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Construction and performance of an experimental large scale wall reinforced with geosynthetics

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ABSTRACT: The construction, instrumentation and performance of a vertical test wall reinforced with geosynthetic are described. The wall, 5 m high, was built in Northern Italy using a geocomposite having tensile strength of 27 kN/m and a strong cohesionless sandy to gravelly soil from quarried quartz porphyry. Geosynthetic elongations and displacements at wall face were monitored during construction, subsequent loading up to 84 kPa and unloading, as collapse did not occur. Measurements under maximum surcharge are reported and briefly discussed. They suggest that little strength was mobilized by the reinforcements and that the behaviour of the wall was affected by the stiffness of the loading system.

1 INTRODUCTION

During the past thirty years the design of reinforced soil structures and the prediction of their behaviour under the loading conditions expected during the design life have been extensively investigated through theoretical as well as experimental analyses. Despite the relatively large amount of contributions presented sofar in the technical literature, the assessment of stability, serviceability and durability of geosynthetic reinforced soil walls is still a matter of debate.

Most of the procedures availaible to design this type of structures are focused, in fact, on limit state conditions and have generally been applied to rather typical solutions, in which good cohesionless backfill materials are combined with stiff or medium stiff reinforcements, extending well beyond the most critical failure surface to provide enough pull out resistance. Fewer data have been presented instead on reinforced soil walls constructed using strong and well compacted soils, such as crushed rocks, combined with low strength, though comparatively stiff, reinforcements having truncated lengths or viceversa using poor backfill materials and high strength reinforcements.

While contributions on serviceability limit state design are still limited, results of model and full scale tests, though extremely valuable, are often affected by uncertainties that make them difficult to interpret.

In the attempt to contribute to designing model and full scale tests of geosynthetic reinforced soil structures and understanding their behaviour when loaded in situ, the construction, instrumentation and performance of a 5 m high test wall, built in Northern Italy between March-April 1995 on behalf of the Provincia Autonoma di Trento, are described in this paper.

2 SITE CONDITIONS AND WALL GEOMETRY

The test site was situated few kilometers North of Trento, in a wide flat area which stretches essentially in the North-South direction along the Adige river valley. Due to road access to the site and space availability, the test wall was located some 500 m far from the East bank of the river, which flows in a North to South direction close to the steep mountains bounding the river plain on the West side.

Based on data of previous explorations it was estimated that the original soil profile within a large area encompassing the test site consisted of interbedded layers of normally consolidated sands, silty sands and clayey silts, extending to great depths. In 1990 in fact, site conditions were modified after the ground surface was raised an average of about 5 m with placement of a granular fill. This fill was constructed using a locally available well graded crushed quartz porphyry, with particles reaching a maximum size of 0.6 m, placed in lifts and densely compacted with vibratory rollers. Data on the properties of the fill material are quite meagre and limited to the results of several plate load tests performed with a circular plate, having a diameter of 0.6 m. Test results indicated values of the plate settlement modulus in the range 45 to 98 MPa.

Being controlled by the river level the ground water table at the test site was estimated to fluctuate between about 1 to 3 m below ground surface. Since no records of pore pressure measurements were available at the time the test wall was constructed it is not known if placement of the fill and later of the test wall resulted in consolidation settlements during the experimental program.

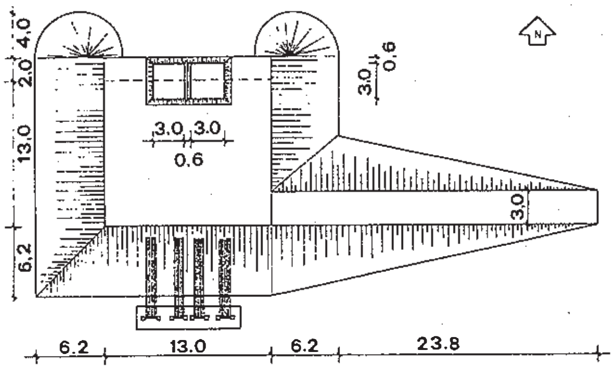

Figure 1. Plan view of the Trento test wall.

The experimental program was defined on the basis of numerical analyses, performed using data on material properties obtained from laboratory tests (De Col et al 1995). With due consideration of available materials, space and financial resources as well as of construction and loading procedure, the final layout shown in Fig. 1 was eventually selected.

In the attempt to ensure stresses and strains distributions as close as possible to those in plain strain conditions without increasing construction costs excessively, the geometry of the test wall was designed with two adjacent sections: an unreinforced one on the South side and a reinforced section, with vertical facing and measuring 5 m high by 13 m long by 2 m wide, on the North side. Support of both sections was provided by dumping the same construction material at some 40° to the horizontal along the East and South sides and by placing it at about 9° on the East side where an access and construction ramp was located. At the top of the wall two additional lifts, 0.4 and 0.3 m thick, were placed to ensure anchoring of the topmost reinforcing layer and provide two rather deformable loading platforms, each 3 by 3 m wide.

Surcharge loading was applied by means of cast iron ingots, each weighing 4.75 kN and having a length of 3.6 m and square cross section with sides of 0.13 m. The iron ingots were placed on each of the two loading platforms and stacked in cross lifts.

3 CONSTRUCTION MATERIALS

The reinforced and unreinforced sections and the dumped sides and access ramp were all constructed from the same crushed quartz porphyry, delivered from a nearby quarry. The quarried material, generally well graded, had particles ranging in size between 0.1 and 600 mm. The larger boulder and cobble size elements were often flake shaped while gravel size and smaller particles were mostly angular or subangular. Laboratory measurements of the specific gravity of the soil solids, from crushed grains, indicated an average value of 2.64.

For construction of the reinforced section however, a selected material, prepared at the quarry site by removal of particles greater than 50 mm from the parent crushed rock, was used to avoid damaging the reinforcements and to obtain a relatively uniform well compacted backfill. Typical grain size distributions of the selected material are presented in Fig. 2a.

The non scalped non selected quarried product was used instead throughout the remaining zones.

Density and strength of the selected material after compaction were investigated from laboratory tests on reconstituted samples prepared after additional scalping to remove any particle greater than 10 mm. Results of Standard Proctor tests yielded an optimum water content of 7.7% and corresponding maximum dry density of 20.4 kN/m^3. Data on shear strength were obtained from isotropically consolidated drained triaxial tests on 70 mm diameter samples, loaded to failure under confining pressures in the range 50-300 kPa. Samples were prepared using material moistened to an average water content of 5% and placed inside a mould to form 25 mm thick layers which were tamped untill a dry unit weight of 17.4 kN/m^3 was attained. The results of such tests are presented in Fig. 2b. On the Mohr-Coulomb plane the material exhibited a rather markedly non linear behaviour. An estimation of average values of strength properties within the range of applied confining pressures indicated c'=100 kPa and ϕ'=40° (Benigni 1995).

The reinforcing geosynthetic, produced by Polyfelt of Linz (Austria) under heading PEC 50/25 and supplied in rolls 3 m wide, was a geocomposite consisting of a nonwoven polyester sheet on which woven polypropylene threads were knitted at constant spacings of 10 mm and 15 mm in the production and transversal directions respectively.

Properties of the geocomposite were determined from laboratory tests performed at ENEL S.p.A. CRIS in Milano. The melting temperatures (ASTM D 3418) of the nonwoven polyester sheet and of the polypropylene threads resulted 262 °C and 164 °C respectively. Measurements of mass per unit area (ISO 9864) and nominal thickness (ISO 9863) indicated values of 309 g/m^2 and 2.31 mm respectively. The maximum tensile strength in the transversal direction (R_t), determined from wide width tests performed according to EN ISO 10319 (Cazzuffi 1996), averaged 27 kN/m (Fig. 2c), thus confirming the data given by the manufacturer indicating values of tensile strength R_p = 50 kN/m and R_t = 25 kN/m in the production and transversal directions respectively. During the tensile tests no significant Poisson's ratio effects were observed.

4 CONSTRUCTION AND INSTRUMENTATION

The reinforced section of the wall was constructed in lifts separated by the geocomposite layers and having a final height of 0.5 m. Backfill material was placed

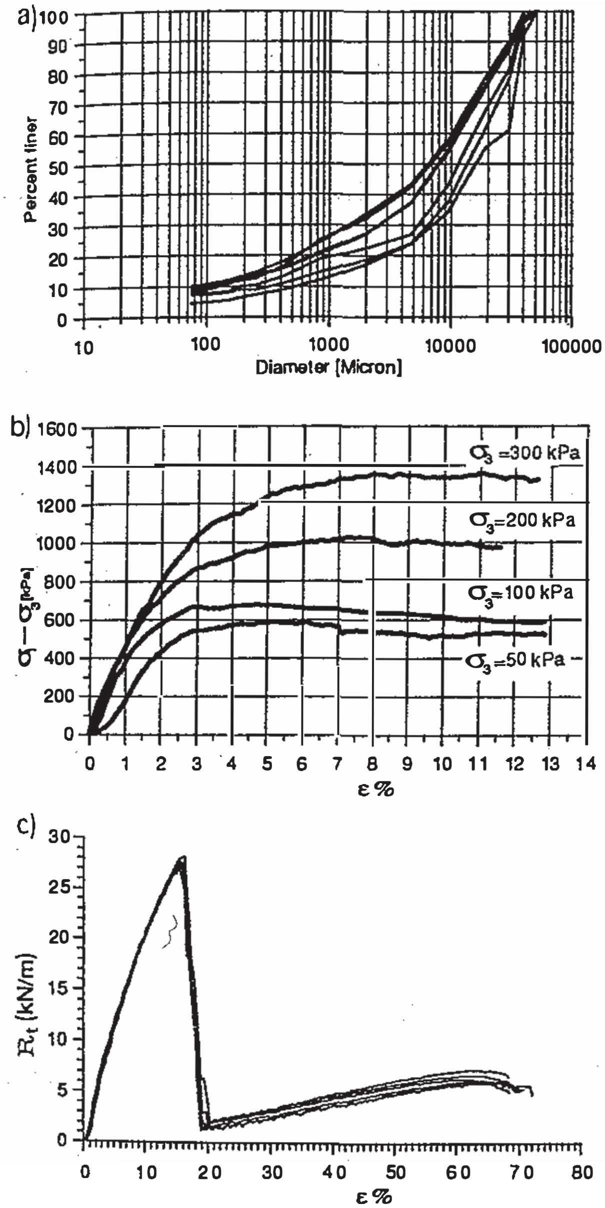

Figure 2. Properties of construction materials: a) typical grading curves of fill; b) results of triaxial tests on scalped material; c) results of tensile tests on geocomposite specimens loaded in transversal direction.

manually in two strata, each one reaching a height of 0.3 m and compacted untill its thickness reduced to 0.25 m. To this purpose a hand operated vibrating compactor having static weight of 736 N was used.

During construction the wall face was supported by a 1 m high wooden form, assembled with wide long boards nailed to brackets, which was wedged against a temporary scaffolding. Sufficent reaction was provided with vertical wooden poles, stuck into the ground at close distance from the wall and fixed to rakers and steel pegs. At any time during backfilling the wooden form was set in place to support the lift being placed and the one immediately below.

Upon completion of each lift the underlaying geosynthetic was wrapped around at the face and extended 2 m inside the backfill. The new reinforcing layer was then unrolled parallel to the wall face and positioned so that a 0.5 m long tail rested on top of the one already wrapped around, while the remaining 2.5 m long part draped over the wooden form. No windrows were used to anchor the reinforcements which were simply resting on each other along the 0.5 m overlap distance. Also no overlapping was necessary in the direction parallel to the wall face, since fabric was delivered in rolls of total length greater than 13 m. With the lowest geocomposite placed directly on the foundation soil a total of 11 layers of reinforcement were placed to reach final grade at 5 m above ground.

Placement and compaction of fill in the reinforced section was controlled through field density measurements carried out along six of the ten total lifts. The resulting dry unit weight ranged between 19.6-20.4 kN/m^3 with in situ water contents falling in the interval 5.5%-2.4%. One measurement on material from a loading platform yielded a dry unit weight of 19.6 kN/m^3 and corresponding water content of 3.9%.

To monitor the performance of the reinforcements during construction and surcharge loading, the elongations of the geocomposite sheets were measured at different locations. Measurements were taken using 2 mm diameter steel cables bolted only to the nonwoven polyester sheet and threaded backward to a simple though effective reference system. After the cables had been bolted at the site and the geocomposites unrolled, they were exited from the slope of the wall on the South side and guided through holes drilled in wooden panels, fixed vertically at close distance from the base of this slope. The cables were then draped along the wooden panels through small pulleys, and kept taut with 10 N iron ballasts fixed to the tails and capable to provide enough tension. Movements of each cable were measured by visual reading the position of a pointer fixed to the ballasts against a 1 mm ruler nailed to the wooden panels (Fig. 3).

For purpose of protection and to minimize friction at the soil-cable interface, all cables were housed inside thin PVC tubes. The behaviour of the PVC tubes in the field was checked through laboratory compaction and compression tests, performed with a tube and cable placed inside a soil sample, to verify that no permanent deformations occurred and that the steel cable would still slide freely with negligible friction.

To provide close information on the response of the different geocomposite layers a total of 280 cables were installed on the 9 reinforcing layers at heights between 0.5 and 4.5 m above base of wall. Odd numbered layers were equipped with 4 groups of 10 cables each; the other layers carried 2 groups of 10 cables each. Cables were grouped at constant interval of 0.2 and 0.1 m in the directions perpendicular and parallel to the wall face respectively (Fig. 4a).

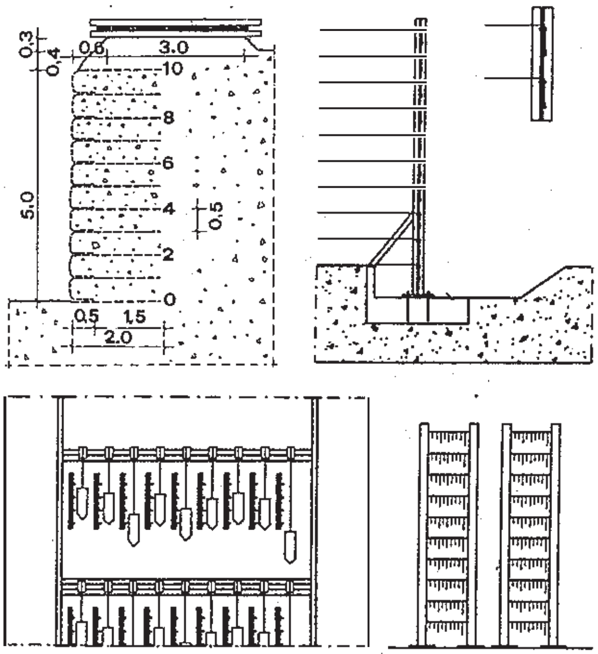

Figure 3. Cross section of wall through the reinforced section and sketch of reference system used to measure reinforcements elongations.

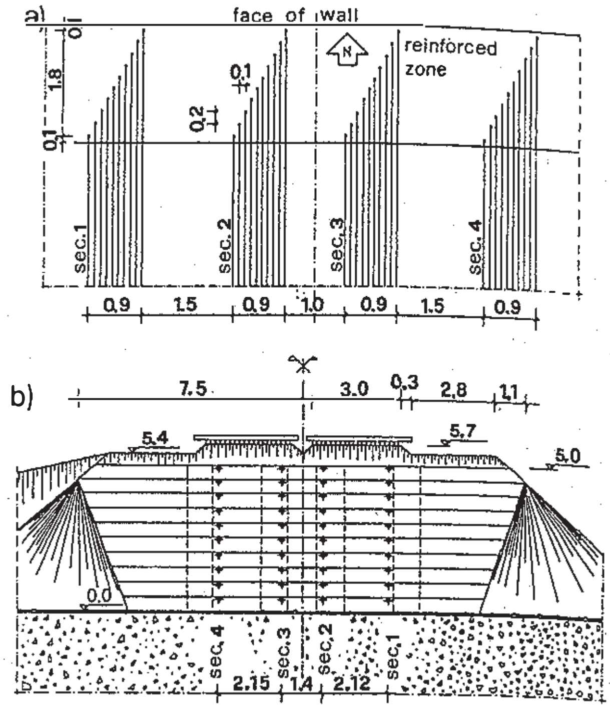

Figure 4. Layout of instrumentation: a) plan view of cables installed on odd numbered reinforcement layers; b) front view of test wall with reference plates to measure displacements at the face.

In addition to elongation measurements, wall movements were monitored by measuring the displacements of 40 reference plates which were fixed at mid layer onto the surface of the geocomposites exposed along the vertical facing. Such displacement measurements, which did not begin untill end of construction, were obtained along 4 vertical sections (Fig.4b) with conventional precision topographic surveys extended well beyond the test area.

5 PERFORMANCE AND RESULTS

Upon completion of the test wall and with the top lift and the loading platforms placed and compacted, the iron ingots were delivered at site and lifted directly from trucks onto the loading platforms.

The maximum surcharge loading, reached after 51 hours, was estimated at 84 kPa from the weight of the stacked ingots evenly distributed on the two 3 by 3 m wide loading platforms. Because of the length of the ingots the loaded area extended beyond the reinforced section onto the unreinforced one, towards the South side of the wall.

Reinforcement elongations measured under maximum applied surcharge are presented in Fig. 5 together with values of horizontal displacements at wall face, obtained from the topographic survey. It is worth noting that no significant elongations were recorded in the lower half of the test wall. Also relative movements between adjacent cables were generally negligible suggesting that very little strength was

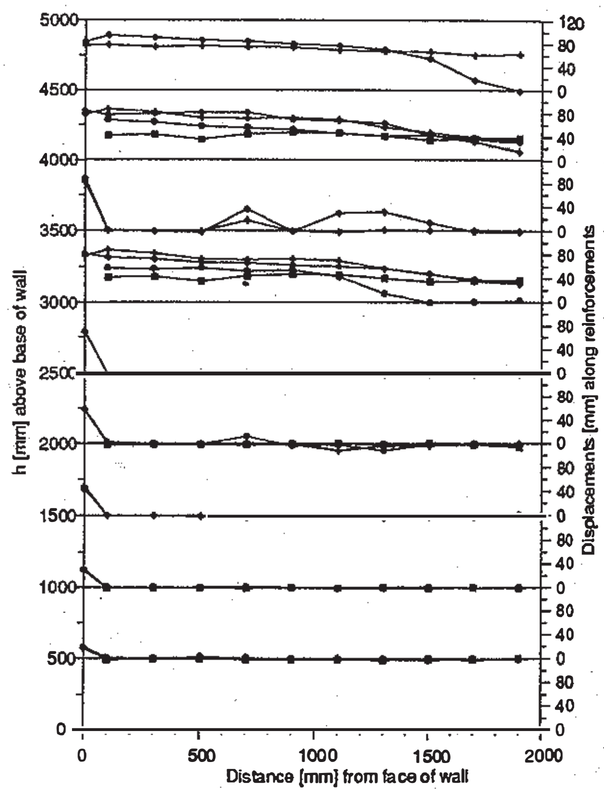

Figure 5. Reinforcements elongations under the maximum applied surcharge loading.

mobilized by the reinforcements.

Horizontal and vertical displacements at the wall face, measured during application of surcharge loading at plates along section 4 are presented in Fig. 6. Interesting enough the maximum horizontal movements occurred upon unloading and at some 2 m below the loaded area. Given the data shown in Fig. 5 it may be inferred that the top part of the wall moved rigidly outward and that horizontal displacements at the top were somewhat restrained by the stiffness of the loading system, iron ingots and loading platforms. At the base instead horizontal movements and lateral spreading were probably impeded by the lowermost reinforcement and the comparatively stiffer foundation fill.

A more regular trend is shown by the vertical settlements which differed little along the wall. In this respect it should be pointed out that Fig. 6 was plotted assuming that neither settlements nor horizontal displacements occurred at the base of wall in the underlying foundation soils and fill. While it is likely that horizontal displacements during construction and loading were negligible, yet it is seems reasonable to expect that the foundation fill sustained some settlements. An indication of such settlements is given in fact by the sudden increase of the slope of the settlement versus depth curves close to the wall base.

Finally the vertical and horizontal displacements and the lateral spread along the wall face under the maximum applied surcharge are summarized in Fig. 7, where data have been plotted as a function of the distance from section 4. Again a tendency for the soil to expand is observed at the top of the wall while maximum boulging occurred somewhat below. Furthermore data on horizontal displacements and lateral spread show that the real distribution of stresses and strains differed from plane strain conditions.

Upon unloading and subsequent demolition of the wall the backfill appeared free of cracks with the geosynthetics intact and the steel cables still firmly bolted and sliding freely. However at the back of the loading platforms, a tension crack running almost parallel to the wall face was observed; such cracking suggests again that the stiffness of the loading system might have affected the general response of the wall.

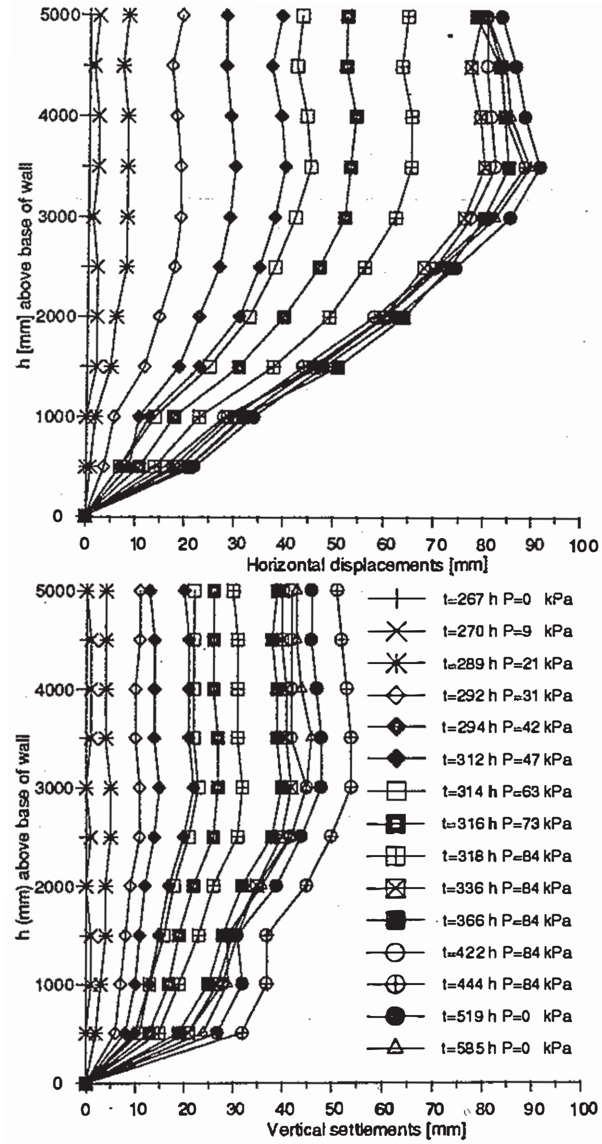

Figure 6 - Displacements at wall face along section 4 measured at different times during loading and after unloading (time = hours from begin of construction).

6 CONCLUSIONS

The results of the experimental program described herein, though quite extensive, do not allow to draw some definite conclusions on the behaviour of geosynthetic reinforced earth walls, constructed combining strong fill materials with comparatively weaker reinforcements.

With regard to serviceability and limit state conditions it is worth noting that while it is undoubtful the wall did not collapse under the applied load, though somewhat large movements were recorded, it is questionable a serviceability limit state was reached. Despite most of the horizontal and vertical displacements were not recovered upon unloading, it appeared in fact the wall itself had sustained almost no damage. Also, based on the recorded displacements the maximum values of vertical and shear strains along the face of the wall of about 1.5% and 0.5% respectively were obtained. Larger vertical strains were determined only close to the base of the wall; however, as stated previously they are affected by the settlements of the foundation fill, for which no data were available. Given the above values of strains it is doubtful that the wall was loaded even close to failure.

Values of elongations measured along the different geocomposite layers suggest that the uppermost part of the wall, about 1.5 m thick, moved almost rigidly forward and also that very little strength was mo-

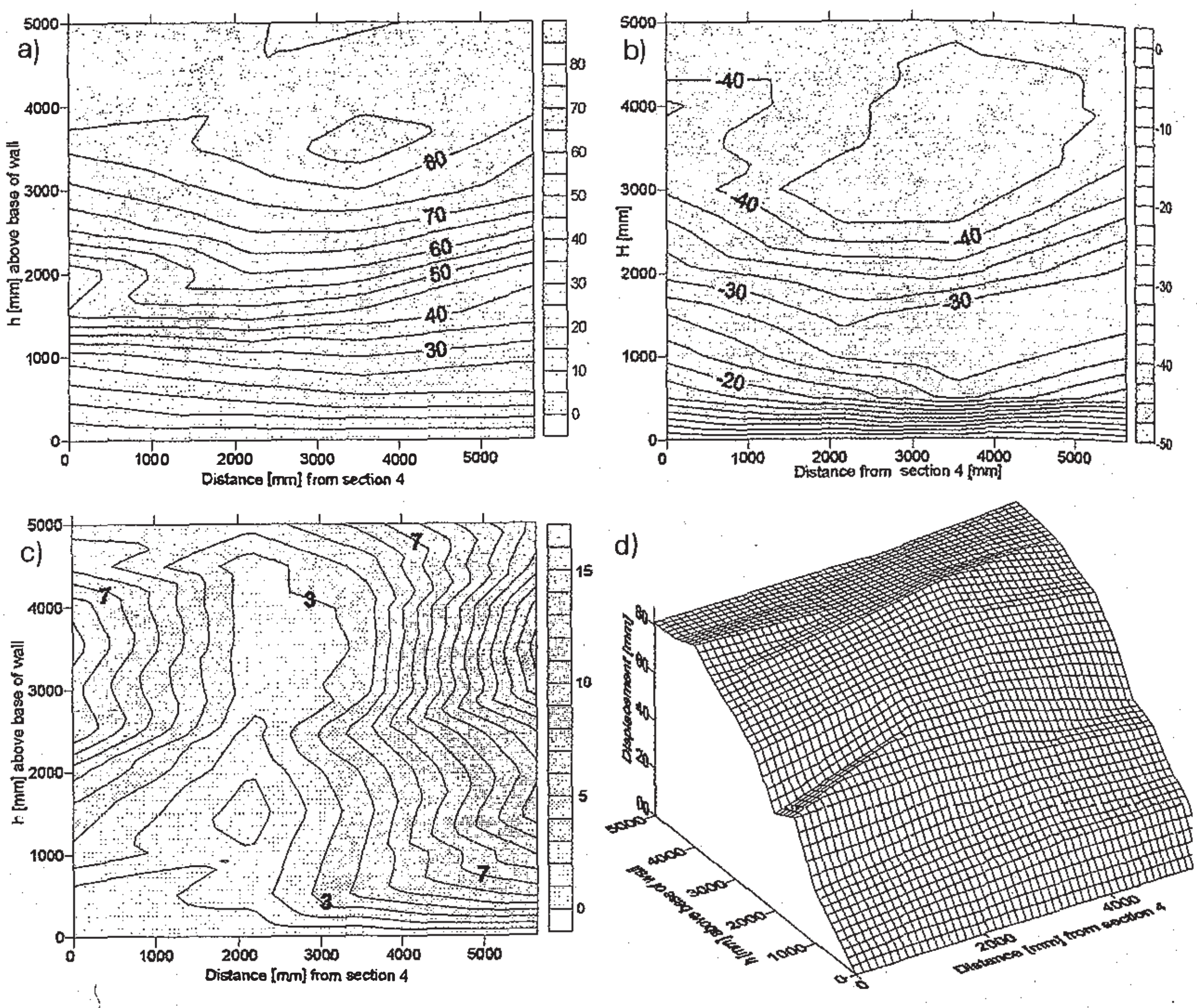

Figure 7 - Pattern of displacements (mm) at wall face as function of distance from section 4: a) horizontal movements; b) vertical settlements; c) lateral spread; d) three dimensional representation.

bilized by the reinforcements. In this respect it cannot be neglected, however, that in most cases values of the measured elongation were rather small and probably close to the level of accuracy of the measuring system itself.

It should also be mentioned that the stiffness of the loading system certainly affected the pattern of displacements as well as the stress distribution within the reinforced and unreinforced sections

Finally the influence of partial saturation and compaction procedures on the strength and deformation properties of the fills were not investigated during the experimental program.

It is believed that the above mentioned factors should be well considered when designing model and full scale tests for purpose of modeling the behaviour of this type of reinforced earth structures.

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