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Full scale test on a retaining wall with non-uniform reinforcements

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ABSTRACT: The paper deals with the results of a full scale test on a wrapped-face retaining wall reinforced by two types of reinforcement materials embedded in sand. Such technique, which alternates a stiff and a relatively soft reinforcement materials, reduces the total cost of the wall but has to maintain an adequate stability as well as an acceptable deformation level. To evaluate the complex behaviour of such structure, a 3.6 m high retaining wall was constructed and instrumented, then loaded to failure; the main gathered results are herein described. Their interpretation contributes to the evaluation of whether the current limit equilibrium analysis method can be applied to non-uniformly reinforced soil structure or if a more sophisticated approach which takes into account the different linear stiffness of the reinforcements should be used.

1. INTRODUCTION

The use of reinforcements with different mechanical properties (i.e. strength and stiffness) allows engineers to obtain more economical designs, as they can benefit from positive aspects of both materials, thus reducing the cost of reinforcement while maintaining adequate internal stability. However, as pointed out by Hatami et al. (2001) the use of reinforcement layers of different deformability may give rise to load concentrations in the stiffer layers that must be carefully checked in order to avoid unforeseen overstressing of the reinforcements.

To the present this kind of mixed structures have been the subject of numerical analysis to investigate their structural response, but according also to Hatami et al. (2001), no reliable results are available in the literature on full scale instrumented walls reinforced with materials having different linear stiffness.

The results of this study are aimed the two following objectives:

- To check the influence of the different deformability of the reinforcements both in design and collapse conditions.
- To evaluate whether the conventional limit equilibrium methods can be applied to non-uniformly reinforced structures or if a more sophisticated approach (i.e. kinematic compatibility concepts: Juran et al. 1990, Lemonnier et al. 1998) such as the so called "displacement method" proposed by Gourc et al. (1986) should be used.

2. WALL TEST FACILITY

The test model has been constructed in the Static Tests Facility of the Civil Engineering Laboratory of ENEL-HYDRO, a company of the Italian National Electricity Board, in Seriate, Italy. The soil is contained on one side by a reinforced concrete counterfort wall 1 m thick; on the other side and on the back the model has been confined by massive steel frames.

The internal dimensions of the rigid test tank were 2130 x 5100 x 4000 mm; at the end of the construction of the containing structure, a test has been carried out in order to verify its stiffness. The internal surfaces of the test facility are comprised of polyethylene sheeting to ensure that model performance approaches a plane strain condition.

The test facility allows full scale model to be constructed, surcharged, excavated and monitored in a controlled laboratory environment.

3. PROPERTIES OF SOIL AND REINFORCEMENT

The soil used for the fill is a medium uniformly graded silica sand (Ticino sand, TS), with a Proctor density of 15.58 kN/m^3 and an optimum moisture content of 4.38%.

This sand has been extensively investigated in the past from a geotechnical point of view. Strength and deformability parameters obtained from triaxial tests carried out on specimen reconstituted at the same relative density (D_R=80 %) as that considered in the present research are summarised as follows

- peak friction angle $\varnothing'_p = 43°$
- drained cohesion $c' = 0$
- constant volume friction angle $\varnothing'_{cv} = 34°$
- Young modulus ($\varepsilon = 0.1\%$) $E' = 60$ MPa

The reinforcement consisted of two different kinds: a woven wire mesh (Maccaferri Green Terramesh, tensile strength = 50.11 kN/m when tested in accordance with ASTM A975-97 and secant modulus J=900 kN/m) and a polyester geogrid (Terramgrid 3/3-W, tensile strength = 30 kN/m when tested in accordance with EN ISO 10319 and secant modulus J=300 kN/m); both reinforcements have a PVC coating to increase their durability and resistance to damage.

The wire mesh units are manufactured from a single sheet of hexagonal double twist 2.7/3.7 ∅ mm wire woven mesh formed in three sections:

- the horizontal reinforcement layer
- a face section lined with a biomat
- the horizontal return at the top

The interfacial shear resistance between sand and reinforcements was determined from pullout and direct shear tests on a large apparatus capable of housing samples of the following dimensions:

- direct shear tests: 0.7 x 0.7 m
- pullout tests: 0.7 x 1.5 m

4. MODEL CONSTRUCTION

A wrapped-face wall 2.0 m wide has been constructed up to a total height of 3.6 m consisting of 6 reinforcement layers alternatively placed: the bottom layer is made with woven wire mesh, over which a layer made with geogrid is laid and so on (Figs. 1-2). The wall is battered with a sloping face angle β=20° from the vertical.

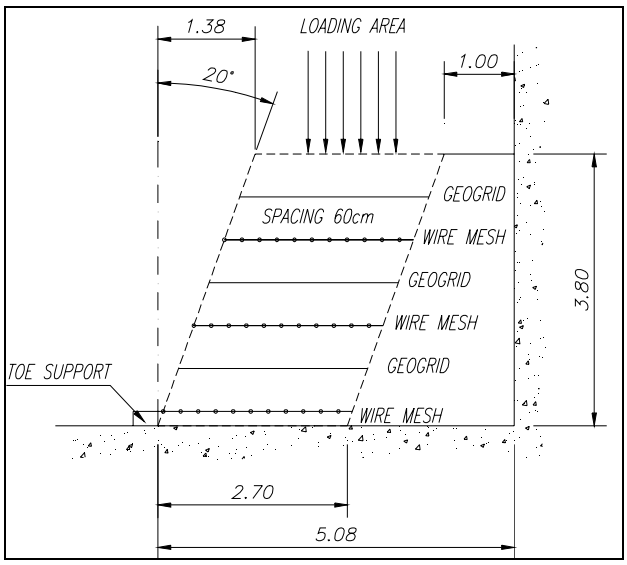

Figure 1: Cross section with layout of the reinforcements

The model was built on a basic layer (0.20 m thick) made of the same sand used for the structural embankment and the backfill: therefore the structure is eventually 3.80 m high. Polyethylene sheets have been used inside the reinforcement facing to prevent loss of the sand through the apertures.

The length of the reinforcements was 2.70 m at each layer and the cuff was 1 m for the wire mesh layers and 0.50 m for the geogrid one. Each layer has been constructed with the following procedure:

- Placing and stretching of the reinforcing unit
- Backfilling
- Compaction with vibrating plate.

The laying and compacting operations have been made on 0.20 m thick layers, thus achieving the 100% of the optimum Proctor density.

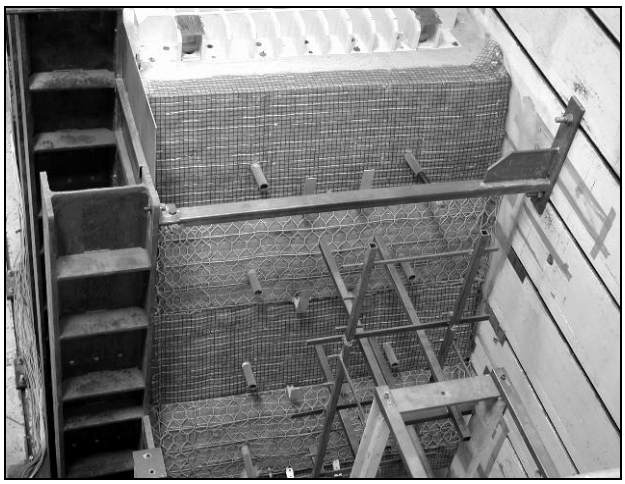
Figure 2: General view of the completed structure

5. LOAD ARRANGEMENT

The loading was applied through a steel plate with stiffening ribs (dimensions: L=1.90 m; B=1.10 m) initially placed 0.35 m backward from the upper front of the wall structure. Further loading was applied by an hydraulic system acting on the plate through a spherical joint. (Fig. 3). In order to hold up the required loads, a steel frame anchored to the reinforced concrete wall has been installed on the top of the hydraulic piston.

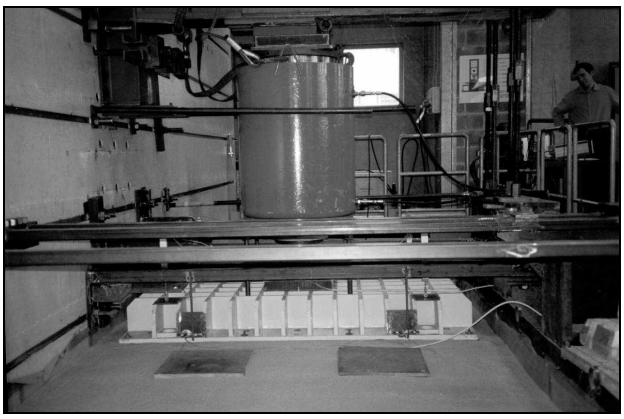
Figure 3: Detail of the loading system

6. INSTRUMENTATION

The instrumentation layout used in the test is shown in the following figures:

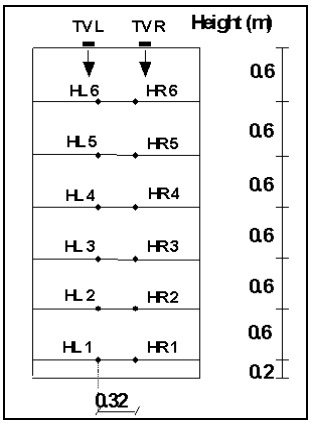

Figure 4: Instrumentation on the facing

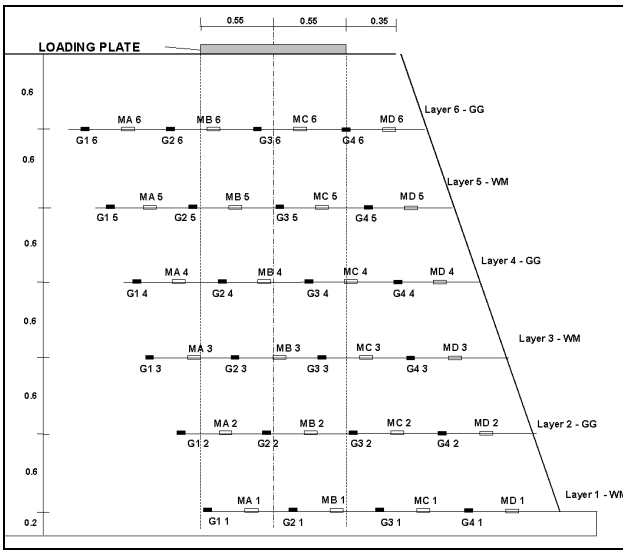

Figure 5: Cross section with measuring equipment

Each reinforcement has been instrumented with (Figs. 4-5):

- 2 LVDT for measuring the horizontal deformations of the front facing (HL HR).
- 4 LVDT for measuring the mesh/geogrids deformations (MA, MB, MC, MD) over a reinforcement length equal to 160 mm, that is 1 longitudinal mesh opening for the wire mesh and 7 mesh opening for the geogrid (Fig. 6)

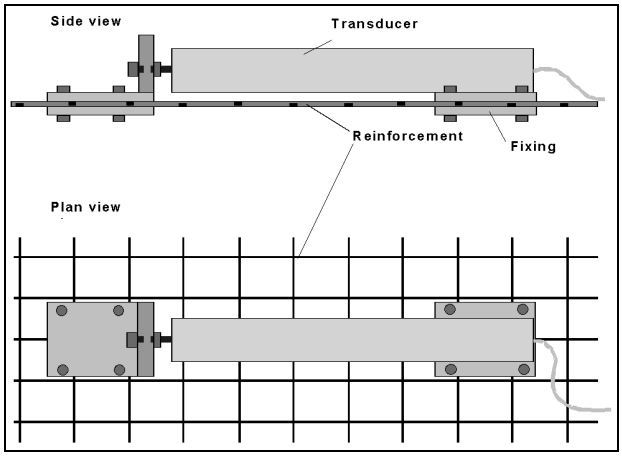

Figure 6: LVDT used for measuring the axial strains in the reinforcements

- 4 cable extensometers (G1, G2, G3, G4) for measuring the horizontal displacements within the reinforcement length.

The following instrumentation was also used:

- 4 LVDT (VP1, VP2, VP3, VP4) for measuring the vertical displacements of the loading plate
- 2 LVDT (HPR, HPL) for measuring the horizontal displacements of the loading plate
- 2 LVDT (TVR, TVL) for measuring the vertical displacements of the upper front of the wall
- 7 pressure cells (C1 to C7) for measuring the vertical earth pressure on the reinforcement layers.
- 1 pressure cell for measuring the hydraulic pressure in the actuator.

Furthermore, in order to measure the settlement profiles at each layer, two HDPE inclinometer pipes have been placed horizontally at each layer for the whole length up to front face; the pipes have been used in conjunction with a Mercury pressure transducer.

7. LOADING PHASE

Once completed, the wall was left in place for 840 hours, during this time all the instruments were monitored.

The first loading step corresponded to the application of the loading system (plate+joint+actuator=35 kN); further loading was applied with 107 kN per step. This phase lasted 6 weeks.

When a load of 1638 kN, corresponding to an average applied pressure on top of the face equal to 784 kPa, was reached, the settlement of the plate increased from 40 to 68 mm in a very short time, thus indicating the collapse of the reinforced soil structure (Fig. 7).

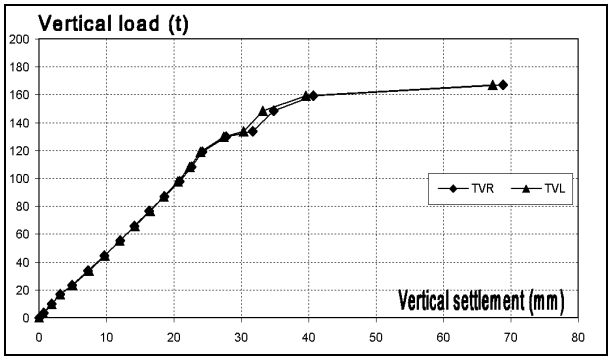

Figure 7: Load deformation behaviour of the upper front of the wall

In figure 8 the horizontal displacements of the wall face are plotted for different load steps, while in figure 9 the measured distribution of the strains in the reinforcements after collapse is shown, together with the measurements taken during loading.

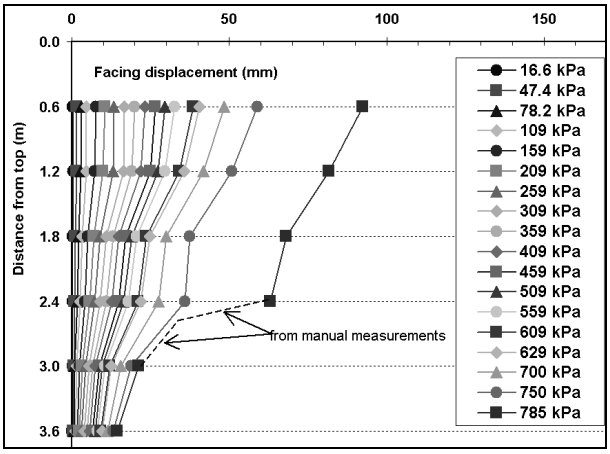

Figure 8: Horizontal deformation of the wall face

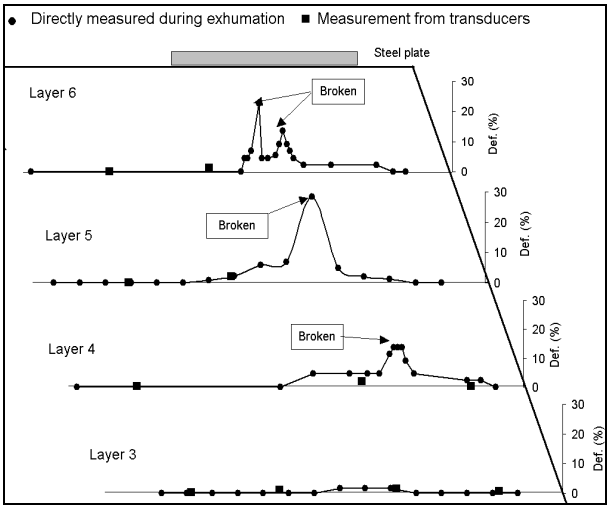

Figure 9: Strain distribution in the reinforcements 3-6 after collapse

8. FAILURE MECHANISM

The analysis of all measurements gathered during loading phase (Figs. 8 and 9) and the observations of the integrity of the reinforcements and the deformed shape of the inclinometer pipes after material removing at the end of the test (Fig. 10), allowed the determination of the failure surface (Fig. 11).

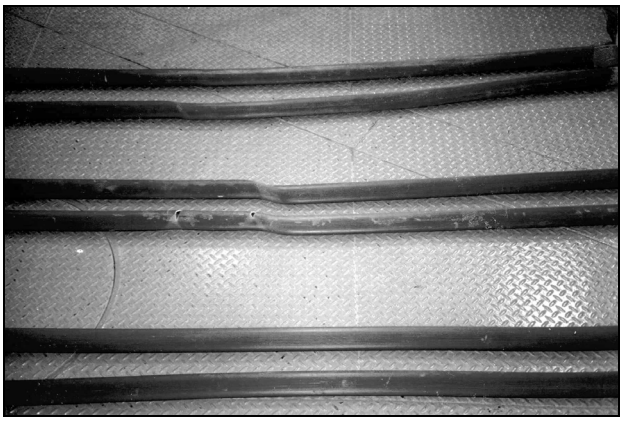

Figure 10: Pipes after removal

The failure surface seems to be of log-spiral shape; it begins closely inside the plate's rear side and ends at the basis of the third layer. This is consistent with the observations made in previous full-scale tests carried out on top-loaded wrapped-face uniformly reinforced walls (Thamm et al. 1990, Gotteland et al. 1997, Bathrust 1999, Haza et al. 2000). Also the trend of the axial strains in the reinforcement layers (Fig. 9) is in good agreement with that observed in the aforementioned experiences (see, for example, Gotteland et al. 1997)

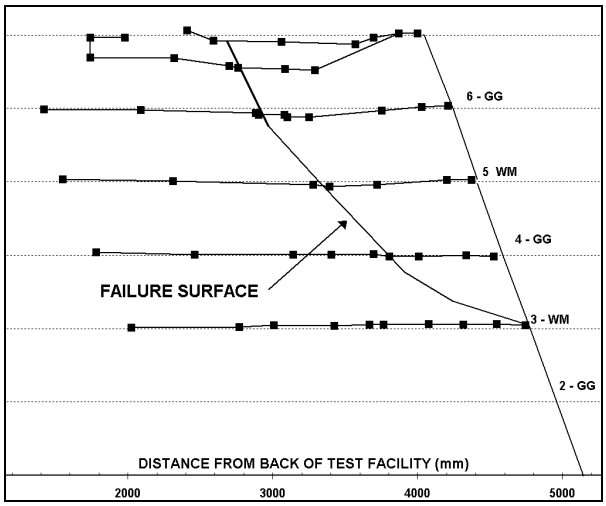

Figure 11: Excavated internal failure surface

9. ANALYSIS OF THE RESULTS

A large amount of data from the test program is currently being analysed; some preliminary observations can be made:

- The result of the stability analysis run along the measured surface with the limit equilibrium approach and selecting the triaxial peak friction angle underestimates the failure load by approximately 14 %
- The load-settlement curves of the front of the wall (Figs. 9and 10) show an almost linear behaviour for vertical loads up to 1177 kN (i.e. surcharge pressure equal to 563 kPa); for higher loads the behaviour becomes non-linear because of increasing plastic deformations

10. FURTHER RESEARCH

Plane-strain numerical analysis using a two-dimensional finite difference code (FLAC) will be performed in order to model the reinforced soil structure taking into account the sequential bottom-up construction of the wall facing, soil, reinforcement and surcharge.

This analysis will allow to evaluate whether the measured stress-strain and collapse behaviour of the wall can be fitted by the conventional limit-state equilibrium methods or if a more sophisticated method which takes into account the different linear stiffness of the reinforcements, such as the previously mentioned "displacements method", must be used.

To this purpose a new software (MACSTARS 2000) has been developed to perform reinforced walls stability analysis using different types of reinforcement and complex design scenarios. The software, apart from the conventional limit equilibrium approach (Bishop, Janbu), has been implemented with an original application of the "displacements method" that allows to automatically adjust the pullout stiffness of the reinforcement layers according to the actual failure surface considered in the stability analysis.

11. CONCLUSIONS

A full scale test on a locally surcharged non-uniformly reinforced soil structure has been carried out to evaluate the stress-strain response of the two adopted reinforcement materials with different linear stiffness (woven wire mesh and polyester geogrid).

The instrumentation used for the measures proved to be adequate to the requirements in terms of precision and reliability. The load deformation behaviour of the foundation slab was approximately linear up to an applied load equal to 1177 kN, then plastic deformations started to significantly increase up to the collapse, reached at 1638 kN (i.e. a pressure on the plate equal to 784 kPa).

The combined analysis of the measured deformations (within and outside the reinforced soil structure) and of the visual inspection on the reinforcements sheets during the exhumation of the wall allowed for the thorough determination of the failure surface.

The undergoing interpretation and numerical modelling activity will allow to evaluate whether the calculation methods normally used for analysing the stability of the reinforced walls can be applied also to non-uniformly reinforced soil structure or if more sophisticated approaches (such as the "displacement method" or other similar ones based on kinematic compatibility concepts) which take into account the different linear stiffness of the reinforcements should be used.

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