Numerical modelling of a geogrid reinforced wall

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ABSTRACT: A 4 meter height vertical retaining wall has been built more than two years ago by reinforcing the backfill with geogrid layers. The wall has two different kind of reinforcements a one directional geogrid with nominal strength 20 kN/m and a bi-directional geogrid with nominal strength 45 kN/m. Reinforcements have been installed along the two section of five meter of wall with different vertical spacing and length. The wall was designed with a safety factor close to unit and has been surcharged up to failure. To control the static behavior of the reinforced structures in the elapsed time after completion, instrumentation has been installed inside the wall. Geotechnical characterization of the constitutive elements of the wall has been carried out by means of in situ and laboratory test. In this paper the structure has been modeled by means of the finite difference approach employed in the program FLAC-2D. It has been possible to investigate the influence of different parameters on the wall deformations such as the angle of shear strength of the backfill and the interface friction between soil and geogrid. Comparison between the results of the numerical analysis and the collected data on the wall behavior, have allowed a better understanding of the interaction effects which develop in geogrid reinforced structures.

1 INTRODUCTION

The use of reinforced retaining structures in many practical applications of civil engineering has become very widespread because these structures are less expensive than concrete structures and are able to reduce the environmental impact. The geotechnical design of these structures is routinely performed using numerical methods or close solutions based on limit equilibrium methods; this approach is widely used in geotechnical applications for its simplicity.

However a design procedure based on limit equilibrium methods neglects any aspect of soil and reinforcement deformability; moreover, due to little data available about geogrid reinforced structures, their behavior are still not clearly known. Consequently the approach based on limit equilibrium methods is not satisfactory if the objective of the study is to investigate the effect of geometrical and mechanical properties of materials on deformations.

This last aspect is very important because the geotechnical design of retaining reinforced structures depends not only on failure condition but also on serviceability aspects in exercise conditions.

To overcome this problem a more rigorous analysis should be performed, taking into account the interaction between the soil and the geogrids under loading condition close or away from failure.

In this paper the static behavior of a 4m height vertical retaining wall, 10 m long, was modeled by means of the finite difference approach employed in the program code FLAC-2D.

During its construction, the reinforced wall was instrumented with strain gauges, tensile load transducers, horizontal displacement sensors and total pressure transducers; data were collected over a period of about 16,000 hours. Comparisons between the results of the numerical analysis and the collected data is presented to allow a better understanding of the interaction effects which develop in geogrid-reinforced structures.

2 EXPERIMENTAL DATA

2.1 *Construction and design criteria of reinforced wall*

A 4m height vertical retaining wall, 10 m long, was constructed more than two years ago by reinforcing the backfill with two different kinds of geogrids: one section (named wall section “A”), 5m long, had a one-directional geogrids LBO 220 SAMP with nominal strength of 20 kN/m, the remaining section (named wall section “B”), 5m long, had a bi-directional geogrids TT 201 SAMP with nominal strength of 45 kN/m.

To ensure the face stability, the walls were constructed using “left in place” welded wire formworks. These are wire mesh (ϕ = 8 mm, # 200 $\times$ 200 mm) 1.5 m in height, forming an angle of 85° with the horizontal. The soil was compacted in layers of 0.3 m in thickness, using a vibrating roller (Carrubba et al, 1999).

After completion the wall was surcharged with 3.5 meter of backfill up to failure; the backfill ensures a uniform vertical pressure of 62 kN/m^2. A complete description of the structures and of collected data is reported in a companion paper by Carrubba et al. (2000) in the present conference.

The reinforced wall was designed using the limit equilibrium approach implemented in the RESLOPE code (Leshchinsky 1995) with a safety factor close to unit. The two sections were designed according to different failure mechanisms: the tie-back tensile failure, for the portion of wall reinforced with GG20PP geogrids (wall section “A”), and the pullout failure for the other portion reinforced with GG45PE geogrids (wall section “B”).

To control the static behavior of the structure in the elapsed time after completion, reinforcements were instrumented with strain gauges, tensile load transducers and horizontal displacement sensors. Figure 1 shows the cross section of the two reinforced wall together with the localization of strain gauges along reinforcement.

Data were collected over a period of about 16,000 hours starting from the early phase of construction until failure.

3 NUMERICAL MODELING USING FLAC-2D

3.1 *Finite difference analysis*

The two-dimensional finite difference program FLAC-2D (1995) was used to model the static behavior of the reinforced wall and to compare numerical results with experimental data.

This code is an explicit finite difference solutions based on the lagrangian analysis of continua that is well suited for modeling large distortions and material collapse. A complete description of numerical formulations is reported in FLAC user manual.

Using this approach it is possible to investigate the influence of different parameters on the static behavior of the wall such as the stress-state before the loading, mechanical properties of geogrid, backfill and foundation, the stiffness of the wall face, the influence of the interface friction angle between soil and geogrid. In this paper a parametric analysis has been carried out to mainly investigate the influence of the interface friction and of soil shear strength angle

No attempt was made to model compaction induced stresses during construction and only the final surcharge configuration was modeled in the finite difference analysis.

Even if large scale pull-out tests provided interaction factors between the soil and the geogrids, close to unit, interface elements between soil and geogrid were introduced in the analyses to evaluate its influence on results.

Boundaries were placed far enough to not interact with stress and strain distribution inside the structure. The boundaries on the left and on the right sides of the model were able to displace only in the vertical direction, while at the bottom both horizontal and vertical displacements were prevented. The mash employed in this study (figure 2) has 4600 plane elements.

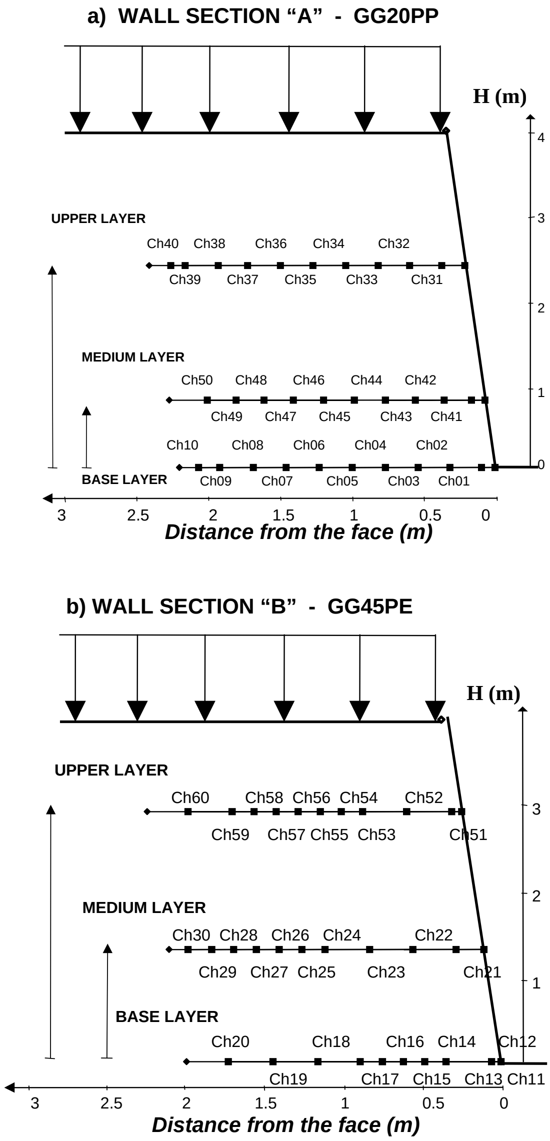

Figure1. Cross section of the two reinforced walls and strain gauges location (after Carrubba et al., 2000)

3.2 *Material properties and modeling*

To model the embankment and foundation, the Mohr-Coulomb failure criterion was employed. Associated flow was considered with a dilatancy angle ψ coincident with the friction angle ϕ'. For frictional materials this assumption could lead to much greater plastic volumetric strains; however, if boundary restrictions over deformations are not serious, as in the case of an active earth mass behind a wall, the hypothesis of associated flow could still be considered acceptable.
Following the results of laboratory tests under K_0 conditions, an initial at rest soil pressure coefficient $K_0 = 0.4$ was considered.

The following soil geotechnical properties are used in conjuction with the Mohr-Coulomb model in finite difference analysis: the angle of shear resistance ϕ' and cohesion was respectively assumed 46.5° and 15 kPa, the dilatancy angle ψ was assumed 46.5°, the unit weight of foundation and fill soil was assumed 17.7 kN/m^3.

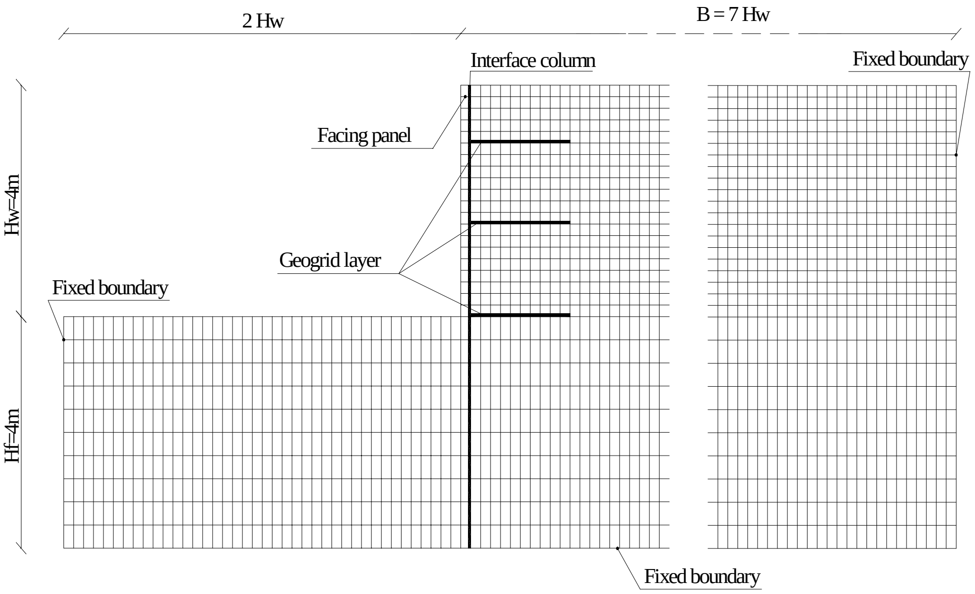

Figure2.Bi-dimensional mesh used in the analyses.

The foundation soil layer was considered highly stiff, with an elastic modulus E' ten times greater than the backfill one. The bulk modulus K and shear modulus G of foundation and backfill have been obtained by laboratory triaxial tests; finally Poisson's ratio was assume equal to 0.3.

Reinforcement layers where modeled using linear elasto-plastic cable elements with negligible flexural stiffness and compressive strength; the cross section area A is equivalent to geogrids area. These elements are one-dimensional and may be linked at fixed point respect to the reference grid. The mechanical properties of the cable element are the yield strength T and the young modulus E which are reported in table 1. The Young modulus for geogrid was selected at 2 % of axial strain. The length L of reinforcement was assumed as described in paragraph 2.2.

The interface between the reinforcement and soil was modelled by two layers above and below the reinforcements, with friction angle δ and negligible thickness. In this study the values of interface friction angle varied in the range between 0.5 ϕ' and 1.5 ϕ'.

To ensure the face stability between reinforcements some cohesion was introduced in the elements close to the face.

All the parameters adopted in numerical analysis are summarized in table 1.

4 COMPARISON OF RESULT AND DISCUSSION

4.1 *Geogrid tensile strain*

The results of analyses in terms of tensile strain along reinforcement are reported in figures 3 and 4 for wall section "A" and in figures 5 and 6 for wall section "B".

In this figures the evaluated tensile strain are compared with the experimental ones for two condition of loading: figures 3 and 5 are related to the full embankment without surcharge, while figures 4 and 6 considers also the presence of the final surcharge.

These figures are reported in a normalized form for both tensile strain and distance from the face; tensile strain has been normalized with respect to the maximum value reached in each geogrid,

Table 1. Soil and geogrid constitutive model and parameters used in numerical analysis.

Material	***Model / Element***	***Model Parameters***
Fill Soil	▪ Elastic perfectly-plastic model ▪ Mohr – Coulomb failure criterion	$\gamma = 17{,}7\ kN/m^3$ K = 46 MPa G = 12 MPa $\nu = 0.3$ $\varphi' = 46{,}5°$ $c' = 15\ kPa$ $k_o = 0{,}4$
Foundation Soil	▪ Elastic perfectly-plastic model ▪ Mohr – Coulomb failure criterion	$\gamma = 17{,}7\ kN/m^3$ K = 460 MPa G = 120 MPa $\nu = 0.3$ $\varphi' = 46{,}5°$ $c' = 15\ kPa$ $k_o = 0{,}4$
Polymer Geogrid GG20PP (Wall Section “A”)	▪ Elastic perfectly-plastic model ▪ Cable element	E= 8500 MPa $A = 0{,}5 \cdot 10^{-4}\ m^2$ $\delta = 0{,}5 \cdot \varphi' / \varphi' / 1{,}5 \cdot \varphi'$ T = 20 kN/m
Polymer Geogrid GG45PE (Wall Section “B”)	▪ Elastic perfectly-plastic model ▪ Cable element	E = 6000 MPa $A = 1.5 \cdot 10^{-4}\ m^2$ $\delta = 0{,}5 \cdot \varphi' / \varphi' / 1{,}5 \cdot \varphi'$ T = 45 kN/m

while the distance from the face has been normalized with respect to the full length of geogrid reinforcement. All the performed analyses are referred to a ratio ϕ/δ varying in the range between 0.5 and 1.5.

As it is possible to observe there is a general good agreement between experimental data and numerical results; in particular for embankment without surcharge the best fit is obtained by considering interface friction lower than the soil friction angle; while in presence of full surcharge, i.e. for stress conditions close to failure, the best fit is obtained by considering interface friction greater than the soil friction angle.

4.2 *Localization of maximum tensile strain*

The maximum values of tensile strains allow a localization of failure surface inside the walls, even if the failure mode planned for the walls regarded the tie-back or pull-out failure of reinforcements. The envelopes of maximum tensile strain in geogrids are shown in figures 7 a) for wall section "A" and 7b) for wall section "B". It is possible to point out that for the more deformable wall section "A" these envelopes are fairly planar and close to the Rankine surface, while for the less deformable wall section "B", equipped with more stiff geogrids, these envelopes are curved and similar to a log spiral.

A comparison between the computed maximum tensile strain along the geogrid layers and the experimental ones, is shown in figure 8a) for the wall section "A" and in figure 8b) for the wall section "B". It is possible to observe that the localization of maximum experimental tensile strain is in a good agreement with that obtained by numerical method for interface friction greater than soil friction angle.

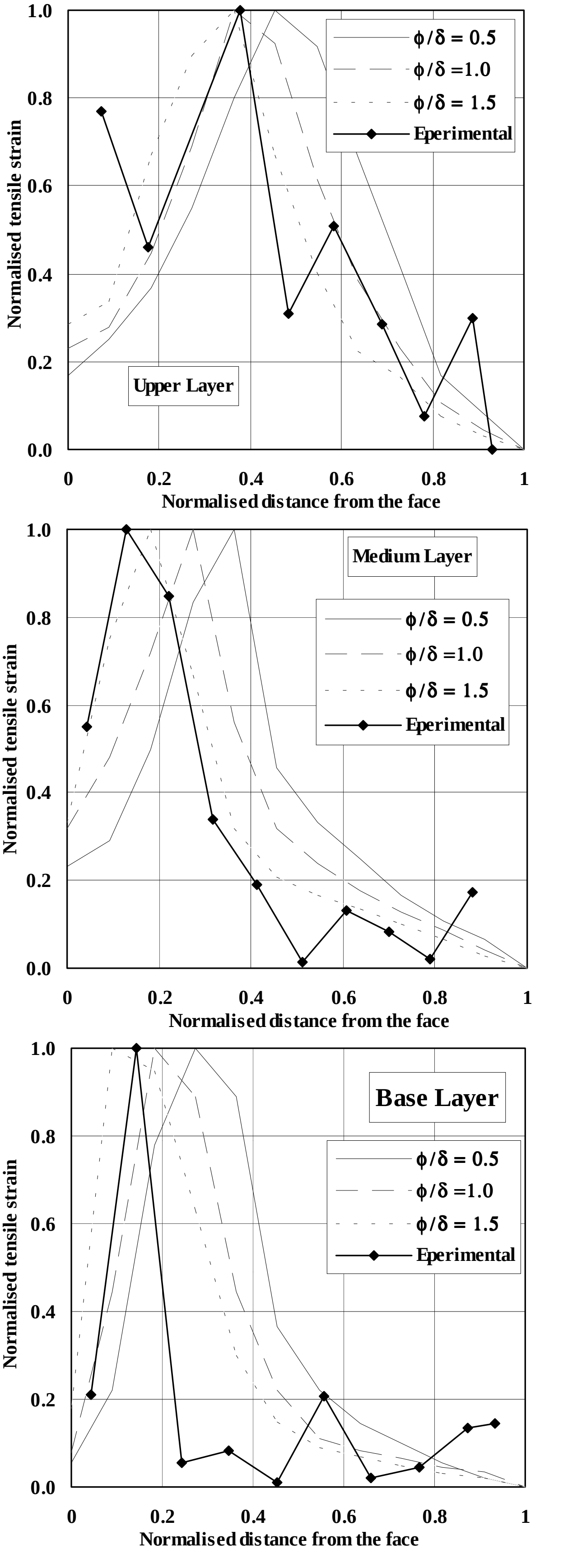

Figure 3. Comparison between experimental data and data and numerical analysis result for wall section "A" tion "A" 750 hours after construction

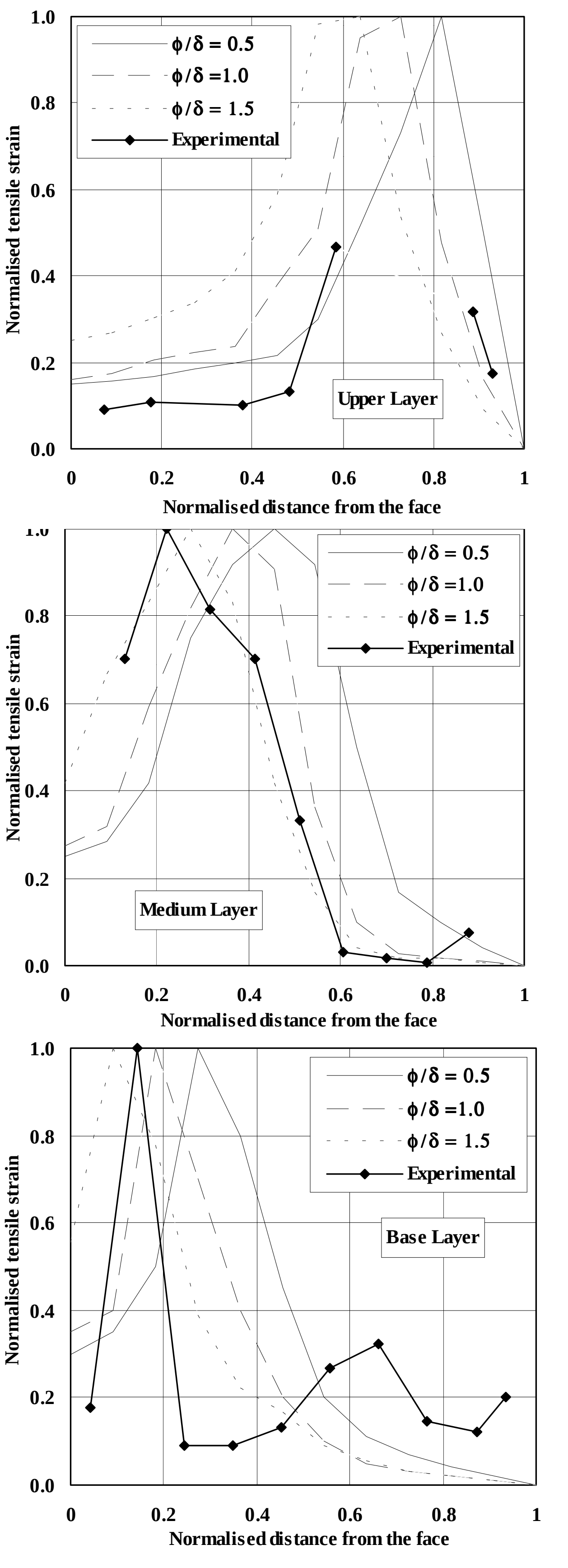

Figure 4. Comparison between experimental and numerical analysis result for wall sec- 1600 hours after construction

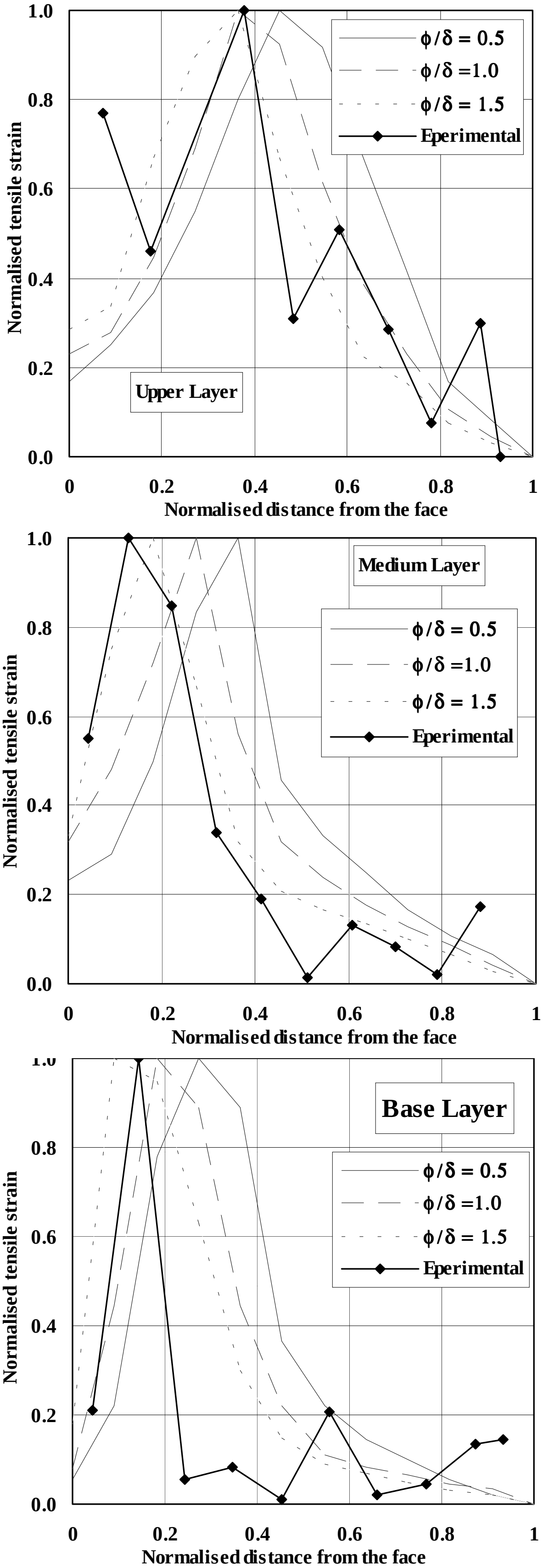

Figure 5. Comparison between experimental data and
data and numerical analysis result for wall section “B”
tion “B” 750 hours after construction

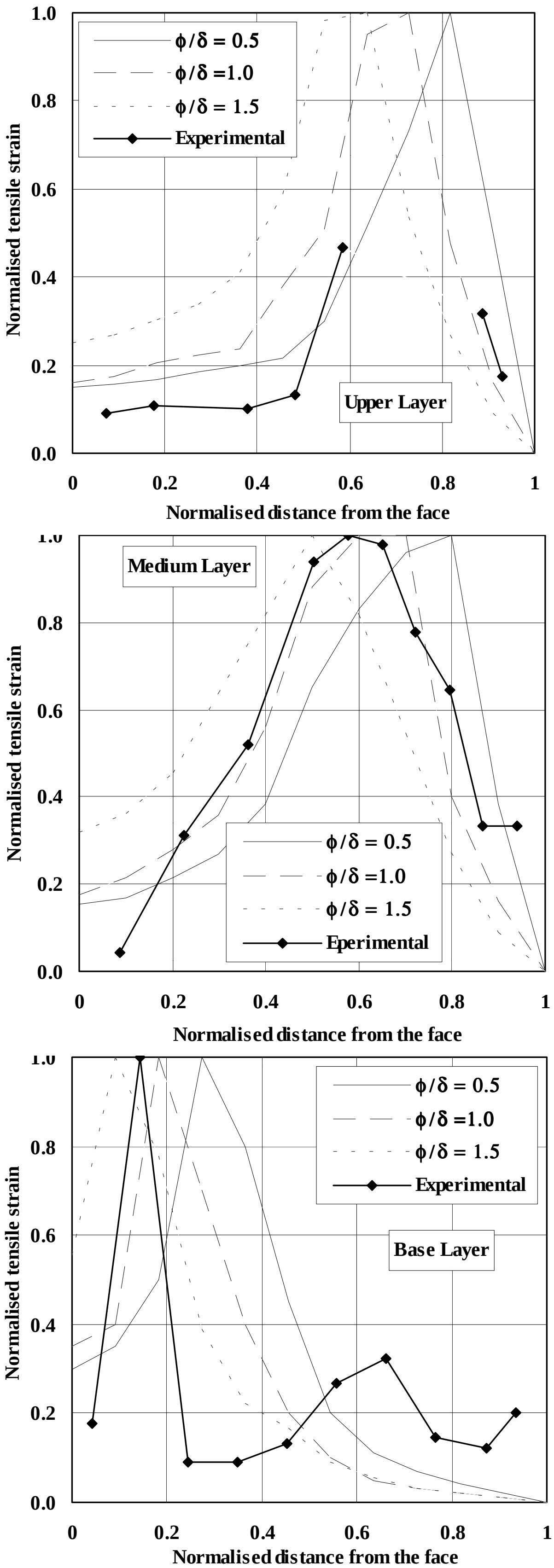

Figure 6. Comparison between experimental
and numerical analysis result for wall sec-
1600 hours after construction

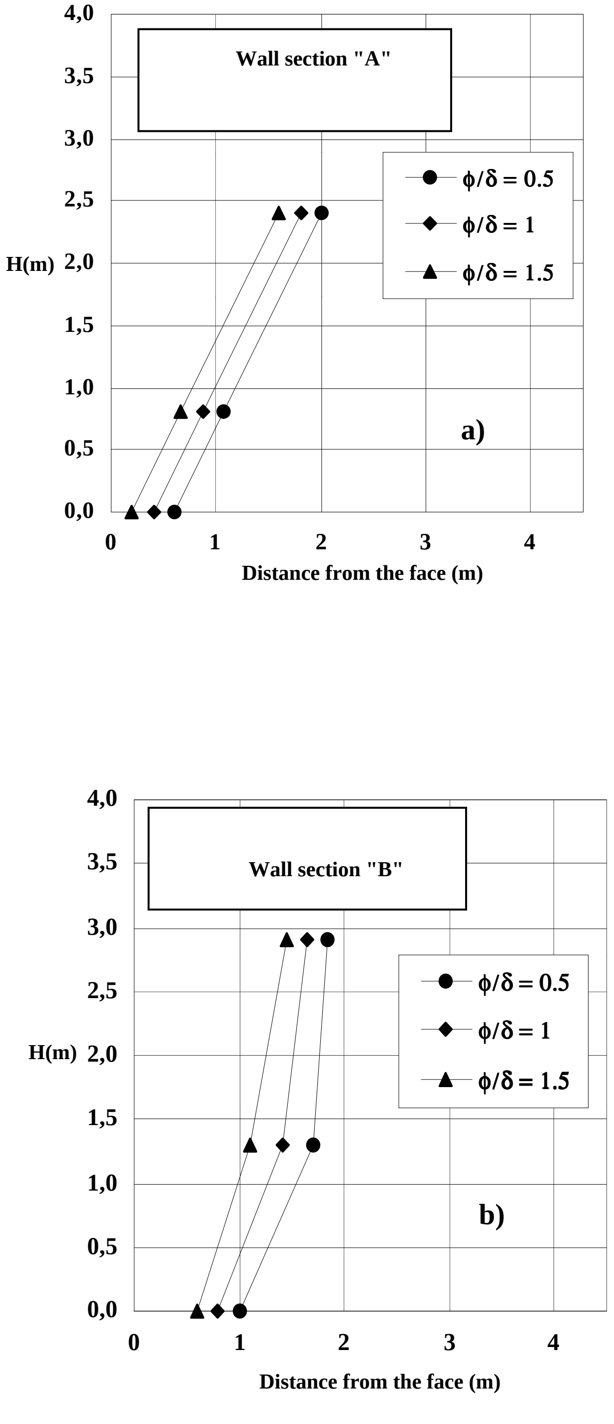

Figure 7. Maximum tensile strains in geosynthetic obtained by numerical analysis.

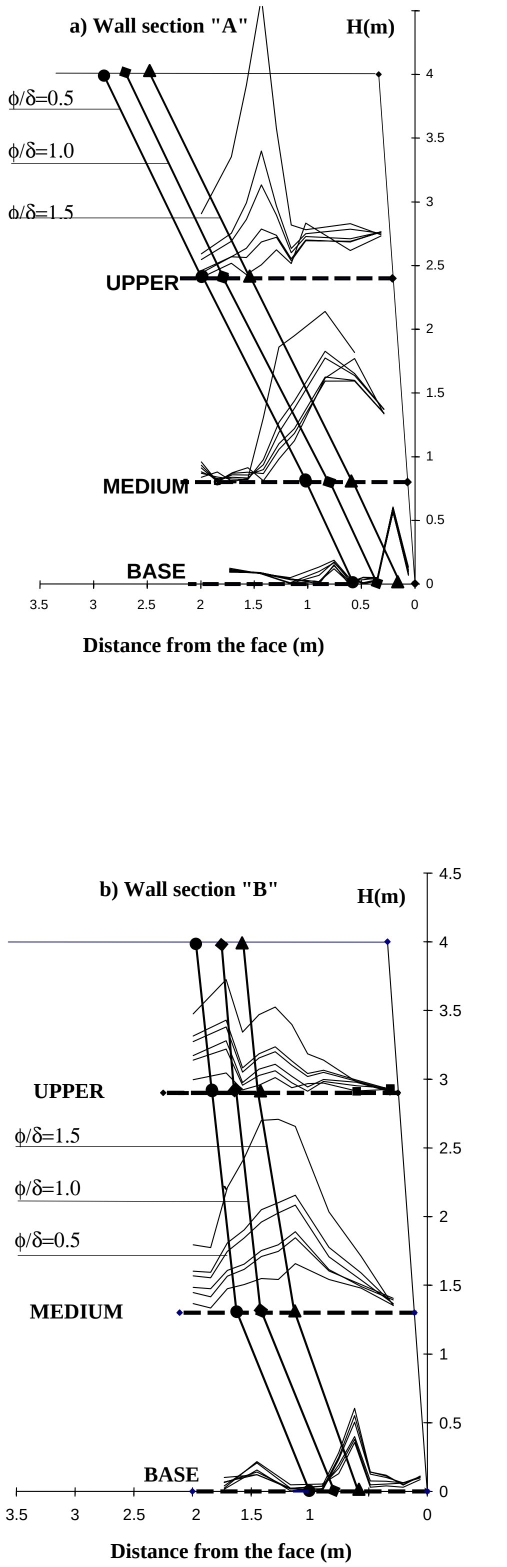

Figure 8. Comparison between computed and measured maximum tensile strains in geosynthetics.

5 CONCLUDING REMARKS

In this paper the static behavior of a 4m height vertical retaining wall, 10 m long, was modeled by means of the finite difference approach employed in the program code FLAC-2D.

During construction, the reinforced were instrumented with strain gauges; data were collected over a period of about 16,000 hours. Comparisons between the results of the numerical analysis and the collected data have shown a general good agreement; in particular for embankment without surcharge the best fit is obtained by considering interface friction lower than the soil friction angle; while in presence of full surcharge, i.e. for stress conditions close to failure, the best fit is obtained by considering interface friction greater than the soil friction angle.

Finally, the envelope of maximum tensile strain in geogrids, evaluated by numerical model, agree very well with the empirical data for interface friction grater than soil friction angle.

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