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ABSTRACT

In this paper two case histories are addressed. The first case history describes the underpinning and reinforcement of huge retaining walls due to the crossing of the old Rossio railway station building (1886) by an underground gallery (for the Lisbon Metro). A careful study was performed for this station in order to avoid unacceptable damages. These walls sustain an embankment on which the railway platforms were built.

The second case history deals with the stabilization works of a landslide in Portugal. The lessons arising from technical and non technical factors related with slides are analysed, and the anchored retaining walls founded on micropiles to stabilize the hills are described. The field and laboratory tests are referred, as well as the geological-geotechnical model.

For the both case histories the design of the retaining walls was based in Eurocode 7. Some results of the global stability analyses as well as the stress-strain analyses are presented. The seismic design was based in Eurocode 8. The results of anchorages tests and micropiles to calibrate the design values are presented.

INTRODUCTION

This paper begins with a background of the Rossio railway station located in Lisbon. For the construction of an underground gallery for the Lisbon Metro, it was necessary to perform the underpinning of an east-west and a north-south retaining walls, with 13m in height and a thickness around 3 m. Both walls sustain an embankment on which the railway platforms were built. The walls were later reinforced in order to ensure their long-term safety against rotational failure.

The second case history deals with the stabilization works of a landslide in Portugal. The lessons arising from technical and non technical factors related with slides are analysed, and the anchored retaining walls founded on micropiles to stabilize the hills are described.

For the two case histories the main geological conditions are described. The field and laboratory tests are referred, as well as the geotechnical characteristics. The methodology to design the retaining walls, based in Eurocode 7, is introduced. The results of the global stability analyses, as well as the numerical analyses, are described. The seismic design is based in Eurocode 8. The results of anchorage tests and investigation tests performed on micropiles are discussed.

Some final considerations are presented.

ROSSIO STATION CASE HISTORY

BACKGROUND

Due to the crossing of the Rossio railway station building (1886-) by an underground gallery (for the Lisbon Metro), a careful study was performed for this station in order to avoid unacceptable damages. The building had originally three levels: ground floor (+11.86), 1st floor (+19.93) and 2nd floor (+25.96). For the installation of a Commercial Centre two new floors were built: intermediate first floor (+15.21) and intermediate second floor (+22.79).

For the construction of the underground gallery, it was necessary to perform the underpinning of an east-west and a north-south retaining walls, with 13m in height and a thickness around 3m. Both walls sustain an embankment on which the railways platforms were built.

The underground gallery was built underneath the whole extension of the north-south retaining wall and part of the east-west retaining wall.

For the underpinning of the retaining walls, more than a dozen large reinforced concrete frames were erected, side by side, under the base of the walls (Consortium, 1996b). This simple but massive structures allowed the excavation of the space for the installation of the Metro gallery (Fig. 1), under the beam of each frame.

Other parts of the station building had also to be underpinned, by means of r.c. frames, micro-piles and jet-grout columns.

In the design phase of the underpinning of the retaining walls, it was recognized that the walls didn't verify the safety conditions required by Eurocodes 7 and 8 and Portuguese Actions Code, specially in what concerns the rotational failure scenario for both static and seismic actions. Therefore it was decided to reinforce both walls by means of (i) definitive anchorages and a grid of r.c. beams, in the case of the north-south wall (Figueiredo Ferraz, 1997); (ii) micropiles going through the body of the wall and into the ground behind and under it, in the case of the east-west wall (Figueiredo Ferraz, 1998).

MAIN GEOLOGICAL CHARACTERISTICS

In this section the main geological characteristics of each layer are presented, based on the information collected from borings executed at Rossio site, as shown in Fig. 1 (Consortium, 1996a).

<u>Landfill</u>
The landfill is composed by sandy clayey material with a thickness varying from 3.5 to 20 meters.

<u>Estefânia Areolas</u>
The unit is composed by sandy materials or silty sandy materials with thickness around 3 meters.

<u>Clays and calcareous rocks from Prazeres</u>
This lower unit is composed by dark grey to dark brown clay, with layers of sandy material, up to the depth which is significant to the design of the structures of the Rossio station.

GEOTECHNICAL CHARACTERISTICS

After an evaluation of the results of the field tests and the laboratory tests, the following geotechnical characteristics were adopted (Consortium, 1996a):

<u>Landfill</u>
Unified classification: CL, CH
% passing sieve # 200 (ASTM): 28 to 100%
Liquid limit: 18 to 47%
Plastic index: 3.4 to 26.7%
Uniaxial compressive tests:
Strength (C_u): 32.7 to 169.7 kPa
Tangent elasticity modulus: 4.3 to 10.7 MPa
Triaxial tests (C.U.):
Cohesion c (in effective stresses): 10-30 kPa
Friction angle ϕ (in effective stresses): 30-33^o
k (permeability coefficient): 10^{-9} m/s
Poisson ratio: 0.35
The most representative SPT values of this material are between 4 and 18.
CPT results were between 0.2 and 5 MPa.

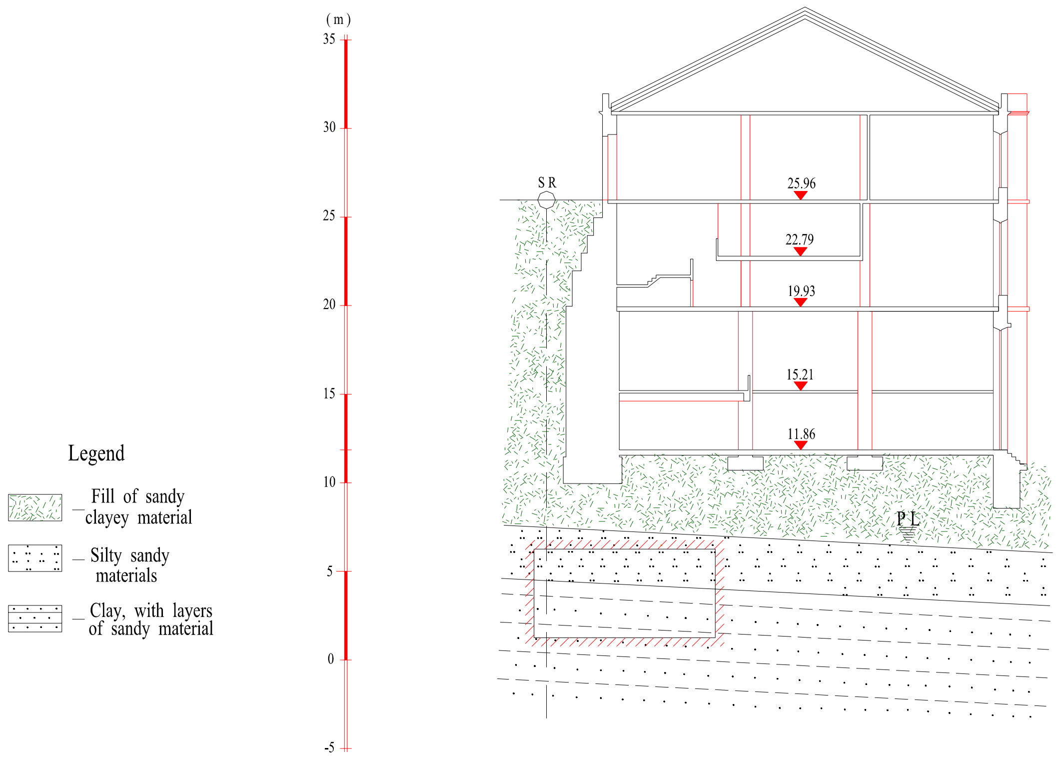


Fig. 1. Geological profile (adopted from Consortium, 1996a)

Estefânia Areolas
Cohesion c (in effective stresses): 0
Friction angle ϕ (in effective stresses): 33°
Elasticity modulus: 36 MPa
k (permeability coefficient): 10^{-5} m/s.
SPT results were between 6 and 39 blows.

Clays and calcareous rocks from Prazeres
Unified classification: CL, CH
% passing sieve # 200 (ASTM): 32 to 75%
Liquid limit: 28.3 to 43.4%
Plastic index: 4.3 to 21%
Triaxial tests (C.U.):
Cohesion c (in effective stresses): 2.5-33 kPa
Friction angle ϕ (in effective stresses): 19-35°
k (permeability coefficient): 10^{-6} to 10^{-7} m/s
Poisson ratio: 0.35

SPT results were between 10 and 60 blows, with the upper layer having SPT values lower than 40 blows and the lower layer having SPT values higher than 60.

DESIGN OF RETAINING WALLS

Static Conditions

The gravity type retaining structures were designed taking into consideration limit states (Eurocode 7, 1997).

The following ultimate states (with severe consequences) can occur: (i) loss of overall stability; (ii) failure of a structural element such as the retaining wall, an anchorage, a micropile, etc; (iii) foundation failure; (v) unacceptable leakage through or beneath the wall; (v) rotational failure; (vi) movements of the retaining structure which may cause collapse of other structures; (vii) unacceptable change to the flow of groundwater; and (viii) failure by sliding at the base of the wall.

The following serviceability limit states (with less severe consequences) can occur: (i) movements of the retaining structure which may affect the appearance or efficient use of the building and other structures; and (ii) excessive vibrations.
The values of partial factors for permanent and variable actions given in Table 1 were used for verification of ultimate limit states situations. For accidental situations all numerical values of partial factors for actions were taken equal to [1.0] (Eurocode 7, 1997).

Cases A, B and C have been introduced in order to ensure stability and adequate strength in the structure and in the ground.

Case A is only relevant to buoyancy problems, where hydrostatic forces are included in the main unfavourable action.

Case B is often critical to the design of the strength of structural elements involved in foundations or retaining structures. Where there is no strength of structural materials involved, Case B is irrelevant.

Case C is generally critical in cases, such as slope stability problems, where there is no strength of structural elements involved. Case C is often critical to the sizing of structural elements involved in foundations or retaining structures, and sometimes to the strength of structural elements. Where there is no strength of ground involved in the verification, Case C is irrelevant.

Permanent actions include self weight of structural and non structural components and those actions caused by ground, groundwater and free water.

In calculation of design earth pressures for Case B, the partial factors given in Table 1 are applied to characteristic earth pressures. Characteristic earth pressures comprise characteristic water pressures together with stresses that are admissible in relation to the characteristic ground properties and characteristic surface loads.

All permanent characteristic earth pressures on both sides of a wall are multiplied by [1.35] if the total resulting action is unfavourable and by [1.00] if the total resulting action effect is favourable. Thus, all characteristic earth pressures are treated as being derived from a single source.

For the verification of serviceability limit states, partial safety factors are used for all permanent and variable actions except where specified otherwise.

Table 1. Partial factors - ultimate limit states in persistent and transient situations (Eurocode 7, 1997)

Case	Actions			Ground Properties			
	Permanent		Variable	$\tan\phi'$	c'	c_u	q_u 1)
	Unfavour-able	Favourable	Unfavourable				
Case A	[1.00]	[0.95]	[1.50]	[1.1]	[1.3]	[1.2]	[1.2]
Case B	[1.35]	[1.00]	[1.50]	[1.0]	[1.0]	[1.0]	[1.0]
Case C	[1.00]	[1.00]	[1.30]	[1.25]	[1.6]	[1.4]	[1.4]
1) Compressive strength of soil or rock.							

Design values of ground properties, X_d, are derived from characteristic values, X_k, using the equation:

$$X_d = X_k / Y_m \tag{1}$$

where Y_m is the safety factor for the ground property, or shall be assessed directly.

The characteristic value of a soil or rock parameter is selected as a cautious estimate of the value affecting the occurrence of the limit state.

For serviceability limit states all values of Y_m are equal to [1.0].

The application of cases B and C for the east-west retaining wall is shown in Table 2.

<u>Numerical Analysis of the North-South Retaining Wall.</u>

For the design and behavior evaluation of the retaining walls, numerical analyses were performed using the FLAC code (Consortium, 1996b), with the following purposes:

- identification of plastic zones;
- computation of displacements distribution;
- distribution of stresses and strains in the retaining walls.

The analysis of the north-south wall was performed with the following phases:

- phase 1 - calculation of the ground initial stresses;
- phase 2 - introduction of the effect of the landfill and the retaining wall;
- phase 3 - introduction of the effect of Rossio station structure;
- phase 4 - introduction of two alignments of temporary anchorages for the retaining walls;
- phase 5 - introduction of the effect of the frames spaced 5.5m;
- phase 6 - introduction of the effect of the initial excavation phase;
- phase 7 - introduction of the effect of the final excavation phase;
- phase 8 - concreting of the gallery and deactivation of temporary anchorages;
- phase 9 – introduction of the effect of the 5 alignments of definitive anchorages;
- phase 10 – introduction of seismic actions.

The displacements distribution obtained for phases 9 and 10 are presented in Figs. 2 and 3.

<u>Slope Stability Analysis.</u>

The safety factor values obtained for the global slope stability analyses, by Janbu method, for the east-west retaining wall, are summarized in Table 2.

The analyses nº 1 and 2 were performed with a global safety factor while the analyses nº 3 to nº 6 were performed following the methodology proposed by Eurocode 7 (1997).

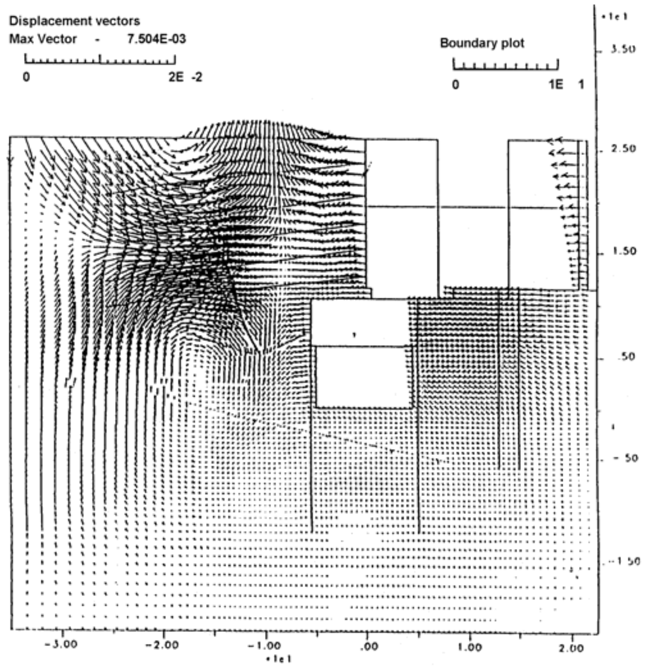


Fig. 2. Displacements distribution for phase 9 (adopted from Consortium, 1996b)

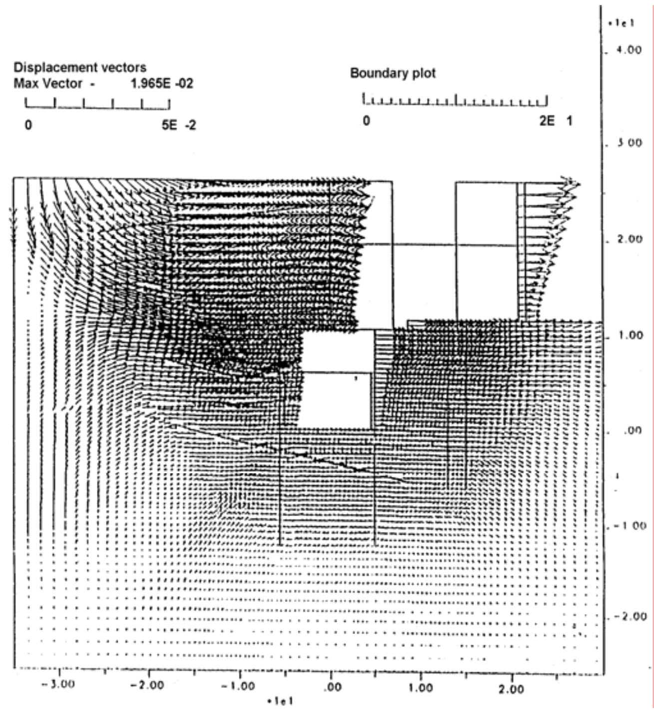
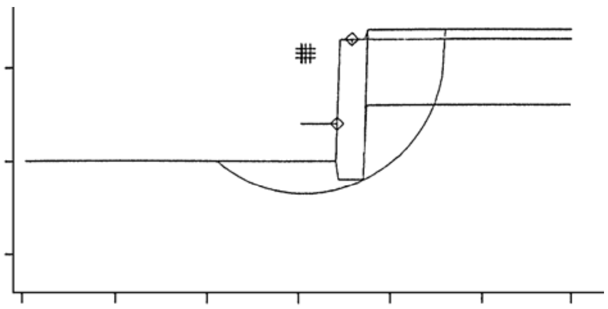


Fig. 3. Displacements distribution for phase 10 (adopted from Consortium, 1996b)

Table 2. Slope stability analyses (adopted from Figueiredo Ferraz, 1998)

Analysis	Type of analysis	Safety factor
1	Initial conditions	1.84
2	Pseudo-static for initial conditions	1.2
3	Case B for initial conditions	1.72
4	Case C for initial conditions	1.39
5	Case B with micropile reinforcement	1.76
6	Case C with micropile reinforcement	1.47
7	Pseudo-static with micropile reinforcement	1.44

The critical surface obtained for analysis 6 is presented in Fig. 4.



MODEL 6

Fig. 4. Critical surface for analysis 6 (adopted from Figueiredo Ferraz, 1998)

It is important to stress that for the east-west retaining wall the critical scenario was rotational failure of the wallthat required its reinforcement.

<u>Seismic Analysis</u>

The seismic action was based on the Portuguese Code (RSA, 1983) and defined by a stochastic gaussian stationary vectorial process (two horizontal orthogonal components and one vertical component). The Portuguese territory is affected by two seismotectonic sources: (i) near source which represents a moderate magnitude earthquake at a short focal distance with a duration of 10 seconds; (ii) far source which represents a higher magnitude earthquake at a longer focal distance with a duration of 30 seconds.

In Eurocode 8 the seismic hazard is described in terms of a single parameter, i.e. the value a_g of the effective peak ground acceleration in rock or firm soil called "design ground acceleration" expressed in terms of: (i) the reference seismic action associated with a probability of exceeding (P_{NCR}) of 10 % in 50 years; or (ii) a reference return period (T_{NCR})= 475 years. These recommended values may be changed by the National Annex of each country.

The earthquake motion in EC 8 is represented by the elastic response spectrum defined by 3 components.

The following factors can be listed to explain the behavior of gravity retaining structures during an earthquake: (i) increasing of dynamic earth pressures; (ii) variation of hydrodynamic pressure of the backfill; (iii) decreasing of stabilizing forces related with the weight of the structure; (iv) increasing of pore pressures and consequently reduction of effective pressures; and (v) soil liquefaction of backfill and/or foundation material.
The stability of soil foundation shall be assessed for the following conditions: (i) overall stability; and (ii) local soil failure.

For the pseudo-static analysis of rotating structures the seismic coefficients can be taken as (Eurocode 8, 1998):

$k_h = \alpha_{gr}\gamma_f S / g.r$ (2)

$k_v = \pm 0.5\ k_h$ when the ratio α_{vg}/α_{gr} is greater than 0.6 (3)

$k_v = \pm 0.33\ k_h$ otherwise (4)

Where α_{gr} is the reference peak ground acceleration, α_{vg} is the vertical component of acceleration, S is the soil parameter, γ_f is the importance factor of the structure and the factor r takes the values listed in Table 3.

Table 3. Factor affecting the horizontal seismic coefficient (Eurocode 8, 1998)

Type of retaining structure	r
Free gravity walls that can accept a displacement $d_r \leq 300\ \alpha$ S(mm)	2
As above with $d_r \leq 200\ \alpha$ S(mm)	1.5
Flexural r.c. walls, anchored or braced walls, r.c. walls founded on vertical piles, Restrained basement walls and bridge abutments.	1.0

In Rossio station, the following values were adopted: for the near source, k_h = 0.27 and k_v = 0.135; for the far source, k_h = 0.16 and k_v = 0.08.

The earth pressure coefficients were computed by the Mononobe and Okabe method.

ANCHORAGES

For the anchored structures the following additional limit states are considered (Eurocode 7, 1997):

- failure of the ground anchorage by tension;
- structural failure of the ground anchorage due to shear forces, distortion at anchorage head or corrosion;
- loss of anchorage load due to excessive displacements of the anchorage head or by creep and relaxation;

- failure or excessive deformation of parts of the structure due to the applied anchorage force.

For permanent ground anchorages (those which service life is greater than two years) protective corrosion barriers must be provided.

Experimental Tests

Three types of on-site anchorage tests are usually considered and were used in the case of the anchored reinforcement of the north-south wall of Rossio station: (i) investigation tests; (ii) suitability tests; and (iii) acceptance tests.

The lack of results of system tests to evaluate the long term behaviour of anchorages of the type which was intended to be used in Rossio station also required a test of this type to be performed in one anchorage. According to standard CEN EN 1537 "Execution of special geotechnical work - Ground Anchorages", a test of this type requires the excavation of the anchorage after the load protocol has been carried out, and so this anchorage had to be built outside the station.

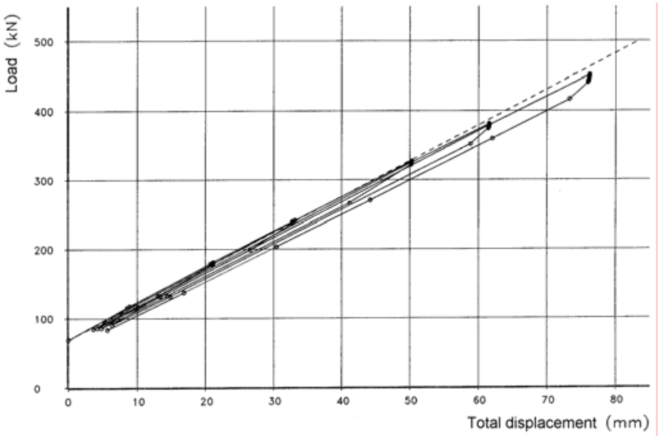


Fig. 5. Applied loads versus displacement for AN A14

Results of anchorage tests of three of the four types of tests mentioned above (all except acceptance tests) are presented in Table 5.

Investigation tests were needed to establish for the designer, in advance of the installation of the working ground anchorages, the ultimate load resistance in relation to the ground conditions and materials used.

Suitability tests were carried out to confirm the acceptable creep and load characteristics at proof and lock-off load levels, following the procedure recommended in the above mentioned CEN EN 1537.

For the determination of the anchorage characteristic load value R_{ak}, from R_{am} values measured in one or more suitability tests, a reduction factor was used to take into account the variability of ground and the constructive procedure. As minimum both conditions a) and b) from Table 4 were satisfied using equation:

$$R_{ak} = R_a / \zeta \qquad (5)$$

The calculation of the anchorage strength R_{am}, obtained from suitability tests considers the two modes of failure and the creep limit load.

The design value R_a, is given by equation**:**

$$R_a = R_{ak} / \gamma_m \qquad (6)$$

where γ_m = [1.25] for temporary ground anchorages and γ_m = [1.5] for permanent ground anchorages.

The total length of the permanent anchorages is 18m with a tendon free length of 9m and a fixed tendon length bonded to the ground by grout of 9m. The 6 steel cables tendon have a cross sectional area of 592.2 mm^2.

For each load test the anchorages were loaded in 4 incremental cycles from a datum load to a maximum test load, with measurement of displacements of the anchorage head. Displacement values due to creep were also determined.
The results of some of the anchorage tests are summarized in Table 5.

Figs. 5, 6, 7 and 8 show the applied loads versus anchorage head displacements, the applied loads versus elastic and permanent displacements, the displacement values versus time and the Ks values for the evaluation of creep, for the suitability test of anchorage AN A14.

Measurement of electrical resistance between an anchorage and surrounding soil or structure to determine the effectiveness of the applied corrosion protection system was also performed.

Table 4. Coefficient ξ for determination of R_{ak} (Eurocode 7, 1997)

Number of suitability tests	1	2	> 2
a) coefficient ξ applied to the mean value R_{am}	[1.5]	[1.35]	[1.3]
b) coefficient ξ applied to the minimum value R_{am}	[1.5]	[1.25]	[1.1]

Table 5. Anchorage tests

Anchorage	Max. Force (kN)	Max.Dis-placement (mm)	Permanent Displa-cement (mm)	Test Method-ology	Initial Free Length (m)	Calculated Free Length (m)	Ks máx. (mm)
AN S1	454.0	26.1	5.9	Syst. T.	4.73	6.73	0.48
AN EPC11	660.0	119.7	6.0	Inv. T.	21.73	23.97	1.10
AN EP3031	443.6	46.9	14.2	Inv. T.	21,57	17.17	0.48
AN EPC22	682.4	123.5	13.4	Inv. T.	21.41	22.47	0.44
AN A2	453.3	57.4	5.5	Suit. T.	15.63	16.36	0.38
AN A14	451.4	76.2	5.7	Suit. T.	21.63	22.24	0.59
AN A21	442.0	72.7	6.3	Suit. T.	21.63	22.51	1.09
AN A35	449.5	66.6	8.9	Suit. T.	21.63	19.60	0.82

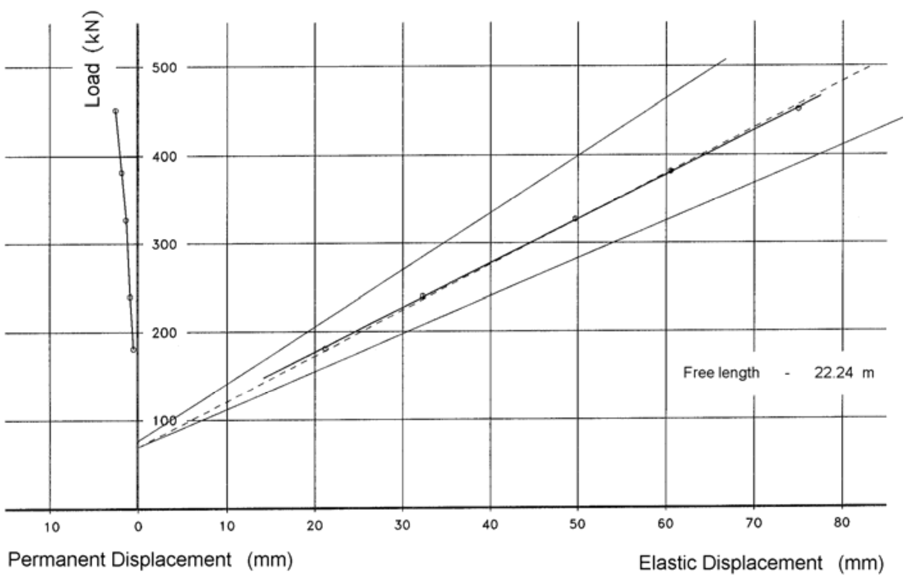


Fig. 6: Applied loads versus elastic and permanent displacements for AN A14

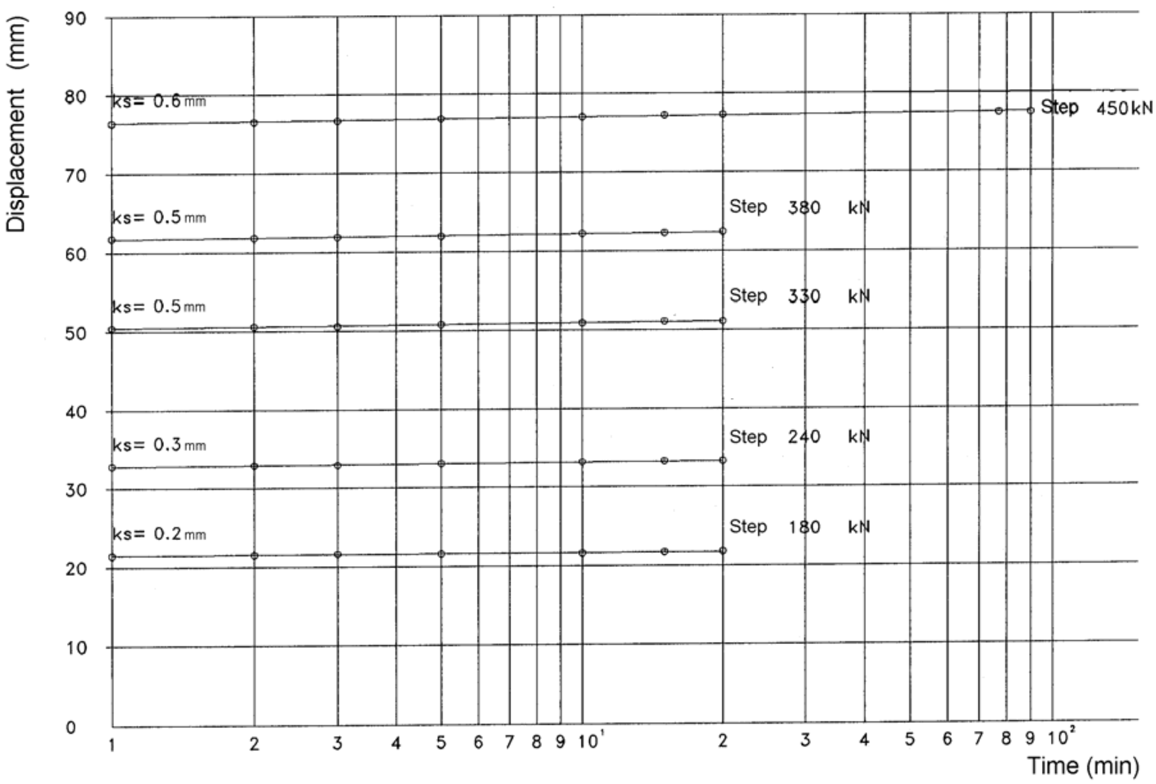


Fig. 7. Displacements versus time for AN A14

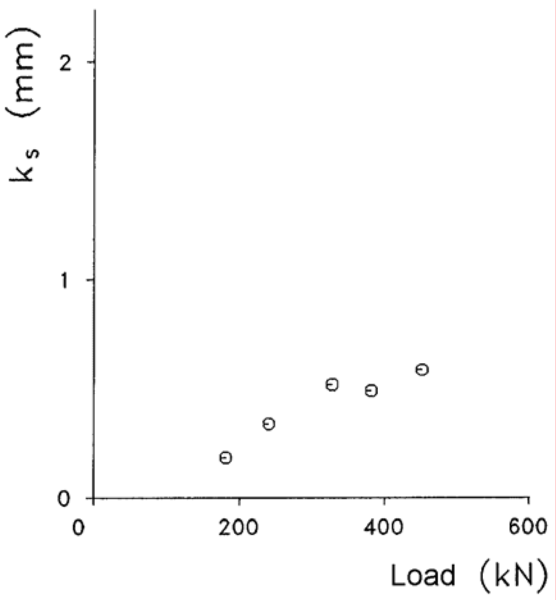


Fig. 8: Values of Ks for AN A14

MICROPILES

Introduction

The following limit states are considered in the design of deep foundations (Eurocode 7, 1997):
(i) loss of overall stability; (ii) bearing resistance failure of the pile foundation; (iii) uplift or insufficient tensile resistance of the pile foundation; (iv) failure in the ground due to transverse loading of the pile foundation; (v) structural failure of the pile in compression, tension, bending or shear; (vi) combined failure in ground and in structure; (vii) excessive settlements; (viii) excessive heave; (ix) unacceptable vibrations.

Experimental tests

The design of micropiles used in the reinforcement of the east-west wall has followed the method recommended by the Eurocode 7 (1997):

$$F_{td} \leq R_{td} \tag{7}$$

where:
F_{td} is the ultimate limit state axial design tensile load
R_{td} is the ultimate limit state axial design tensile resistance
For tension piles two failure mechanisms must be considered:
- pull out of the piles from the ground mass;
- uplift of the block of ground containing the piles.

The characteristic tensile resistance, R_{tk}, can be obtained from the results of pile tests R_{tm}, of one or several piles, taking into consideration the variability of ground and the construction features. Both conditions (a) and (b) of Table 6 must be satisfied.

$$R_{tk} = R_{tm} / \xi \tag{8}$$

Table 6. Factors ξ to derive R_{tk}

Number of load tests	1	2	>>2
a) Factor ξ on mean R_{tm}	[1.5]	[1.35]	[1.3]
b) Factor ξ on lowest R_{tm}	[1.5]	[1.25]	[1.1]

The design tensile resistance R_{td} shall be derived from:

$$R_{td} = R_{tk} / \gamma_m \tag{9}$$

where γ_m = [1.6].

Some micropiles of the lower levels of the reinforcement system of the east-west retaining wall, which are intended to work mainly under axial compression, were subjected to compression load tests of incremental loads, to reach a maximum value around 300 kN. In order to have a reaction for the test loads being applied, by means of a hydraulic jack, a special reaction structure had to be built, for each of the piles subjected to this type of test, composed mainly of a steel beam supported by two micropiles working in tension. Both the supported loads and the corresponding displacements (not greater then 5 mm) were within the values adopted in the design.

Tension load tests were also performed on micropiles, in incremental loads, in order to reach a maximum value around 300 kN. Table 7 summarizes some results of these tests.

Figs. 9, 10, 11 and 12 show the applied loads versus pile head displacements, the applied loads versus elastic and permanent displacements, the displacement values versus time and the Ks values, for the investigation test of micropile MP N45.

Table 7: Micropile tension tests

Micropile	Max. Force (kN)	Max. Displace -ment (mm)	Residual Displa-cement (mm)	Test Method-ology	Initial Free Lenght (m)	Calculated Free Lenght (m)	Ks máx. (mm)
MP N 1	288.5	12.0	1.0	Inv. T.	9.0	6.57	0.18
MP N10	289.5	12.7	1.9	Inv. T.	9.0	6.08	0.10
MP N45	298,8	10.3	0.4	Inv. T.	5.5	4.76	0.05

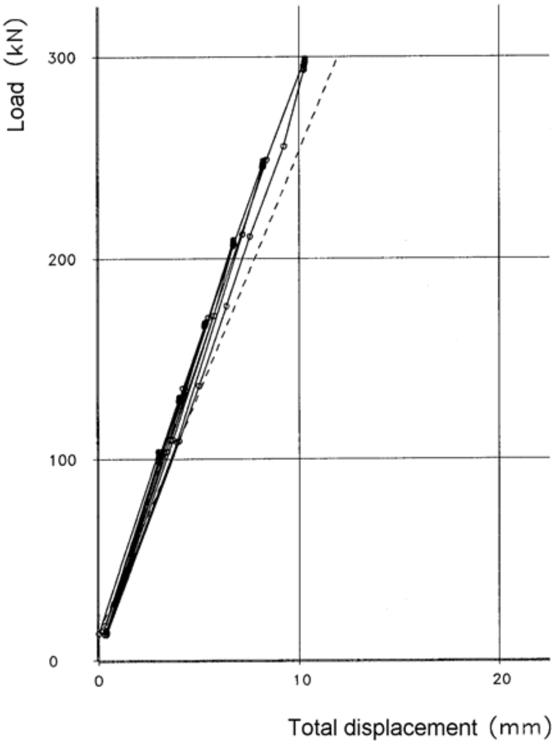


Fig. 9. Applied loads versus displacements for MP N45

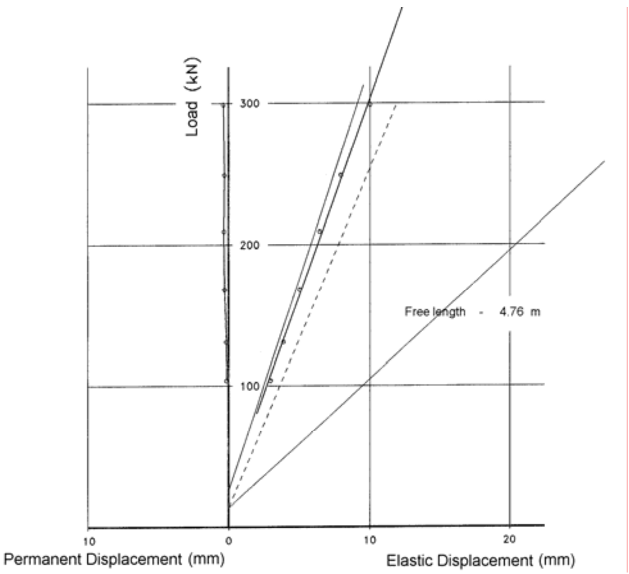


Fig. 10. Applied loads versus elastic and permanent displacements for MP N45

EVALUATION OF THE STATION BUILDING BEHAVIOUR

To study the effects of interaction of the underpinning works (including the already mentioned retaining walls) with the CP Rossio station building, a 3D linear finite element analysis, using SAP 90 code, to predict the maximum stresses and strain values, was performed (Consortium, 1996c).

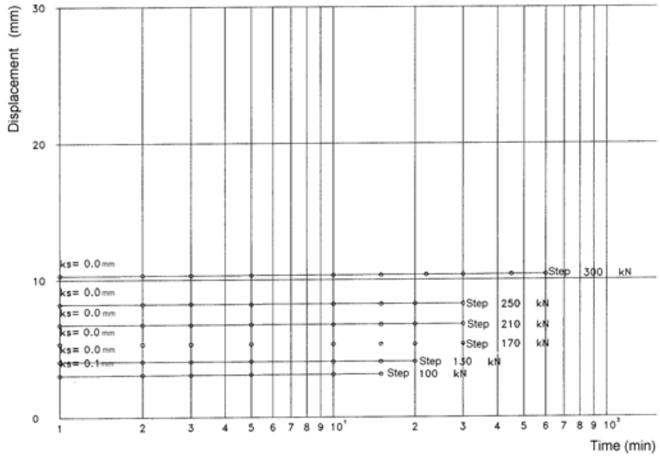


Fig. 11. Displacements versus time for MP N45

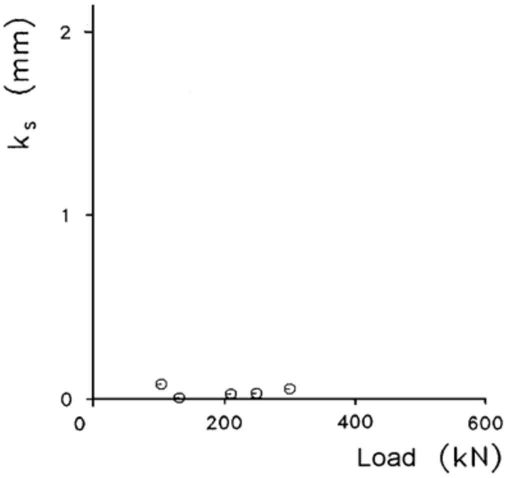


Fig. 12. Values of Ks for MP N45

It is important to remark that:

(i) The Rossio station building has a historical and patrimonial value; (ii) one important purpose was to minimize the damages, even the non structural ones; (iii) the predictable damages were extensive cracks on façades of stonework; iv) the rehabilitation works in this kind of cases are delicate and have high costs; (v) the negative impact on population due to the damages related with underground works would be relevant.

MONITORING AND ENVIRONMENTAL IMPACTS

To control the structure behaviour a monitoring system was installed, including the following types of instruments: clinometers, extensometers, surface movement points and inclinometers. Some anchorages were also instrumented with load cells.

To minimize the environmental impacts the following actions were taken: (i) protection of anchorage tendons and micro pile reinforcements against electric currents; (ii) control of vibrations due to drilling works.

ALFANGE LANDSLIDES CASE HISTORY

BACKGROUND

The slope of Alfange incorporates at the crest a medieval wall of historical value, at mid slope an access road and at its toe a small stream. Since 1883 there are records of Alfange slope instability with significant landslides in 1912, 1916 and the period of 1937 to 1941. The implemented actions have shown insufficient with further landslides in 1966, 1969 and 1979.

The slope is composed by sandy layers intercalated by clay and marl layers with vegetation cover at the crest (Fig. 13).

In general the slope is 25 to 30º (Fig. 14), with the exception of some sections with higher values of inclination.

The water concentration in the interfaces associated with the susceptibility of these materials are factors that have contributed for the clay softening and consequently have triggered landslides. In addition these clay materials are submitted to dry and wet cycles that will provoke new cracks. These factors are provoking a decrease of strength of clay materials. Some evidences of these occurrences are illustrated in Figs. 15 to 17. Due to these situations strong rehabilitation actions were implemented in the period 1986-1987 (first phase) and during 1992-1994 (second phase) with the construction of concrete retaining walls founded on vertical anchored piles and also an intermediate anchorage retaining wall and internal drainage (Fig. 18).

Due to the severe winter of 2000/2001 a new landslide has occurred in January 2001 (Figs. 19 and 20).

The implemented actions have shown to be still insufficient and there is a need to implement a global solution to assure the slope stability.

GEOTECHNICAL CHARACTERISTICS

A site investigation program was implemented for a better characterization and definition of geological and geotechnical properties (LNEC, 2002).

Within this framework 12 boreholes were performed with SPT tests 1.5m apart.

The laboratory tests have integrated identification tests (sieve analysis and Atterberg limits), and triaxial tests

The following results were obtained:
Fills (a_1)

c´(cohesion) = 0
ϕ`(friction angle) = 28º to 30º
E (elasticity modulus) = 8 to 10 MPa

Slope Deposits (a_2)

c´(cohesion) = 0
ϕ`(friction angle) = 28º to 30º
E (elasticity modulus) = 8 to 10 MPa

Alluvial deposits(b_1)

c´(cohesion) = 0
ϕ`(friction angle) = 32º to 34º
E (elasticity modulus) = 12 to 20 MPa

Sandy clay material (c_2)

c´(cohesion) = 0 to 10 kPa
ϕ`(friction angle) = 25 º
E (elasticity modulus) = 15 to 20 MPa

Silty clay with marls (c_4)

c´(cohesion) = 0 kPa
ϕ`(friction angle) = 32 º
E (elasticity modulus) = 18 to 25 MPa

Calcareous rock (c_1)

c´(cohesion) = 100 to 150 kPa
ϕ`(friction angle) = 40º to 44º
E (elasticity modulus) = 50 to 100 MPa

Marls with clays (c_1)

c´(cohesion) = 0 to 5 kPa
ϕ`(friction angle) = 32º to 34º
E (elasticity modulus) = 20 to 25 MPa

Sandy materials (c_3)
c´(cohesion) = 0 kPa
ϕ`(friction angle) = 28º to 30º
E (elasticity modulus) = 12 to 15 MPa

Coarse sands
c´(cohesion) = 0 kPa
ϕ`(friction angle) = 38º
E (elasticity modulus) = 35 to 50 MPa

IMPLEMENTED ACTIONS

After the failures that occurred in January 2001, the design of the stabilization works namely the design of retaining walls and slopes considered several hazards scenarios in order to minimize the occurrence of incidents and accidents during their expected life period, taking into consideration the necessary durability and reduction of maintenance costs (DGEMN-LNEC, 2004a; 2004b).

The following codes were taken into account: Eurocode 0-Basis of Design, Eurocode 1 Actions, Eurocode 7-Geotechnical Design (1997), Eurocode 8- Seismic Design of Structures (1998) and RSA – “Regulamento de Segurança e Acções”.

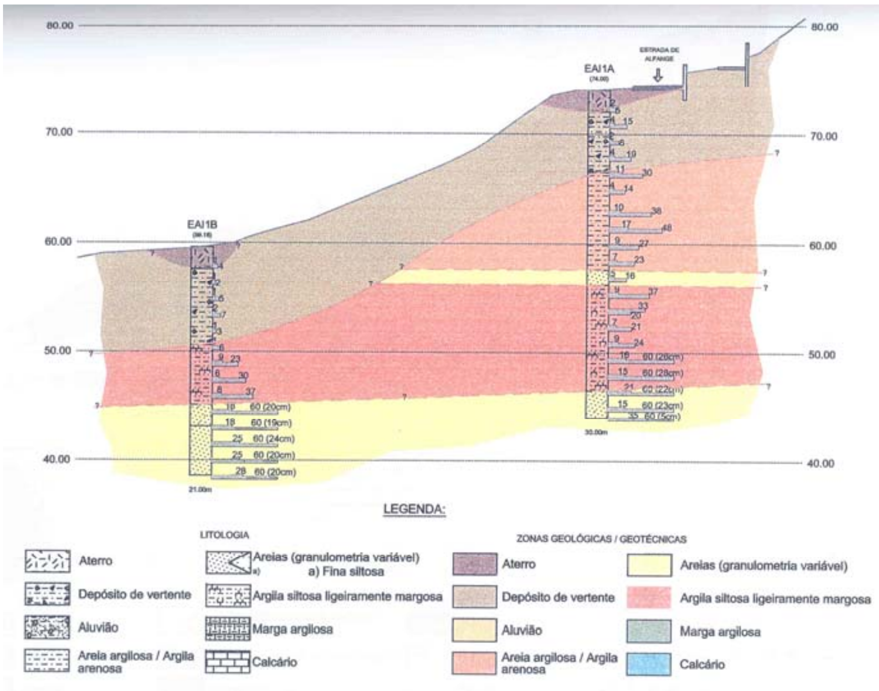


Fig. 13 –Geological Profile (adopted from LNEC, 2002)

Fig. 14 Inclination of the slope

Fig. 15- Landslide view

Fig. 16. Landslide view

Fig. 17. Occurrence of cracks

Fig. 18. View of the retaining wall

Fig. 19. View of the occurred failure

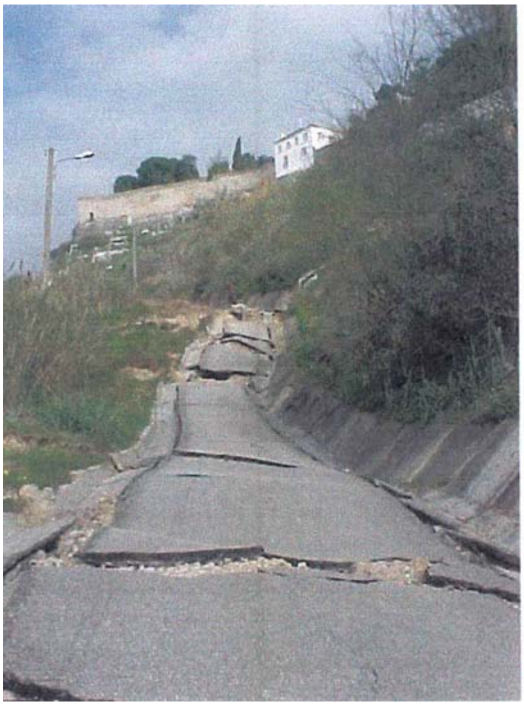


Fig. 20. View of the road failure

Within this framework the following actions were considered:

1) For the slope stability an approach that integrates (LNEC, 2003) was implemented:
— the maximum design phreatic surface.
—the bearing resistance of the soil;
— the failure by sliding at the toe;
— the failure by toppling;
—the global stability analysis.

Where ground or embankment material was relatively homogeneous and isotropic, circular failure surfaces were normally assumed.

For slopes in layered soils with considerable variations of shear strength, special attention was paid to the layers with lower shear strength and .non- circular failure surfaces were analysed. The minimum obtained safety factor was 1.52.

In order to assure a better internal drainage with the use of geodrains.

2) For the retaining walls the following design situations were considered (Eurocode 7, 1997):
— failure by rotation or translation of the wall or parts thereof;
— failure by lack of vertical equilibrium.

In cases where a combined failure of structural members and the ground could occur, ground-structure interaction was considered by allowing for the difference in their relative stiffness. Such cases include failure surfaces intersecting structural members such as piles and flexible walls.

The retaining wall with a thickness of 0.35m and pre-stressed anchorages of 500 kN, and 4 m apart, was founded on piles with 0.8m of diameter and 20 m long (Tecnasol, 2001).

The construction phase has incorporated the following steps:

i) Execution of the platform and construction of micropiles;
ii) Execution of the first section of the wall and partial pre-stressed of lower level of anchorages;
iii) Construction of the wall, execution of the 2nd level of anchorages and application of anchors load of 500 kN for the anchorages of the upper level and application of 300kN load for the lower level of anchorages;
iv) Execution of the intermediate level of anchorages and application of pre-stressed of 500 kN.

The details of the construction phasing are shown in Fig. 21.

The slope view after the reinforcement works is shown in Fig. 22.

The results of the anchorages tests performed by LNEC to calibrate the design values are shown in Figs. 23 and 24.

3) The stability of the bottom of an excavation was checked in relation to the design pore-water pressure in the ground and hydraulic failure. Heave of the bottom of deep excavations due to unloading was considered.

4) Deviation of the river, filling of the banks with rockfill material placed on geotextile and placement of geosynthetic material near the left bank abutment. Cleaning of solid wastes transported by the river.

5) Design and works near the river bank in order to avoid the regressive erosions of the slopes (LNEC, 2004).

Due the space limitations, for the initial section 20 m long gabion retaining wall founded on Reno mattress was adopted and in the other sections a trapezoidal section with Reno mattress with 0.30m thick and founded on a geotextile material was used.

For the definition of maximum flow flood 3 methodologies were used, namely Giandotti, Rational formulation and Mockus formulation. Considering a return period of 100 years the average obtained flow value was 4.55 m3/s (INAG, 2004).

A superficial drainage system was also implemented.

For a better dissipation of the water energy 44 steps were adopted.

A view of the works that were executed at the toe slope is shown in Fig. 25.

6) Due the existence of archaeological vestiges adequate treatment of zones of excavation.

7) The implemented rehabilitation solution has taken into account an adequate paisagist integration.

8) For a better understanding of the triggered mechanisms the installed monitoring equipments including namely bench marks, inclinometers, piezometers, clinometers and anchor load cells were measured with adequate frequency considering higher frequency during the winter time.

9) Regular visual inspections, in order to detect the instability zones, unexpected settlements, occurrence of cracks, with the purpose to implement in due time of mitigation actions.

10) Cleaning and maintenance of the superficial drainage system.

11) Adequate maintenance of the slope vegetation cover.

12) Emergency actions to evacuate the people if the interpretation of the monitoring instruments would show an increase of the instability rate or in case of other severe detrimental evidences detected by the visual inspections (CSOPT, 2003).

INSTRUMENTATION

The following devices were installed in 8 profiles (LNEC, 2005): 12 bench marks, 22 inclinometers, 10 piezometers, 28 clinometers, 1 crackmeters, and 16 anchorages.

The location of these equipments is given in Fig 26.

The frequency of the readings was defined with more readings during the wet season.

Although there were no signs of instability of the landslide some punctual situations were detected: (i) some instrumented anchor load cells have shown loss tension values around 15%; (ii) also for EAI2C inclinometer it was noticed the occurrence of a local sliding for depths higher than a 7,5m.

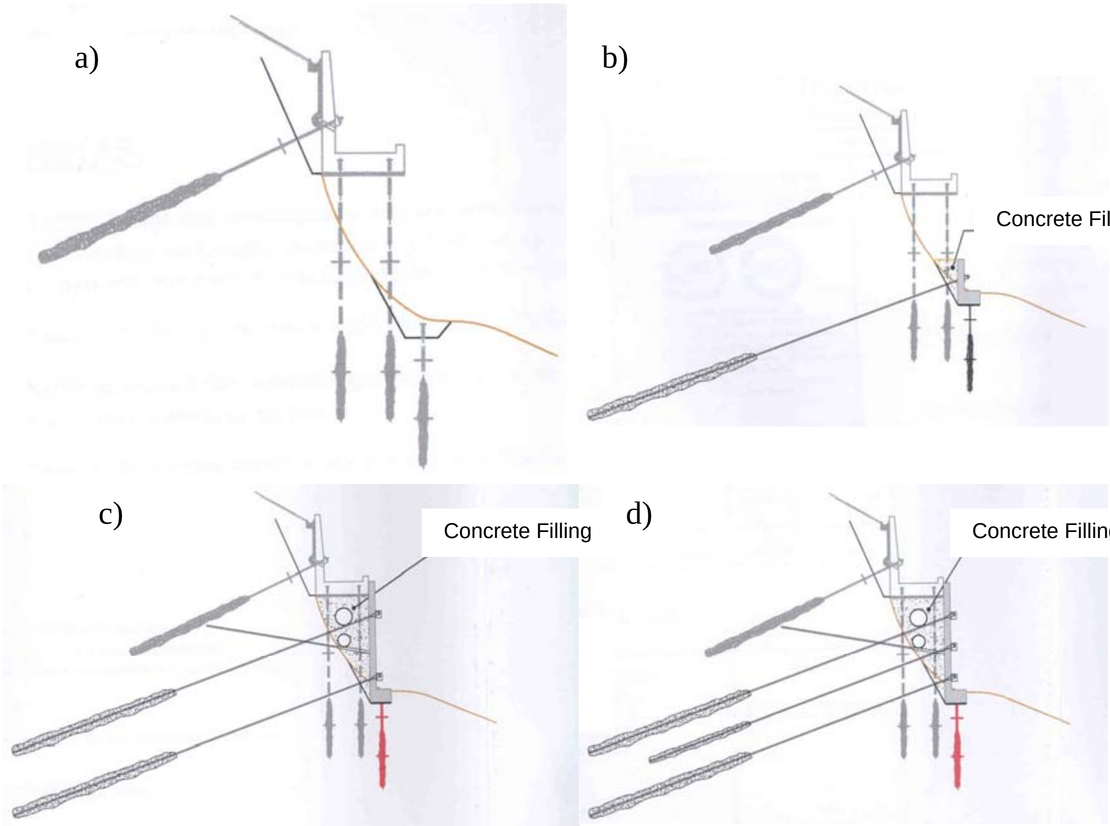


Fig. 21. Construction Phasing (adopted from Tecanasol, 2001)

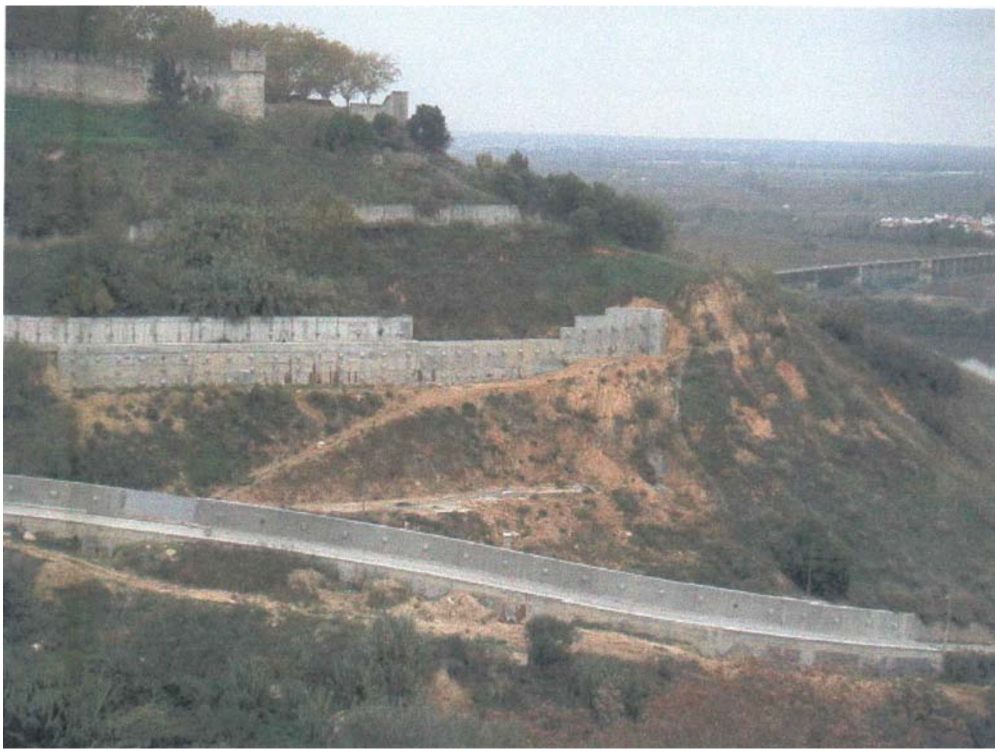


Fig.22. Actual view of the slope

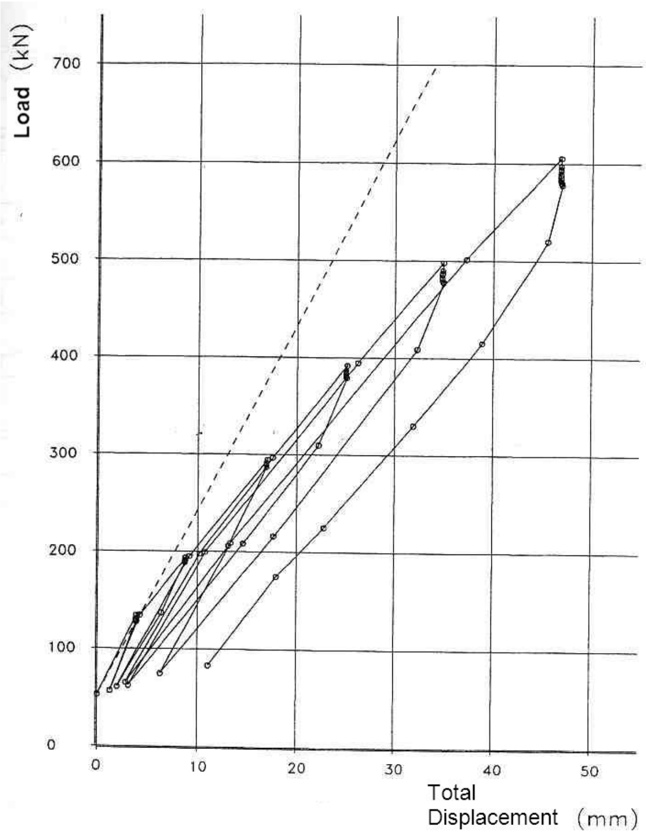
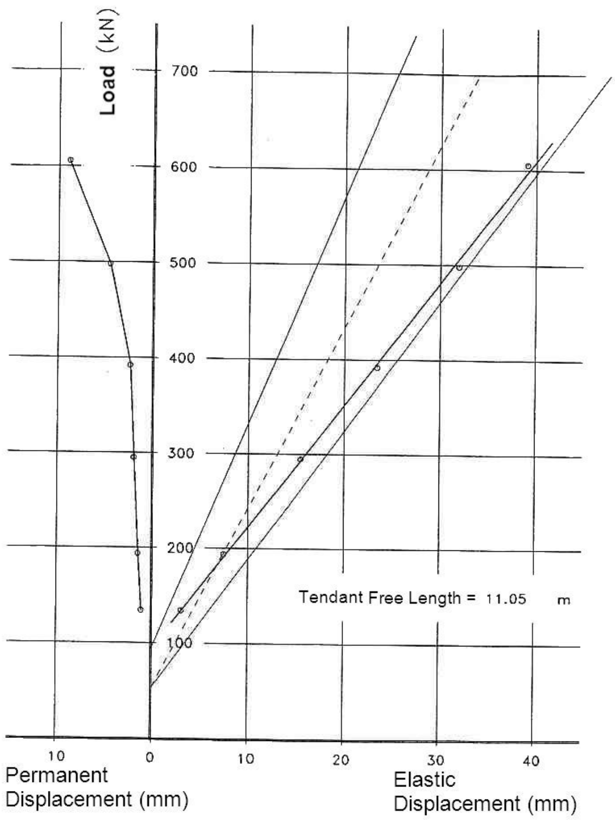


Fig. 23. Anchorage detailed tests



Fig, 24 Anchorage detailed tests

Fig. 25 Works performed at the slope toe

CONCLUSIONS

The following conclusions can be drawn:

Rossio Railway Station

a) The selected underpinning solutions allowed the construction of the underground gallery for the Lisbon Metro and excessive damages in the station building and in other structures were avoided. This solution enabled the use of the railway platforms of the station without any major constraints.

b) The numerical analyses with simulation of the construction phases allowed the calculation of the stresses and strains distributions and also the identification of the plastic zones. The analyses of the alternative solutions were of great importance to select the one which was retained in each zone.

c) The monitoring of the structures during construction allowed the safe implementation of the solutions including the purposes of maintaining most of the station functionality.

d) The need to reinforce both main retaining walls against rotational failure arose from the lack of safety conditions for this scenario which was identified during the design analyses for the underpinning of the station.

e) The tests performed for the anchorages and micropiles of the reinforcement works of the two main walls supported the design of these works and allowed an adequate quality control.

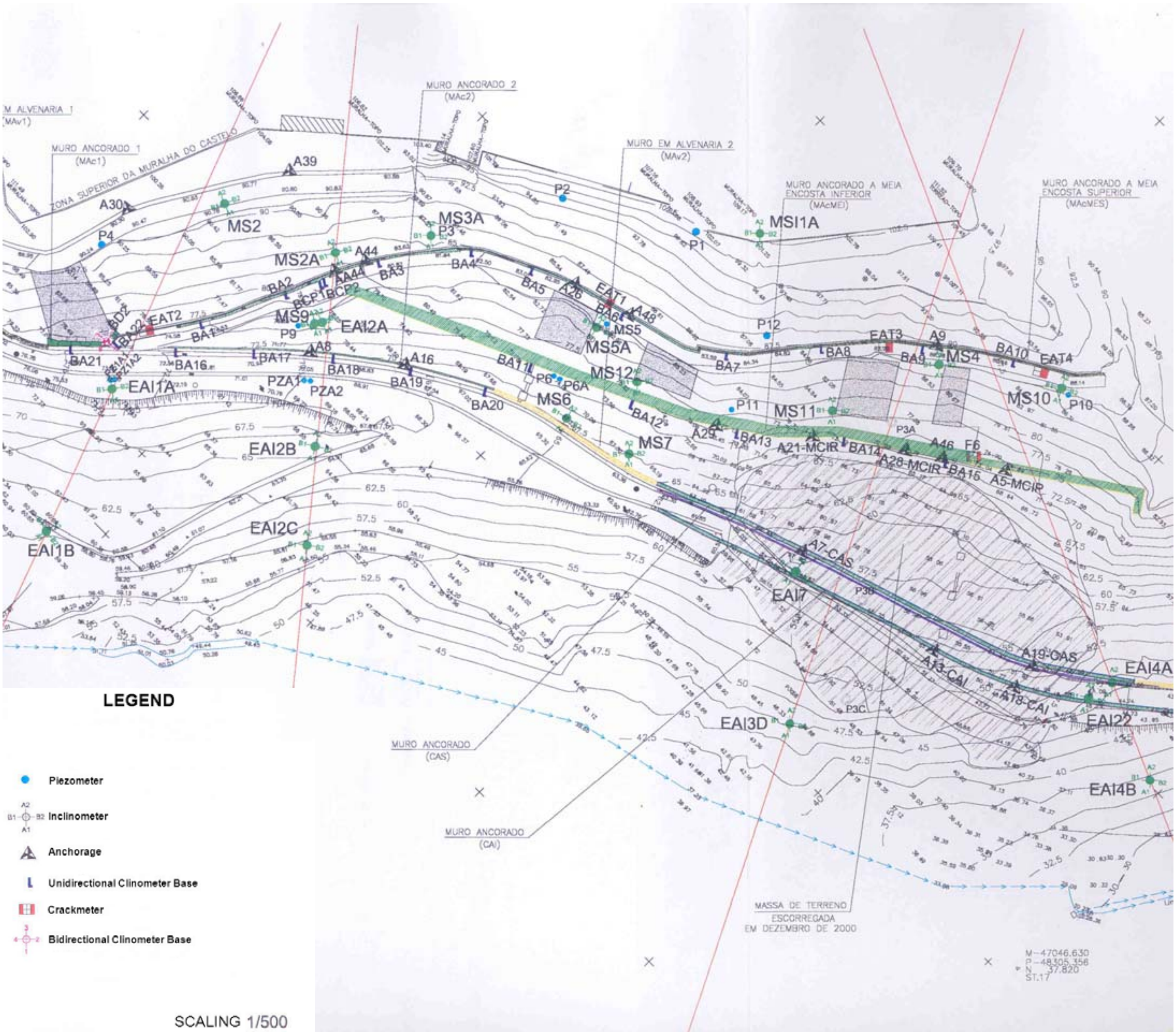


Fig. 26. Plan of instrumentation devices (adopted from LNEC, 2005)

Alfange Landslides

i) As the implemented actions to stabilize Alfange slope have shown to be insufficient a global solution to assure the slope stability was developed

ii) The different hazard scenarios were incorporated in the slope stability analyses and retaining wall design.

iii) To avoid regressive erosions of the slopes, modification of the river geometry and reinforcement works of its banks were performed.

iv) To calibrate the design values, anchorages tests were performed.

v) To assess the slope stability and retaining walls behavior a monitoring plan was established.

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