Case Study. *Journal of Performance of Constructed Facilities*, 35-38 pp.128.

Noville, J. F. (2015) TEMPCORE®, the most convenient process to produce low cost high strength rebars from 8 to 75 mm. In Proceedings of the METEC and 2nd European Steel Technology and Application Days (ESTAD) Conference, Düsseldorf, Germany, 15–19 June 2015.

Ruggeri P., Fruzzetti V.M.E., Scarpelli G. (2020). Design strategies to mitigate slope instabilities in structurally complex formations. *Geosciences* (Switzerland), 10(2), Article number 82.

Ruggeri P., Segato D., Scarpelli G. (2013). Sheet Pile Quay Wall Safety: Investigation of Posttensioned Anchor Failures. Journal of Geotechnical and Geoenvironmental Engineering, 139(9), 1567-1574.

TC16 DMT Report (2001). *The Flat Dilatometer Test (DMT) in Soil Investigations*. A Report by the ISSMGE Committee TC16. 41 pp. Reprinted in Proc. DMT 2006, Washington D.C.