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Experimental investigation on an instrumented reinforced earth wall

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ABSTRACT: The paper deals with the behaviour of an instrumented reinforced embankment during its construction and the first years of its life, checked with a vertical inclinometer, a series of benchmarks on the abutment, a wide number of electric strain gauges on the reinforcing elements, and a few corrosion testing rods.

1 INTRODUCTION

The paper deals with the case of an overpass, built between 1982 and 1984. The road, bridging the railway beneath, is composed of two embankments, linked by a three-span bridge (fig.1). The overpass is almost composed of reinforced earth. The total length of two halves is about 500 m and the height varies from 0 to 10 m. The bridge beams are supported by reinforced earth, by means of a bankseat. The abutment are defined towards the railway by a vertical facing, composed by suitably joined precast concrete panels, measuring 1.50x1.50 m.

With regard to the foundation soil (fig.2), there are alternating layers of clayey silt and silty clay, with thin intercalations of fine silty clay, from ground level to the depth of 20 m. Cohesive soils present medium plasticity and medium high consistency.

2 SHEAR STRENGTH PROPERTIES OF THE FILL

The soil used as granular fill was a sandy gravel slightly silty with a few cobbles (fig.3). The fraction finer than 0.42 mm was not plastic. The soil was classified as A.1.A. according to AASHTO Classification System, and as GP according to Unified Soil Classification System.

An AASHTO modified compaction test was carried out in order to determine the 'optimum' water content and the 'maximum' dry unit weight of soil: these were equal to 5.3% and 23.8 kN/m^3 respectively (fig.4). Then a series of triaxial compression tests were carried out to evaluate the shear strength properties of the compacted granular fill. These tests were performed with isotropic consolidation and drainage was permitted.

The triaxial samples were first compacted with a rubber membrane around them, in a specially developed split mold, clamped together and fitted around a pedestal. The mold diameter and height were re-

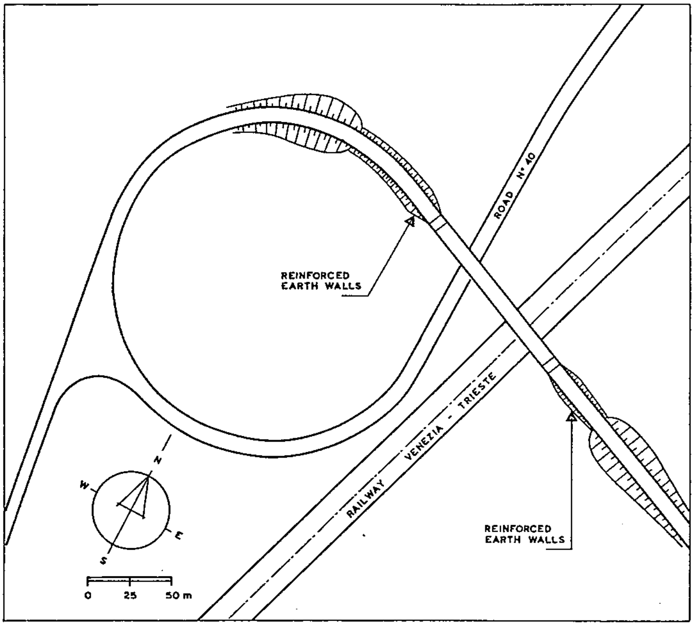

Fig.1. Site plan.

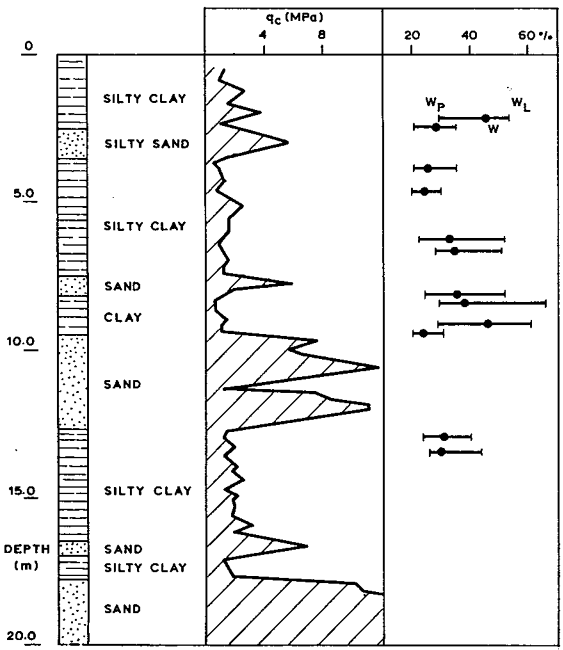

Fig.2. Foundation soil.

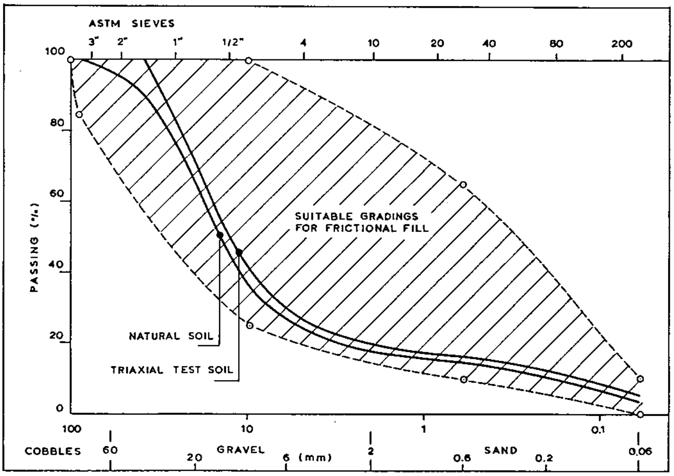

Fig.3. Grain size distribution range of suitable frictional fill.

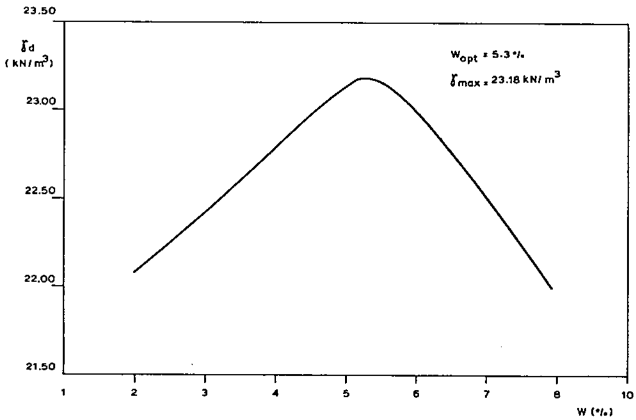

Fig.4. Modified AASHTO compaction curve.

spectively of 200 and 410 mm. Energy equal to that used for AASHTO compaction test was chosen. After compaction the mold was removed and a second thick rubber membrane was rolled down over the specimen; this because the original membrane was often punctured during compaction by the sharp edges of the particles. Both membranes were sealed by additional o-rings at the top and the bot-tom. To complete the setup the initial specimen height and diameter size were taken.

The perspex chamber of the triaxial cell was then put in place, the cell was sealed and, lastly, moved to its test position in the loading frame. After the cell had been filled with water, an isotropic confining pressure was applied. Four triaxial compression tests, with increasing confining isotropic pressure (30, 50, 100, 150 kPa) were

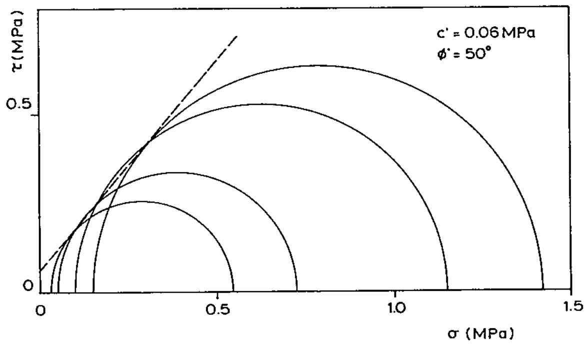

Fig.5. C.I.D. triaxial results.

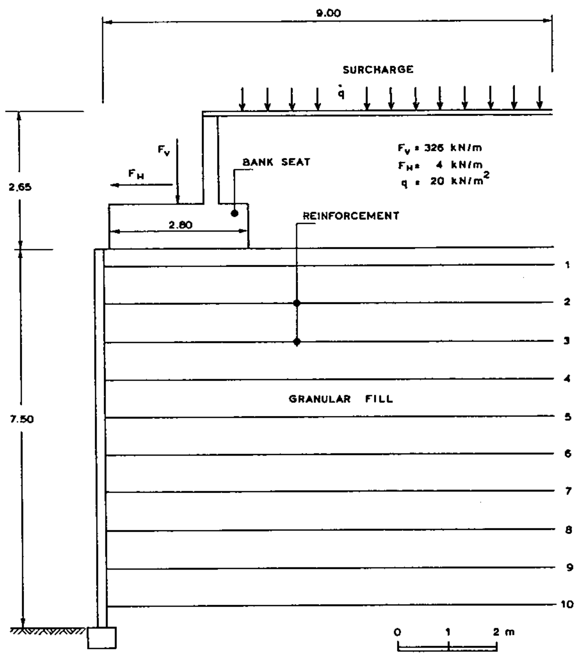

Fig.6. Cross-section of the abutment.

performed. Each specimen was sheared in the compression apparatus with a vertical deformation rate of 0.15 %/min, equal to a vertical displacement of 0.6 mm/min. The average dry unit weight of four compacted specimens was 23.18 kN/m^3, with a range from 23.05 to 23.24 kN/m^3.

The shear strength angle ϕ, calculated for each test assuming that the cohesion value was equal to zero, decreased as the confining pressure was increased. The failure curve, enveloping the Mohr's circles, was significantly not linear (fig.5). However the apparent cohesion could have been actually greater than zero, because of particles interlocking or cementation. For the stresses evaluation in the reinforcing elements, a shear strength angle of 50° and an apparent cohesion of 0° were assumed.

3 INSTRUMENTATION AND MEASUREMENTS

The following considerations refer to the smaller of the two embankments, leading to the bridge. In fig. 6 the main characteristics of the abutment section are summarized. The behaviour of the embankment during its construction and the first few years of its life was controlled by means of the following instruments:

a. an inclinometer, placed vertically inside the embankment, 1 m far from the facing, to a depth of 15 m from ground level;
b. a series of benchmarks, placed on the abutment, for topographic measurement of absolute and differential settlements;
c. three series of electric strain gauges, placed on three reinforcing elements, at different heights;
d. a few corrosion testing rods, for checking corrosion in the reinforcing elements.

Periodically checking of the inclinometer allowed absolute and relative displacements to be measured, between the top and the base of the embankment, and along the shallower fifteen meters. Apart from displacements observed during construction, maximum relative horizontal displacements of 15 mm were measured from the top and the bottom of the embankment. In depth horizontal displacements of less than 20 mm were observed.

Abutment settlement was controlled with special care during the first two years after completion of the embankment. A series of benchmarks placed on a cross-section perpendicular to the road axis near the abutment was also periodically checked. Settlement trends fitted oedometric design expectations. Two years after construction, an average settlement of about 250 mm was observed, nearly equal to 90% of the predicted primary consolidation (fig.7).

The greatest value for the vertical differential settlement, between two benchmarks five meters apart, was of 20 mm, with a distortion equal to 1/250, less than 1/100 assumed by Sclosser (1972) as maximum allowable distorsion for structures with concrete facing panels.

Electric resistance strain gauges for measuring the stresses developing on the reinforcing elements were installed on a cross-section of the embankment, 12 m far from the abutment (fig.8). Three characteristic depths were chosen, one shallow (+4.40 m from the original ground level), one medium (+2.90 m) and one deep (+1.40 m), and electric strain gauges were placed on the corresponding reinforcing elements. In particular the gauges were placed at various distance from the vertical facing, three on the upper strip, four on the middle one and three on the deep one.

This control equipment had two aims: to experimentally verify the distribution of tensile stresses along the reinforcing strips, and to compare the values obtained with those derived from simple design methods. The gauges were bonded in pairs to the top and the bottom of

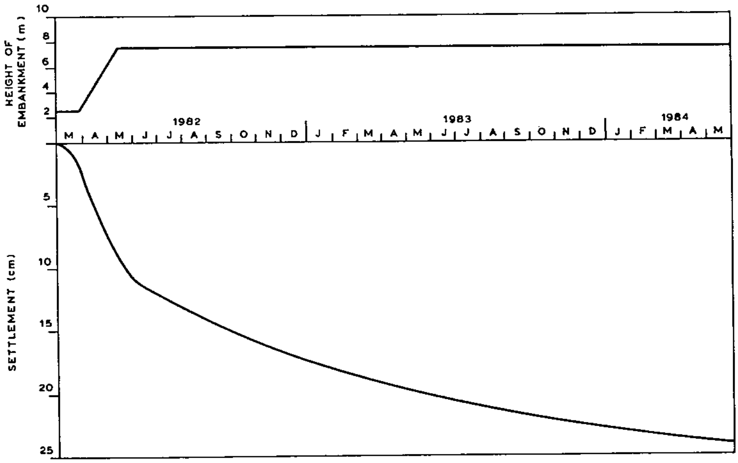

Fig.7. Average settlement of the vertical facing.

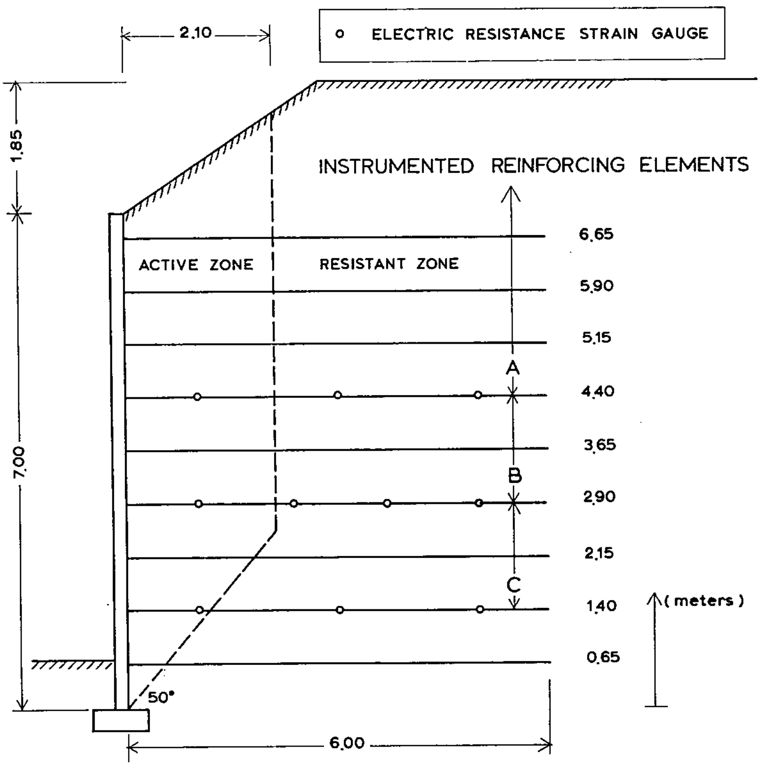

Fig.8. Scheme of the instrumented embankment section.

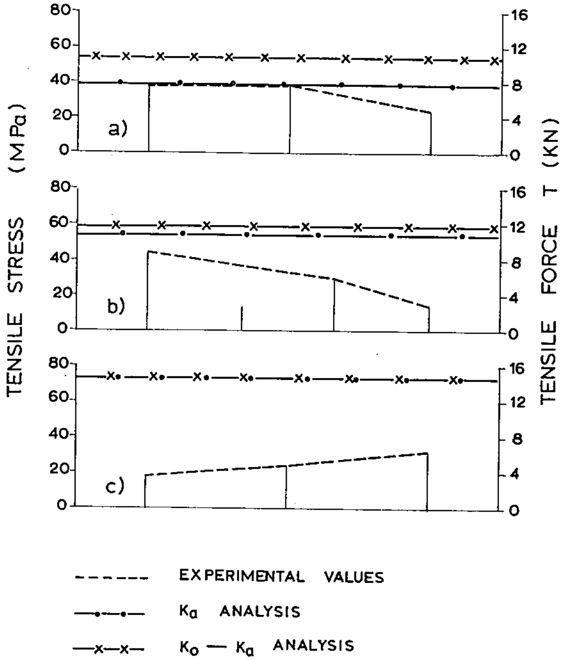

Fig.9. Comparison between theoretical and experimental stresses acting in the instrumented reinforcing elements.

the reinforcing elements to permit separation of axial and bending tension. During installation, short plastic tubes were positioned over each pairs of gauges to prevent the stresses from fill being applied directly to the bonding agent attaching the strain gauges to the element. The space between the inside of the tube and the surface of the element was packed with form-rubber to prevent soil infilling.

The distribution of axial tension on the instrumented reinforcing elements are presented in fig.9, in comparison of the theoretical stresses determined using 'Tie-back (or K_a)' and 'Coherent gravity (or K_a-K_0)' analyses (Jones, 1985). The strains on opposing sides were averaged to show axial tension. The values of the peak tension appeared to be not dependent on strip depth. This conflicts with the conventional design approaches in which tension increases in direct proportion to the depth of the fill, for constant element spacing. Stresses induced by compaction could have had a dominating influence and resulted in much higher locked-in-stress than those produced by the fill weight, according to Murray and Hollinghurst (1986). Furthermore the vertical rod, used to connect all the reinforcing elements in a vertical profile, could have been sufficiently stiff to cause a redistribution of the tensile forces.

The line of maximum tension could not be located. Anyway apart from the deepest reinforcing element, the greatest value of the axial stress was recorded at the point near the facing.

The simple scheme, used for the calculation of the minimum axial stress and corresponding tensile force on each reinforcing element, gave values greater than experimentally recorded ones. This could be due to a not completely satisfying behaviour of the strain gauges equipment or to the extremely conservative values provided by the simple calculation scheme. Only on the shallow element A theoretical values matched with experimental ones; proceeding downwards such differences tended to increase more and more.

Considering the shallow reinforcing element it was possible to check the average mobilized coefficient of the soil-strip interface friction f^* on the basis of the gradient between the peak tension and the rear of the element, where the tension must be equal to zero:

(1) $$T = 2 \times b \times L_r \times p_v \times f^*$$

with:

T (tensile force) = 0.8 kN;

b (strip width) = 0.04 mm;

L_r (resistant length) = 3.5 m;

p'_v (effective vertical pressure) = 100 kPa

The following value of f^* was so determined:

(2) $$f^* = 8/(2 \times 0.04 \times 3.5 \times 100) = 0.29$$

equal to 25% of the maximum estimated allowable value, tan ϕ = tan 50° = 1.2, proposed by Schlosser, Long (1974) for ribbed strips, failing direct measurements.

Lastly corrosion checking rods, 600 mm long, were positioned during inspection of the structure in June 1984. In April '88 a couple of them were removed to check the trend of the corrosion. The weight loss just was of 0.6% of its original weight. The top surface was more oxidized than the bottom one; this fact is certainly due to the possibility that the drops of water remain for a longer time on the top surface rather than on the bottom one. Comparing the loss weight with the total amount of the zinc used for protecting the rod the coating thickness reduction is determinable. Initial thickness of 115 μm decreased to 97 μm in four years, according to the following relationship between coating loss thickness P (μm) and time T (years):

(3) $$P = (2.8 \div 25)\ T^{0.65}$$

proposed by Darbin et Al. (1988), after having performed electro-chemical tests on strips buried in partially saturated soils.

CONCLUSIONS

The experimental investigation carried out on a reinforced-earth embankment has led to the following conclusions.

1. Laboratory triaxial tests performed on big samples of the compacted granular fill are very useful to evaluate the fill shear strength parameters and to estimate the apparent soil-strip interface friction coefficient.
2. Concrete panel facing has substantially maintained its verticality with a maximum horizontal displacement of 15 mm; settlement of the abutment fitted the design expectations and the maximum distorsion of 1/250 did not cause any problem to the outward appearance.
3. Tensile stresses recorded in three different reinforcing elements generally were less than those calculated with conventional design methods.

4. Stresses induced by compaction appeared to have a dominating influence and resulted in higher locked-in-stress than those produced by the fill weight.
5. The mobilized apparent soil-strip interface coefficient was of 0.29, equal to 25% of the maximum available value.
6. Corrosion checking rods, positioned inside the fill and removed after four years, showed not serious oxidized marks: weight loss was equal to 0.6% of its original weight.

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