STRAIN COMPATIBILITY AND GEOGRID STIFFNESS SELECTION IN THE DESIGN OF REINFORCED SOIL WALLS

Riccardo Berardi[1] & Giovanni Pietro Pinzani[2]

[1] *Dept. of Civil, Environmental, Architectural Engng. - Univ. of Genova, Italy (e-mail: riccardo.berardi@unige.it)*
[2] *SEIC Geotecnica, Division of HARPO s.p.a., Trieste, Italy (e-mail: g.pinzani@seic.it)*

Abstract: In designing reinforced soil walls, one of the most important, but often disregarded, aspect is that of strain compatibility. Strain compatibility has to be considered mainly 1) for a proper choice of soil shearing resistance; 2) for defining reinforcement characteristics, in particular axial stiffness; 3) for assessing acceptable movement values of the structure.

Considering geogrid reinforced soil walls, the same mobilised reinforcement force (assumed as a design value by using safety factors or simply as the result of the interaction mechanisms) can be attained with tensile strains that are different by up to an order of magnitude, depending on the adopted geogrid type. This choice influences both the final aspect of the wall (i.e. its serviceability state) and the adopted design soil strength parameter. Therefore, to avoid incorrect (underestimated/overestimated) assumptions of soil strength level, which could lead to an oversize or unsafe structure, strain compatibility has to be taken into account.

With the aim of exploring these aspects and supplying practical design indications, in the paper the results of a number of numerical analyses are reported. The performed FEM analyses have been carried out 1) with a commercial, robust and user-friendly code, especially developed for geotechnical analyses; 2) starting from the evidence provided by well documented and representative case histories, in order to provide confidence in the deduced results; 3) assuming different wall geometries and actual geogrid characteristics; and 4) calibrating soil parameters by considering triaxial test results, performed on dense and low confined sand specimens, in order to define strain levels and corresponding mobilised strength and stiffness.

The evidence gathered from the analyses strengthens the importance of the strain compatibility concept and of the proper choice of geogrid axial stiffness.

Keywords: Strain compatibility, reinforcement stiffness, isochronous curve, finite element, field performance, reinforced soil walls.

INTRODUCTION

For design of reinforced soil walls where limit equilibrium methods are adopted and ultimate conditions are referred to, soil-reinforcement interaction mechanisms are not properly considered and, consequently, walls movements. A different approach, especially as far as serviceability conditions are concerned, relies on the concept of strain compatibility. When analyzing the reinforced wall behaviour in this perspective, particular care has to be paid in defining soil shearing resistance parameters and reinforcement characteristics, such as axial stiffness and creep behaviour.

The same mobilised reinforcement force (assumed as a design value by using safety factors or simply as the result of the interaction mechanisms) can be attained with tensile strains that are different by up to an order of magnitude, depending on the axial stiffness of reinforcing elements. In order to design a reinforced wall that could be safe and in an acceptable working condition, an approach based on strain compatibility could be taken into account.

The equilibrium in the reinforced soil may be investigated using a compatibility curve (Figure 1) constructed by assuming that there is equal tensile strain in the reinforcement and in the soil in the direction of the reinforcement (Jewell 1996). The mobilised soil resistance (i.e. the value of the friction angle) that has to be considered, depends on the expected equilibrium in the reinforced soil mass and, therefore, by the experienced strain levels which, in turn, are strictly related on reinforcement stiffness.

The question if it is the peak friction angle (ϕ_p') or the constant volume (ϕ'_{cv}) friction angle that has to be used in designing reinforced soil walls has been arisen by many Authors (e.g. Leshchinsky 2001; Zornberg 2002). Several methods are based on the peak strength parameter; otherwise, especially when Ultimate Limit State is of concern, analyses based on ϕ'_{cv} are often suggested. For Serviceability Limit States, when the design in mainly governed by allowable displacements, the problem is still open.

Considering geogrid reinforced soil walls, with the aim of exploring these aspects and supplying practical design indications, the paper reports the results obtained by numerical analyses performed in order to investigate the influence of the choice of soil strength and geogrid stiffness values on the wall movements. Different geometries and reinforcements have been considered. The models have been calibrated by the comparison with the evidences provided by of well documented case histories. Current strain levels in geogrid reinforced walls and in the soil have been taken into account by the analysis of published case records as well as by triaxial tests performed on dense and low confined sand specimens.

The evidence gathered from the analyses strengthens the importance of the strain compatibility concept and of the proper choice of geogrid stiffness.

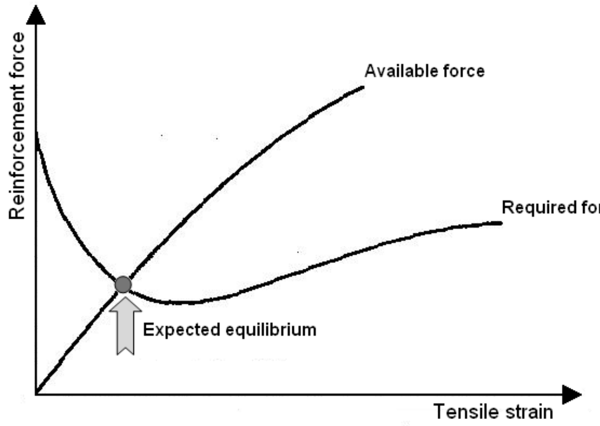

Figure 1. Example of compatibility curve

STRAIN LEVELS FROM CASE HISTORIES

In order to consider the displacements and strain levels actually experienced in walls and in their reinforcing elements, that have to be compared with soil strains occurring in peak/constant volume conditions, a number of case histories have been considered. Only very well documented cases could supply detailed information useful for the scope, so the database collected and published by Allen and Bathurst (2002) and Bathurst *et al.* (2002) has been taken into account. The reported measurements refer to walls characterized by different heights (H=3÷12 m); facing type (wrapped-around, segmental, concrete panel, etc.); reinforcing elements (uniaxial/biaxial geogrids, woven/non woven geotextiles); polymer (HDPE, PP, PET), soil gradation; measurement technique (local/global).

Peak axial strains in the reinforcements do not exceed 3% with a slight increase due to creep phenomena; in most cases strains are less than 1%. The ratio of the maximum horizontal displacement of the facing to the wall height (U_x/H) ranges between 0.1 to 1.5%. At failure peak strain increases up to 10%.

Mc Gown *et al.* (1998) states that the maximum strain measured in reinforcing elements is usually much less than 2%; the values above reported well agree with this indication.

MOBILISED SOIL STRENGTH

The mechanical behaviour of compacted soil fill is characterized by dilation due to high relative density and low confining stresses. Considering usual wall heights and soil unit weight, mean stress less than, say, 100 kN/m^2, could be assumed. Therefore, for properly compacted fills, differences up to 13° between peak (ϕ_p') and constant volume (ϕ'_{cv}) friction angles should be taken into account (Bolton, 1986). Figure 2 illustrates dilation influence on the peak shear strength.

Following the concept of strain compatibility between soil and reinforcement, it appears relevant to assess the average strain levels to which peak and constant volume state have to be referred. Various Authors suggest the strain range ε=3÷6% for mobilisation of peak strength and strains up to 12% for reaching constant volume conditions (Mc Gown *et al.* 1998).

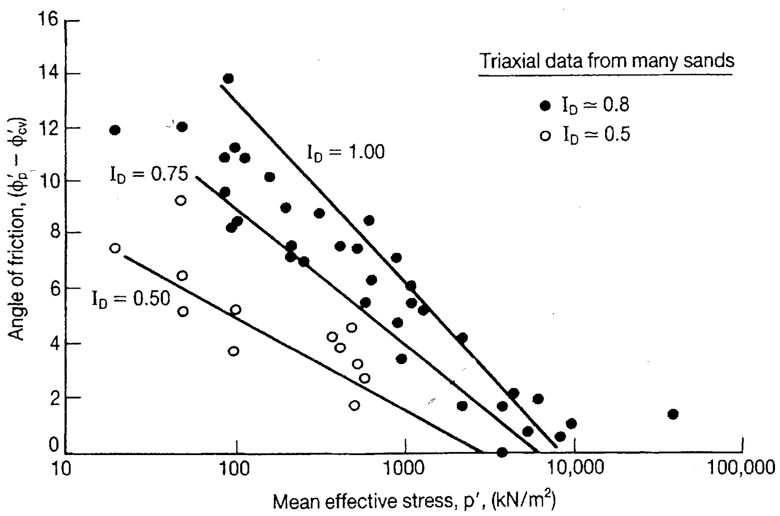

Figure 2. Dilatancy effects on sand strength (Bolton's data, from Jewell 1996)

In order to verify these findings (based on experimental evidences) a set of triaxial compression tests on a uniform silica sand (Ticino Sand), having relative density D_R ranging from 70 to 80% have been interpreted. The main properties of the tested sand are indicated in Table 1. Low confining stresses (σ'_3=20÷50 kN/m^2) have been applied.

The results, illustrated in Figure 3, indicate that the peak strength is mobilized for axial strains ε_1=2÷4%, while constant volume conditions are attained for ε_1>10%.

Table 1. Summary of main physical properties of Ticino Sand

G_S	D_{50} (mm)	U	% fines (<75μ)	e_{min}	e_{max}
2.69	0.55	1.7	0	0.59	0.93

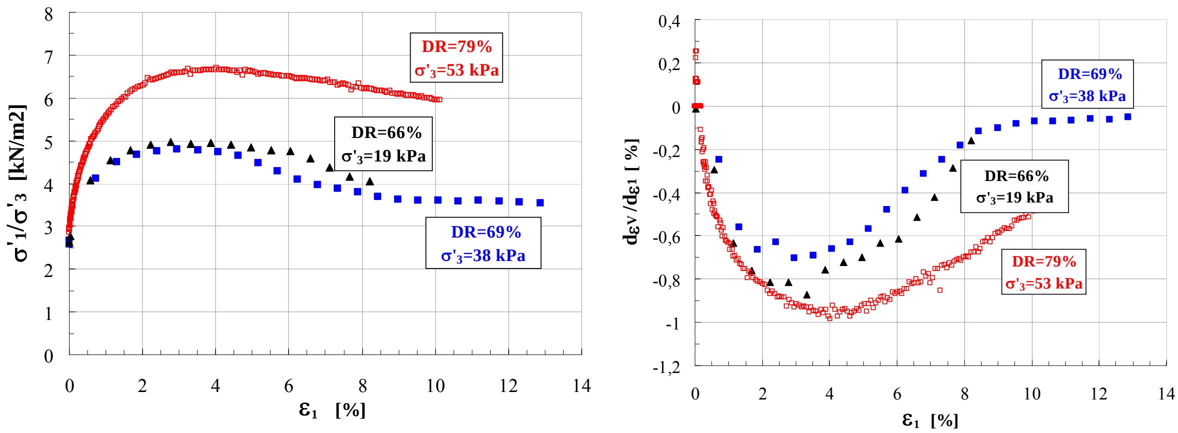

Figure 3. Triaxial tests results

NUMERICAL ANALYSES

In order to analyze reinforced soil wall performance by considering strain compatibility criteria and, possibly, supplying practical design indications, a number of numerical analyses have been performed. The FEM analyses have been carried out with a commercial, robust and user-friendly code especially developed for geotechnical analyses (Plaxis v.8), starting from the evidence of well documented and representative case histories, in order to provide confidence in the deduced results. Different wall geometries, soil and geogrid characteristics have been considered. The relevant role played by the reinforcing element stiffness has been particularly investigated, starting from two actual isochronous curves and assuming axial stiffness related to different strain levels (in the range 0.5÷12%)

Model and parameter calibration

The results coming from numerical analyses are affected by several factors, among which adopted parameters, soil and element discretization, material models. Several tests have carried out to optimize choices and results. In particular, the performance of four actual walls, well documented in Ling *et al.* (2000), Allen and Bathurst (2002), Allen *et al.* (2002) have been reproduced in order to calibrate the model. The walls have the general characteristics indicated in Table 2. The walls are named as in the reference paper.

Table 2. Summary of main characteristics for wall case histories

Wall	Heigth (m)	Reinforcement	Facing	Surcharge	Measurement method
PWRI *	6.0	HDPE uniaxial geogrid	Concrete blocks	no	LVDT's/Strain gauges
GW8 †	6.1	HDPE uniaxial geogrid	Concrete panel	yes	Strain gauges
GW9 †	6.1	PET Woven geogrid	Modular blocks	yes	Extensometers/Strain gauges
GW11 †	2.85	PP biaxial geogrid	Wrap-around	yes	Strain gauges

* Ling *et al.* (2000)

† Allen and Bathurst (2002); Allen *et al.* (2002)

In the following, examples of the results gained by the numerical analyses, in terms of comparison between calculated and observed displacements/strain levels, are reported. A good agreement has been obtained as shown in Figures 4 and 5. Therefore, the choices adopted for element generation, geometry and construction configuration, soil and interface characterization and, finally, calculation phases have been considered in the numerical analyses described in the following section.

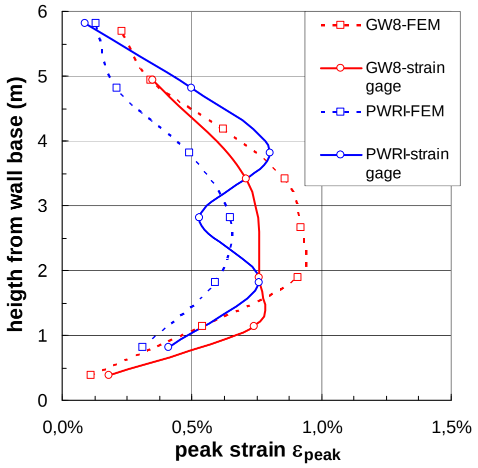

Figure 4. Comparison between measured and calculated reinforcement peak strains

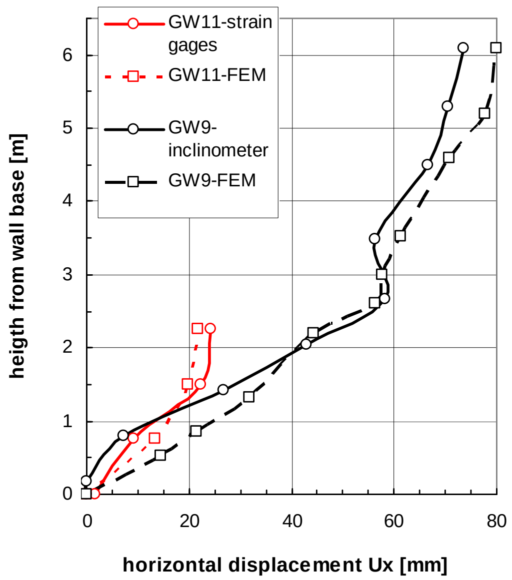

Figure 5. Comparison between measured and calculated wall horizontal displacements

Selection of geogrid stiffness

The reinforcement geometrical and mechanical characteristics obviously influence wall performance, both with regard to stability conditions and to global displacements. In the numerical analyses the reinforcements are modelled as flexible elastic elements that can sustain only tensile forces (no compression). The only property to define is the axial stiffness EA, that is the ratio of the axial force per unit width and the axial strain. Different values of the axial stiffness can be so determined by the knowledge of the normalized isochronous curve of the reinforcement, having defined the ultimate tensile strength.

As far as soil-geogrid interaction is concerned, around the reinforcements set of interfaces (suited to model bond mechanisms) and refinement have been applied. The interface strength (analogous to an "efficiency" parameter) can be defined according to soil and geogrid characteristics, as outlined by Jewell (1996).

Two different geogrids are considered in the following. Table 3 and Figure 6 report, respectively, the main characteristics and the isochronous curves.

Table 3. Summary of geogrid general characteristics

Name	Symbol	Description	Polymer	Grid size (mm)	Ultimate Tensile Strength (kN/m)	Tolerance Tensile Strength (kN/m)
Enkagrid PRO/40	EG	Extruded bars (laser welded)	PET	40x94	48	40
Harpoter 40/20	HT	Woven	PET (PVC coated)	15x15	40	40

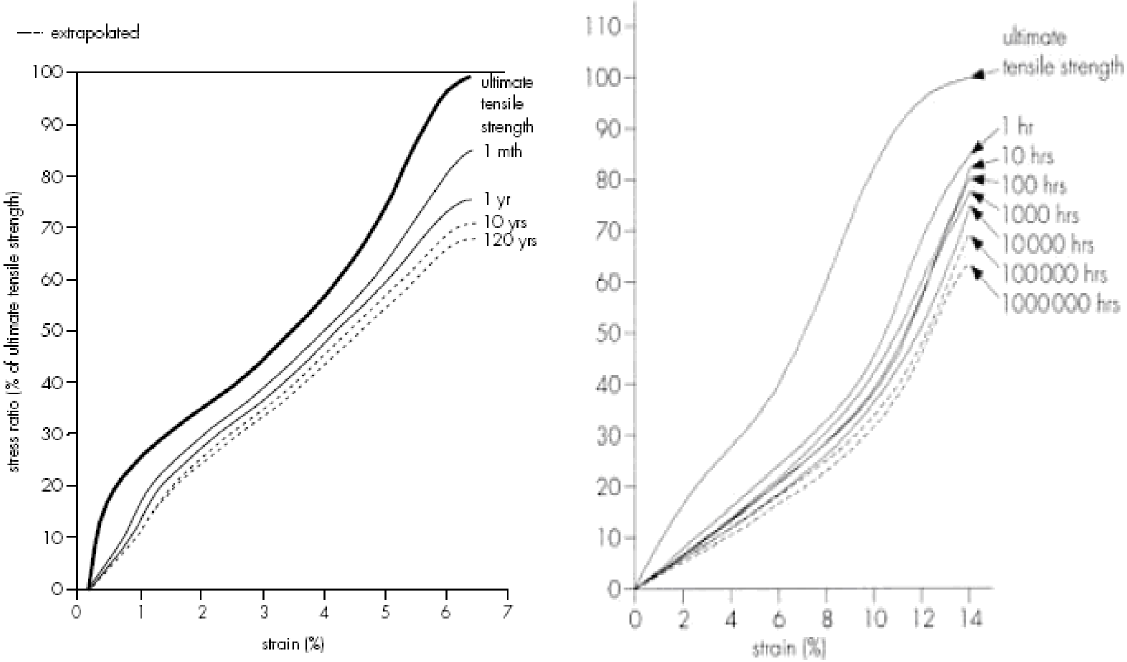

Figure 6. Isochronous curves for geogrids EG (left) and HT (right)

Geogrid "EG" is evidently stiffer than "HT"; the choice of selecting these types of reinforcing elements has been made just to enhance the importance of reinforcement behaviour in approaches based on strain compatibility.

Assuming secant stiffness values, calculated with regard to strain levels (Figure 7), the differences are more evident and stress the importance of defining in which range the wall (and thus the soil) is expected to work. A stiff geogrid, such as type EG, show very high performance especially for strain levels less than 2-3%, typical value, as observed in the section dedicated to the analysis of case histories, for reinforcements walls in a working state. On the contrary, geogrid like type HT, are characterized by a low and almost constant stiffness, regardless to the mobilised strain (and strength) level.

The numerical analyses have been carried out assuming different values of axial stiffness, as indicated in Table 4.

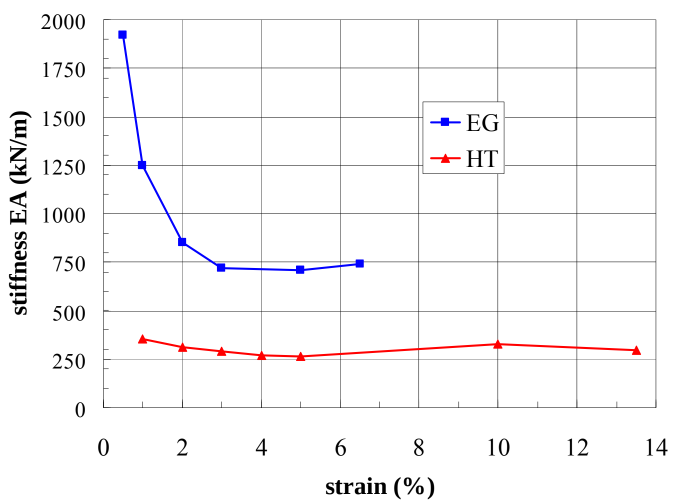

Figure 7. Axial stiffness for geogrids EG and HT

Table 4. Summary of geogrid performance characteristics considered in FEM analyses

Strain (%)	Stress ratio EG (%)	Stress ratio HT (%)	Stiffness EG (kN/m)	Stiffness HT (kN/m)
0.5	20	-	1920	-
1	25	10	1250	350
2	35	20	850	300
3	45	-	720	-
5	75	30	720	250

Cases considered in numerical analyses

The numerical analyses are aimed to highlight the importance of considering compatibility conditions between reinforcement and soil. Therefore, soil parameters representative of a different mobilization of soil strength have been assumed, together with the two geogrids EG and HT.

Three walls of different heights (4, 6, 8 m) are analyzed; the models have been generated starting from the evidences gathered in the calibration phase, extending to the new cases the choices and parameters there adopted. The walls have a wrapped-around facing; the ratio of the geogrid length L to the wall height H is equal to 0.5, in order to have relatively high stressed reinforcements. On the backfill acts a surcharge. Dry conditions are assumed. A sketch of the adopted mesh is illustrated in Figure 8; Tables 5 and 6 report geometrical and soil characteristics . The analyses have been performed with the elastic perfectly-plastic Mohr-Coulomb soil model.

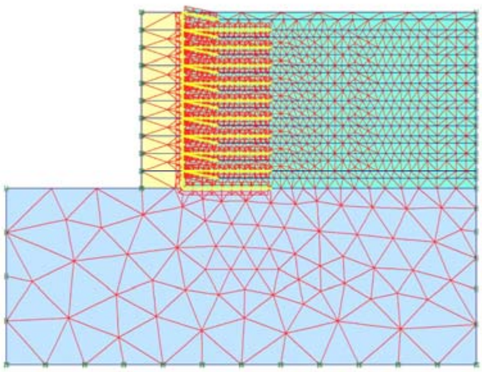

Figure 8. Example of adopted mesh

Table 5. Summary of geogrid performance characteristics considered in FEM analyses

Case	**GW-H4**	**GW-H6**	**GW-H8**
Height H (m)	4.2	6.0	7.8
Geogrid total length L (m)	2.0	3.0	4.0
Vertical spacing Sv (m)	0.60	0.60	0.60
L/H	~ 0.5	0.5	~ 0.5
Sv/H	0.14	0.10	0.08
Number of reinforcements	7	10	13
Overlap length (m)	0.8	1.2	1.6
Surcharge q (kN/m^2)	50	80	100
$q/\sigma_v(H/2)$	~ 1.3	~ 1.5	~ 1.4

Table 6. Soil main characteristics

Soil type	**# 1**	**# 2**
Backfill unit weight γ (kN/m^3)	18.0	18.0
Friction angle ϕ'_p	45°	39°
Constant volume friction angle ϕ'_{cv}	32°	32°
Dilatancy angle ψ	15°	8°
Young modulus E (MN/m^2)	60	24
Shear modulus G (MN/m$^{2)}$	25	10
Poisson’s ratio	0.2	0.2

The soil type #1 may be referred to at-rest or “pre-peak” conditions while type #2 is introduced for a fictitious simulation of “post-peak” conditions, both for strength and deformability parameters.

RESULTS

A set of “calibration” analyses has been carried out on the wall GW-H6 in order to assess the mean level of tensile strain in the reinforcements. It is worth observing that these are assumed as global strains, calculated considering the “net” horizontal displacement, which is deducting from the total displacement of the geogrid the value at the rear.

These tests are been made varying the geogrid stiffness (Table 4) and adopting soil type #1. As expected, the results are particularly affected by the value of EA for the stiff geogrid EG, being the peak strain ε however less than 1%. On the contrary, for geogrid HT, ε is always higher than 1% , with maximum values reaching 3%.

With effect from these preliminary outcomes, all the walls have been simulated, considering, according to soil-reinforcement strain compatibility:

- Geogrid EG – stiffness EA=1250 kN/m - Soil type #1
- Geogrid HT – stiffness EA=250 kN/m - Soil type #1 – Soil type #2

Because of the typical strain levels expected for very extensible geogrids, like HT, it appeared more appropriated analyzing the so reinforced wall performance with both soil types.

The results of the numerical analyses are summarized plotting peak strains ε and normalized horizontal displacements (Ux/H) against normalized reinforcement heights (Y/H) (Figures 9-10).

Figures 11 reports the values of the maximum tensile force mobilized in each reinforcement (divided by the ultimate value); the distributions along wall height and along geogrid length (not shown) follow the expected trends.

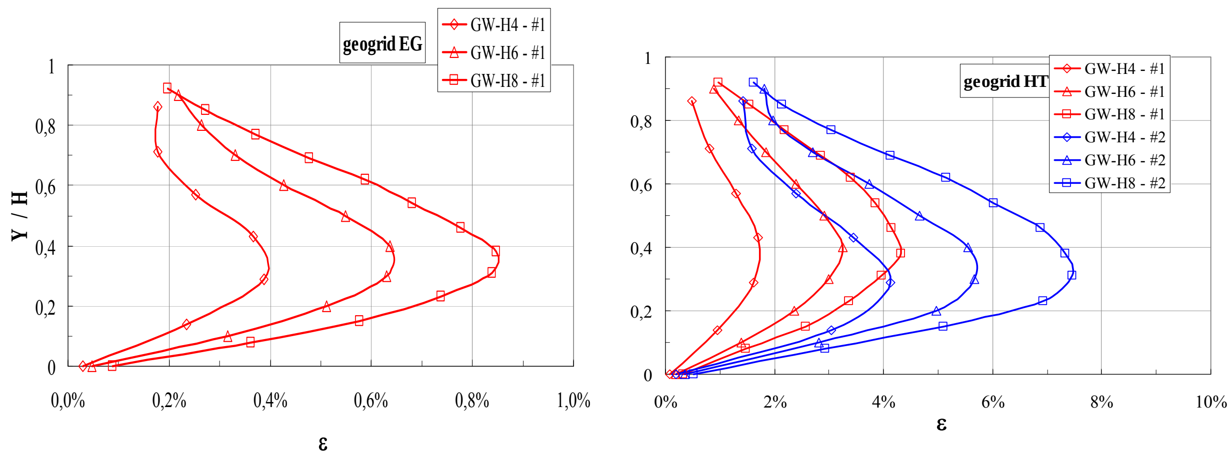

Figure 9. Strains in reinforcing elements

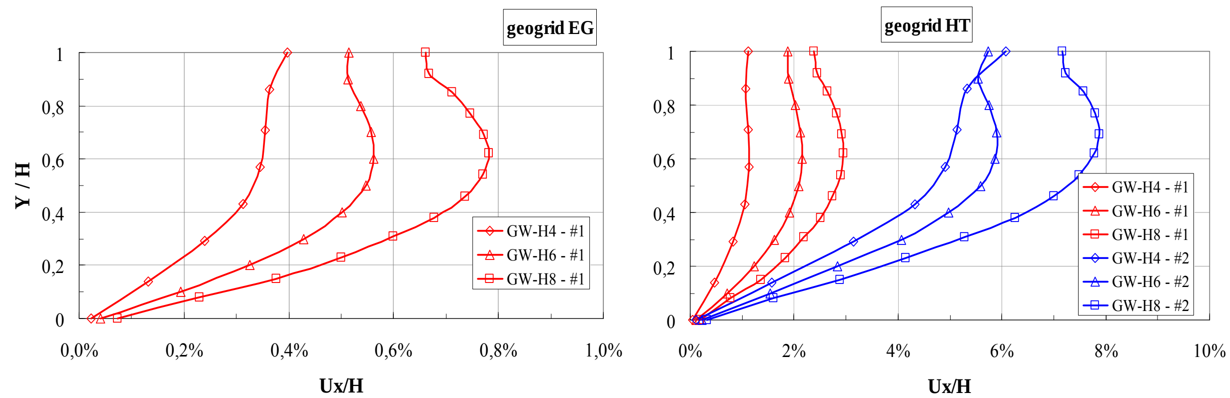

Figure 10. Horizontal displacements in reinforced walls

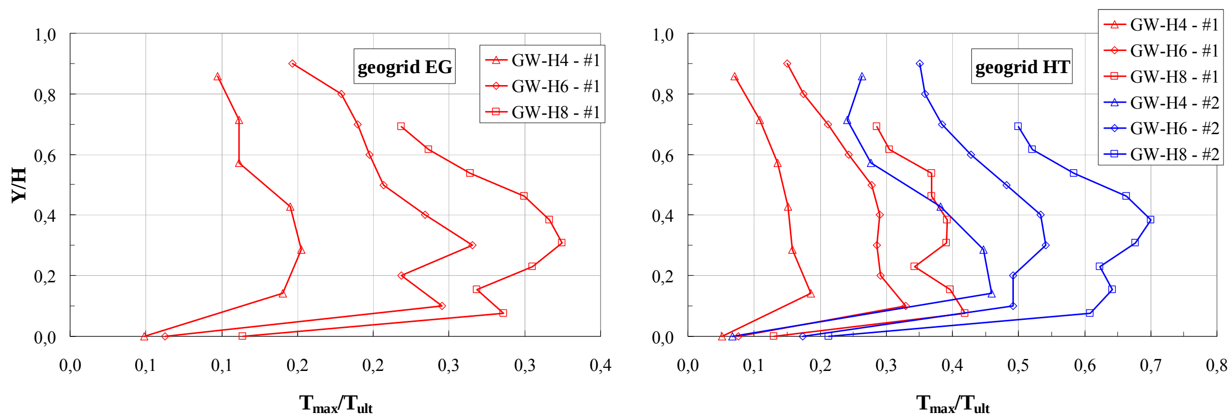

Figure 11. Tensile forces mobilized in the reinforcements

The obtained results allow to remark that:

- as expected, when stiff reinforcing elements are adopted, the global displacements are limited, allowing the structure to be considered in a serviceability limit state;
- the displacements are obviously also affected by the soil mechanical characteristics, which, in turn, may be affected by the compaction quality or, if proper techniques are adopted and backfill soil shows dilative behaviour, by the induced strain levels;
- for these reasons it is important, in the selection of the values of soil parameters, taking into account that with very extensible geogrids large displacements and strains will take place; in this case it is more difficult to define which value of soil strength is to be considered;
- for geogrids characterized by low axial stiffness, it seems advisable to assume the constant volume friction angle ϕ'_{cv}, this resulting in a more correct and conservative choice of soil strength;
- designing with high stiffness geogrids, on the other hand, allows to consider peak strength condition, at least as far as serviceability states are considered; the strain levels in the reinforcements, in fact, are compatible with the strains mobilized in the soil before reaching its peak resistance,
- the stress ratio mobilized in the stiff reinforcements (adopting peak friction angle (ϕ_p') are almost half the ones for very extensible geogrids with same ultimate strength (but lower friction angle);
- this means that in the former case the wall is stable with adequate level of safety while in the latter the stability conditions are critical, the safety factors low and, again, the displacements probably unacceptable.

CONCLUSIONS

In the paper the problem of strain compatibility between soil and geosynthetic reinforcement has been considered as far as geogrid reinforced walls are concerned.

The careful analysis of well selected case histories has indicate that, at working state, in the reinforcing elements the mobilised strains are usually compatible with the levels of strain experienced in dilative soil in pre-peak conditions.

In considering strain compatibility as a design approach, particularly for serviceability limit state, the selection of geogrid axial stiffness plays a relevant role, because the soil strength parameters have to be chosen with respect to the expected wall displacements.

The use of stiff geogrids results in small displacements and in a safe mobilization of soil and reinforcement strengths. The opposite verifies for very extensible geogrids.

These aspects are also relevant if one considers the possible creep phenomena in reinforced walls, which may have very negative consequences for wall displacements, when occurring in an already high loaded and strained reinforcing element, characterized by a low axial stiffness.

Acknowledgements: The authors acknowledge the careful work done by A Verardo, M Tabellario and E Mortola.

Corresponding author: Prof Riccardo Berardi, Dept. of Civil, Environmental, Architectural Engng. - Univ. of Genova, Via Montallegro, 1, Genova, 16145, Italy. Tel: +39 010 353 2506/2491. Email: riccardo.berardi@unige.it.

REFERENCES

Allen, T.M., & Bathurst, R.J. 2002. Soil reinforcement loads in geosynthetic walls at working stress conditions. Geosynthetics International, vol 9, No 5-6, 525-566.

Allen, T.M., Bathurst, R.J. & Berg, R.R. 2002. Global level of safety and performance of geosynthetic walls: an historical perspective. Geosynthetics International, vol 9, No 5-6, 395-450.

Bathurst, R.J., Allen, T.M., & Walters, D.L. 2002. Short term strain and deformation behavior of geosynthetic walls at working stress conditions. Geosynthetics International, vol 9, No 5-6, 451-482.

Bolton, M.D. 1986. The strength and dilatancy of sands. Géotechnique, 36, No.1, 65-78.

Jewell, R. 1996. Soil reinforcement with geotextiles. CIRIA Special Publication 123, Thomas Telford, pp. 332.

Leshchinsky, D. 2001. Design dilemma: use peak or residual strength of soil. Geotextiles and Geomembranes, vol.19, 111-125.

Ling, H.I., Cardany, C.P., Sun, L-X & Hashimoto, H. 2000. Finite Element study of a geosynthetic-reinforced soil retaining wall with concrete-block facing. Geosynthetics International, vol. 7, No. 3, 163-188.

Mc Gown, A., Andrawes, K.Z., Pradham, S. & Khan, A.J. 1998. Limit state design of geosynthetic reinforced structures. Proceedings. 6th Int. Conference on Geosynthetics, Atlanta, IFAI, 143-179.

Zornberg, J.G. 2002. Peak versus residual shear strength in geosynthetic-reinforced soil design. Geosynthetics International, vol. 9, No. 4, 301-318.