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Finite difference analysis of geotextile reinforced earth walls

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ABSTRACT: Some aspects of the behaviour of geotextile reinforced earth walls are analysed by means of numerical techniques. Attention is focused on the different techniques that can be adopted in order to model properly the interaction behaviour between soil and flexible reinforcements, the construction procedures and different facing supports. Results of some of the numerical analyses, all performed by means of a commercial finite difference code, are presented with special reference to two trial walls recently built in France and Italy.

1 INTRODUCTION

Geotextile reinforced soil walls are gaining considerable attention as retaining structures and providing a valuable alternative to traditional concrete walls. With respect to the latter, they present a good ratio between cost and effectiveness and a low environmental impact.

The design methodology of this kind of retaining structures must assure a confident safety factor with respect to collapse mechanisms (tilting or sliding of the whole wall or of some portions of it, pull-out of the reinforcements, etc.), and limiting conditions on the displacements experienced by the structure under working loads. The second aspect is particularly relevant in walls built with flexible reinforcements such as geotextiles, for which displacements often define the serviceability limit.

While, at least for rather simple geometries, the collapse load can be usually defined by means of analytical techniques, derived from classical limit equilibrium methods (Mangiavacchi et al. 1986; Cividini 1995, Gourc et al. 1988), a reliable method for the prediction of displacements is not available for general conditions, as under working loads the behaviour of reinforced walls is the result of a rather complicated interaction problem (Gourc et al. 1994).

Numerical methods are now widely used in order to have an insight of the stress-strain behaviour of this kind of structures, both during construction sequence and working life. It is worth noting that, as in the analysis of any other kind of geotechnical problem, rather complex geometries can be handled, a realistic constitutive model for soil can be assumed, and the stress-strain behaviour of the geotechnical work can be simulated during its entire life. As for reinforced earth walls, some aspects particularly relevant on their behaviour, such as construction procedures, different kinds of facing support or interface behaviour, can be taken into account.

Here, some analyses, all performed with a finite difference code, are presented. Comparisons of results are made between different modelling approaches that can be adopted to take into account the interaction behaviour between soil and geotextile. Different construction procedures have been compared in terms of reinforcement tensile forces and wall displacements.

Finally the results of some analyses performed with reference to two case histories are presented. Two trial walls, recently built in France with the support of the French Ministery of Research (GARDEN Project, Gourc et al. 1995) and in Italy by the Provincia Autonoma di Trento (Trento Wall, De Col et al. 1995), are studied by means of the preceding techniques.

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2 FINITE DIFFERENCE ANALYSIS

All the analyses that will be presented have been performed with the commercial finite difference code FLAC (Fast Lagrangian Analysis of Continua).

The governing equations are substituted by finite differences in space, written in terms of field variables at discrete points (Cundall 1987). In FLAC the contour integral formulation of finite differences (Belytschko, 1983) is implemented. This formulation is able to overcome the difficulties often associated with the best known Taylor series finite difference formulation in terms of mesh pattern and imposition of boundary conditions. The basic idea of this formulation is to make use of Gauss theorem to express the mean value of the gradient of a field variable in a zone by means of a contour integral performed on the boundary of the same zone. Assuming a linear variation of the relevant quantities along the edges of the zone, limited by a discrete numbers of nodes, all the governing equations can be written, in difference form, in terms of the nodal variables (for the complete formulation of the method see for example Herrmann and Bertholf (1983)). With reference to the standard quadrilateral elements implemented in FLAC, it can be shown (Belytschko, 1983) that this method leads, in principle, to a stiffness matrix analogous to that which can be obtained with four node quadrilateral finite elements with reduced integration.

For what concerns the solution of the algebraic system of equations, the dynamic relaxation method is implemented. The steady state solution is achieved by means of successive integrations of the dynamic equations of motion written for the system:

$$\underline{M}\ddot{\underline{u}} + \underline{C}\dot{\underline{u}} + \underline{K}\underline{u} = \underline{f} \tag{1}$$

in which $\underline{M}$ is the mass matrix, $\underline{C}$ the damping matrix, $\underline{K}$ the stiffness matrix, and $\underline{u}$, $\dot{\underline{u}}$ and $\ddot{\underline{u}}$ represent nodal displacements, velocities and accelerations, respectively. The known vector $\underline{f}$ collects the external forces. As an explicit scheme is used for the solution of the system, provided diagonal mass and damping matrices are adopted, no matrix inversion need to be performed. The method is conditionally stable, stability depending on damping coefficient and ratio between mass and time step. As the sought solution is the static solution, mass and damping are in fact fictitious and can be chosen in order to achieve stability and accuracy of the solution. The stability condition, in principle, is linked to the so called Courant-Friederich-Levy condition, which assures, practically, that during the chosen time interval information cannot propagate from one element to another; in this way, inside the calculation step, the increments of the relevant quantities which have to be updated are decoupled from element to element. The solution procedure can be summarized as follows:

1) Assuming known displacements, velocities and nodal forces at time t_i, by solving equation (1) compute new nodal accelerations.
2) Integrate nodal accelerations to give nodal velocities and displacements.
3) Calculate new nodal internal forces from velocities and displacements, by means of integration, along the element boundary, of the new state of stress calculated by imposing the respect of the constitutive law.
4) Increment to next time step, and go back to point 1), until the steady state solution is achieved. From the user point of view, the desired accuracy can be achieved, by imposing a suitable limit to the unbalanced forces, which means the difference between the applied external forces $\underline{f}$ and the internal forces $\underline{Ku}$.

The method, although not well suited to linear problems, becomes competitive with standard implicit-oriented finite element codes when highly non-linear problems have to be tackled. Moreover, due to the solution strategy, a relevant number of degrees of freedom can be considered, also with a PC version of the code. But, in spite of the rather advanced formulation, strict limitations on the mesh pattern still exist, unless paying in terms of efficiency of the calculation. This is particularly felt when elements of relevant different stiffness are adopted.

3 REINFORCED EARTH WALLS MODELLING

All the analyses have been performed with reference to plane strain conditions. The foundation soil and the filling soil have been modelled with an isotropic elastic-perfectly plastic constitutive model, with a Mohr-Coulomb shear strength criterion. Two elastic constants, namely the bulk modulus K and the shear modulus G have to be defined. Cohesion c and friction angle ϕ define the strength criterion, while associated or non-associated flow rule, defined by dilatancy angle ψ can be assumed.

For what concerns the geotextile reinforcements, they usually present an almost perfectly elastic behaviour in tension until they reach their peak strentgh ($T_f = J.\varepsilon_f$), while their post-peak behaviour may range from perfectly plastic to perfectly brittle.

In order to model the geotextiles, two different approaches may be adopted.

As a first possibility, FLAC's "cable" elements may be adopted. These axial elements, conceived for the modelling of supports like rock bolts or tie-backs, cannot sustain bending, and can be anchored at specific points in the grid or grouted to the mesh so that interface shear forces can be mobilized along the length of the reinforcement as the grid deforms. The mechanical behaviour of this kind of structural element is completely defined by its cross sectional area, Young modulus, tensile (and compressive) yield strength, and shear stiffness and strength for interface bond.

As a second possibility, the use of standard quadrilateral elements can be chosen. This choice is mandatory when axisymmetric problems have to be tackled with FLAC, as, for the moment, the cable elements cannot be used in this class of problems. On the other hand, when the interface friction between soil and geotextile may be assumed not less than the shearing resistance friction angle of the soil (so that no special interface elements need to be adopted), standard quadrilateral elements can provide a valuable alternative. Nevertheless, this choice requires usually the definition of a fictious thickness t* for the geotextiles, as they are too thin to be represented in real scale without a great computational cost. On the basis of the equivalence between the deformation work in tension, a fictitious elasticity modulus must be consequently used. If t is the actual thickness of the reinforcements and E the corresponding Young modulus, the fictitious modulus E* is defined on the basis of the equivalence:

$$E^*.t^* = E.t = J \text{ (Geotextile Modulus kN/m) } \quad (2)$$

The appropriate fictitious thickness t* can be chosen after parametric studies, aimed at defining its influence on the numerical solution. An element thickness of 20 times the actual thickness was found to be appropriate. This choice has moreover the advantage of reducing the stiffness ratio between soil and geotextile, with a regularization of the numerical solution. In an analogous manner the short time strength of geotextiles can be scaled.

For what concerns facing modelling, if the facing is made of the wrap-back of the same geotextiles, again the two preceding approaches, namely cable elements or standard quadrilateral elements, may be adoped. If the facing of the wall is built with concrete blocks, quadrilateral elements with mechanical properties of concrete can be adopted. In order to model the interaction behaviour between the blocks, interface elements between them can be adopted, with a proper definition of the parameters.

As for construction procedures, an analysis of the influence of the support conditions during construction will be presented. Through the application of different boundary conditions during the numerical simulation of the construction sequence, propped and unpropped walls can be modelled.

Some observations will be presented too concerning the effects of compaction on the wall behaviour, and suggestions will be made on the modelling of such effects.

4 ANALYSIS OF REINFORCED EARTH WALLS WITH WRAPPED-BACK GEOSYNTHETIC FACING

The analyses that will be presented in the sequel concern earth walls in which the facing is made of the wrap-back of the geotextile. This choice, which is rather economical, has the environmental advantage of leaving the possibility of hiding the front of the wall, by covering it with grass or vegetation. On the other hand, the absence of a rigid facing demands a more strict control on displacements, both during construction and loading of the wall. To analyze this aspect, expecially with respect to construction procedures, some numerical analyses have been performed (Fidler et al; 1994) aimed at a comparison between a propped construction procedure and an unpropped construction procedure.

4.1 *Influence of construction procedures*

The typical profile of retaining structure considered first is a vertical facing wall of: height H = 6m, length of the geotextiles, Lg = 4.5m, geotextile modulus J = 300 KN/m.

The geotextile is wrapped around the facing towards the top of the wall. A surcharge of 10 kPa, is applied on the horizontal ground surface behind the facing. The variation in construction technique which is considered relates to the support provided to the facing during construction. The *propped* condition corresponds to a case in which horizontal support is provided to the facing until the final wall height is reached, when it is removed, allowing the simultaneous mobilization of tension in all reinforcing layers. The unpropped condition corresponds to a case in which support is only provided to each lift of soil (ΔH=0.5m) as it is placed, thus causing reinforcement tensions to develop successively as construction proceeds.

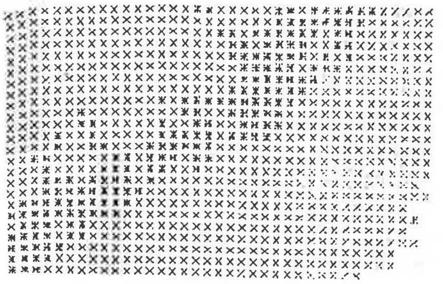


figure 1 : plastic zones (* indicator), propped wall

The soil stiffness is defined with shear modulus G=15 000 kPa and bulk modulus K=25 000 kPa corresponding to a medium dense sand. The failure criteria is defined with an angle of friction $\phi=35°$ and a cohesion c=0 kPa (angle of dilatancy $\psi = 0$).

The reinforcement, including the wrap-back which forms the facing, are represented with cable elements, connected to the soil through interface shear force: the limiting interfacial shear force was specified to develop at a displacement of 15 mm and corresponds to an interfacial friction angle $\phi_g=0,7\phi$.

Development of plastic zones in the soil, as determined by FLAC for the propped case, are illustrated in Fig. 1. It can be seen that soil plasticity is localised in a relatively narrow zone which passes through the toe of the wall. Displacements determined by FLAC for the same case show that the zone of soil platicity corresponds well to the locations where the gradient of displacement is the greatest.

In Fig. 2 (2-a and 2-b) we have the displacements at the free boundaries of the reinforced zone and we can see that smaller displacements are seen to develop for the unpropped case. The propped construction technique leads to an overturning of the wall at the same time as the simultaneaous mobilization of all the reinforcements.

4.2 *The Trento Wall*

The Provincia di Trento, started in 1993 a wide range of experiments, aimed at the analysis and control of already built walls in the region around Trento, in the north-eastern part of Italy. It was also decided to build a prototype wall and to monitor it during construction and under load, in order to compare observed displacements and tensions with those predicted by analytical or numerical methods .

Preliminary numerical analyses were performed in order to support the design of the prototype wall.

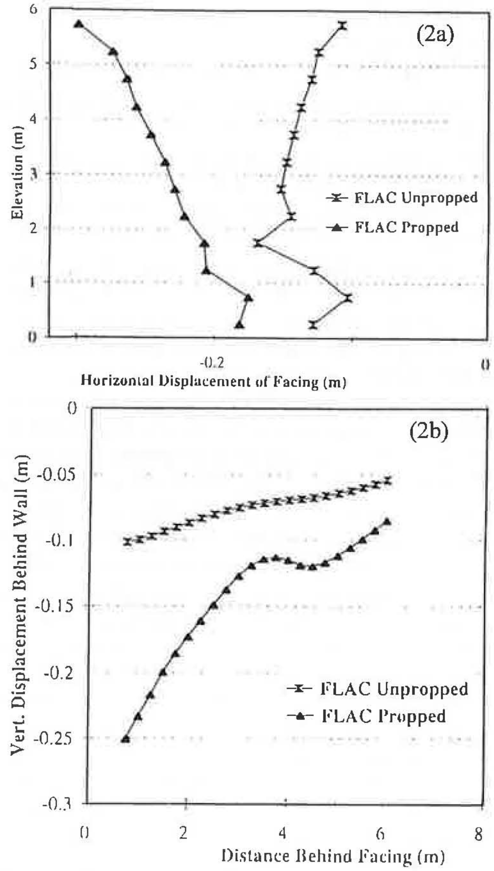


figure 2: horizontal displacements of the wall facing (2a) and vertical displacements behind wall (2b)

A sufficient safety factor was required not to induce a collapse mechanism under load, and displacements and tensions had to be sufficiently high to be observed and compared during the different stages of construction and loading. Parametric analyses were thus performed, in which length and spacing of the reinforcements were varied, toghether with their strength and stiffness. The filling soil consists of a well graded granular soil with sharply angular particles. On the basis of triaxial tests performed, the mechanical parameters were chosen as follows: Young modulus E= 44 MPa and a Poisson ratio $\nu=0.22$ were assumed to describe the average soil stiffness under the range of confining pressure expected during the test. A friction angle $\phi=44°$ and a cohesion c=0kPa were assumed to characterize the shear strength of the filling, while a small dilatancy was observed (dilation angle assumed $\psi=4°$).

After modelling the construction sequence, which in this case corresponds to the unpropped case previously described, a uniform surcharge was applied on the top of the wall and increased by steps, until a collapse mechanism was numerically detected.

The typical grid adopted in the analyses consists in more or less 10000 standard quadrilateral elements, with which filling soil, foundation and geotextiles were modelled. The mesh boundaries were fixed at a distance equal to the height of the wall. This very fine grid was adopted in order to have an accurate description of the evolution of displacements, stresses and strains inside the wall during the test.

All the analyses performed, showed a similar general pattern of behaviour of the walls under load. For low values of the surcharge (tipically up to 25% of the collapse load) the development of plastic zones (Fig.3a), show that the soil behind the reinforced length of the wall tends to behave like an active Coulomb wedge, while the reinforced zone is substantially in the elastic range and works just like a rigid wall. As the surcharge increases, plastic zones begin to develop also inside the reinforced part of the wall, with a concentration of shear strain in its lower portion. Conversely, the plastic zones in the backfill expand mainly in its upper part (Fig.3b).

a

b

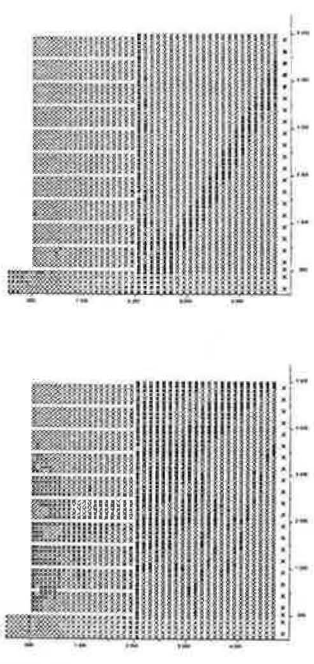


figure 3: development of plastic zones under 40kPa surcharge (3a) and at collapse (3b)

Tensions in the reinforcements go on increasing more or less linearly in the upper part of the wall, thus assuring local equilibrium with the Rankine active thrust, while in the lower part they grow faster as they have to guarantee the global equilibrium of the entire wall, until the tension limit is reached in the second reinforcement from the bottom of the wall. At this stage, collapse of the wall develops, with a dramatical chain effect if the geotextiles present a brittle behaviour after peak.

On the basis of the results of the numerical analyses length and spacing of reinforcements were chosen. Two needs were devised. Firstly, significant displacements were to be expected, so that they could have been measured. But, on the other hand, a good diffusion of reinforcing effects was desired, so that globally the wall could have been considered a stratified continuum. It was then chosen to adopt short reinforcements with low stiffness. A composite polyester was chosen, namely a PEC/25, characterized by a modulus J=330 kN/m (Young modulus E=165 MPa , thickness t=2 mm) and a short time strength T_f=25 kN/m. The length of reinforcements L was chosen equal to 2m, with a ratio L/H=0.4, where H=5m was the total height of the wall The reinforcements spacing was 0.5m. On the top of the wall a final stratum of 0.7m was compacted, to cover the last geotextile layer and prepare the loading area. A schematic diagram of a tipical cross section of the wall is shown in fig.4. The front of the wall is 15m long, and the loaded

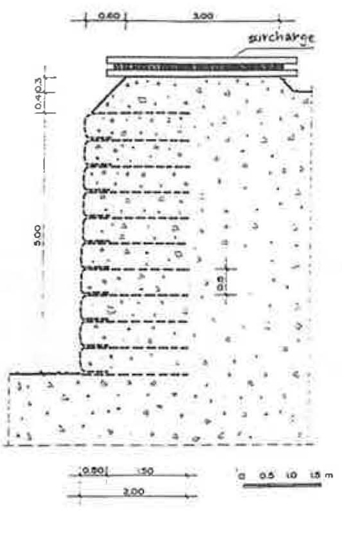


figure 4: schematic view of the central cross section of Trento Wall (from Benigni 1995)

area 7.5m long. Plane strain conditions seem to be plausible in the central portion of the wall.

Construction of the wall was performed by vibration to the optimum Proctor in two stages per stratum. The average volume weight of 20kN/m^3 with a water content of 4% was obtained. The stratum under compaction and the lower one were temporary propped. After the completion of the wall, steel bars were stacked on the top of the wall, in order to apply the surcharge, up to a value of 84 kPa.

Four sections were instrumented for survey measurements of displacements. A series of steel wires, protected by small plastic tubes, were anchored to each of the reinforcing layers at different distances from the facing, and were pretensioned through the application of a weight on their end at the back of the wall on a control panel (Fig.5).

The average displacements experimentally observed are rather smaller than those calculated in advance with the numerical analyses, being around a half of those expected. This discrepancy is likely to be linked to the mechanical behaviour of the compacted soil.

In the numerical analyses, in fact, the variation of mechanical properties of the filling during compaction of the layers was not simulated. Progressive construction in layers was only taken into account through the activation of the weight of each stratum one after the other, while modifying support boundary conditions in order to simulate the temporary propping, but assigning to the soil layers their final mechanical properties. The choice, in this case, seems to be satisfactory due to the compaction procedure. As this was performed by vibration with a low weight hand vibrator in propped conditions, the effect should have been mainly the reduction of earth pressure, and thus of the lateral confining stress, up to the values which are determined by the final friction angle.

figure 5: wires and control panel for reinforcements displacements mesures (Trento Wall)

Nevertheless, the effect of water content seems to have to be taken into account. The presence of a small percentage of water induces in fact an apparent cohesion, that can be extimated around 5-10 kPa for this kind of granular material. Besides, analyzing the shape of the deformed front of the wall under load, it can be observed that the third and fourth strata from the bottom seem to present a smaller deformation than the rest of the wall, as if they had a higher apparent cohesion. Comparison between observed displacements of the front and numerical displacements, calculated by assuming that these two strata possess cohesion is shown (Fig.6). Although this first comparison seems satisfactory from an engeneering standpoint, it is worth noting that a detailed examination of the experimental results, toghether with the chacracterization of the mechanical properties of the filling soil as a function of water content, is required for a definite comparison between calculated and measured displacements, which will be the object of a future work. It can be mentioned, for example, that analyses performed on the same wall in Trento (Benigni, 1995), adopting cable elements for geotextiles and assuming an average cohesion for the whole filling soil, reproduce quite well the displacement pattern of the front of the wall.

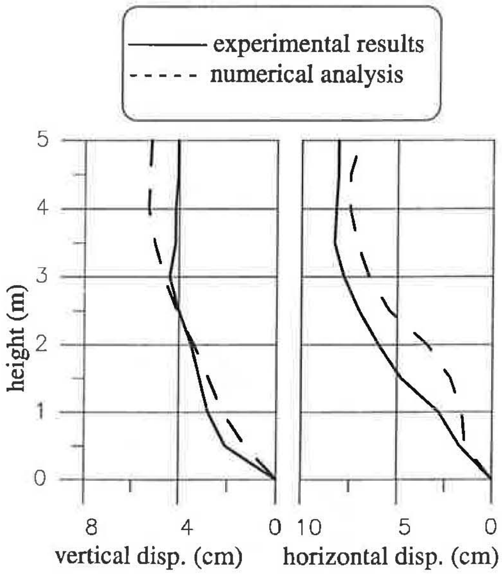


figure 6: measured and calculated displacements of the front of Trento Wall

5. ANALYSIS OF GEOSYNTHETICS REINFORCED EARTH WALL WITH CONCRETE CELLS FACING

We focus on the simulation of two full scale experimentations on walls loaded on the top : the French GARDEN Project (Gourc and al. 95).

The two walls were backfilled with fine sand excavated from the site placed and compacted to a bulk unit weight (γ=19 kN/m^3). One is reinforced with Non-Woven (NW) geotextile and the other with Woven (W) geotextile.

The instrumented embankment (two walls, Fig.7) was 4,35 m high.

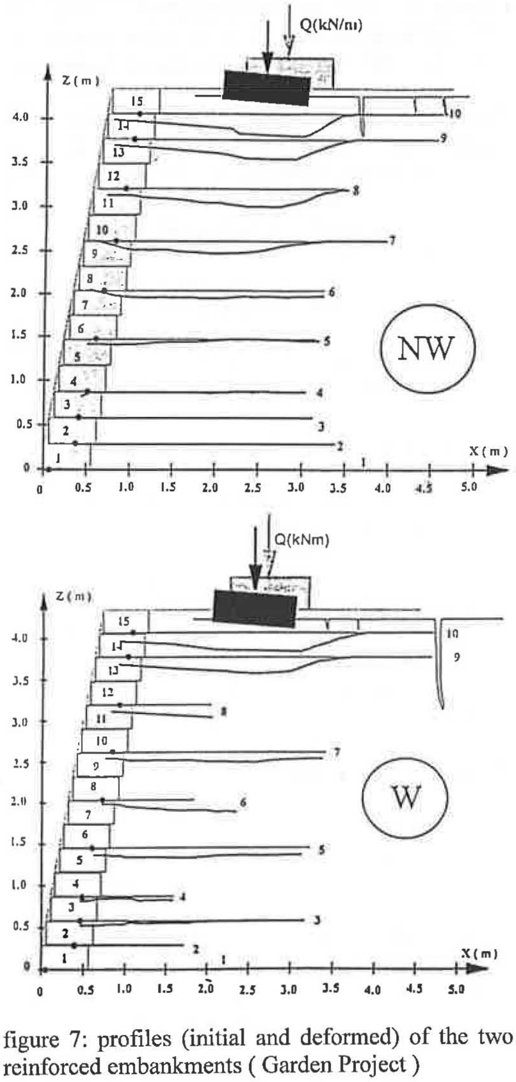


figure 7: profiles (initial and deformed) of the two reinforced embankments (Garden Project)

The reinforcement distribution is not the same, due to the different mechanical properties of the two geotextiles (NW geotextile, failure caracteristics ε_f=30 % and T_f=25 kN/m, secant modulus value at ε=10 %, J=95kN/m and W geotextile, ε_f=15% and T_f=44kN/m, secant modulus J=340kN/m). The Woven reinforcement is therefore 3,5 times stiffer than the Non-Woven one and almost twice as strong.

The embankment is loaded by means of a beam acted upon by two trust rams, each of which is retained by two tie-bars anchored into the foundation ground. The two rams are subjected to the same pressure.

Two months after construction, the additional load was applied on the stab load on the top of the structure until failure occured.

The embankment was well instrumented. The instrumentation included classical measuring and other more original (Gourc and al. 95).

Numerical analysis of these two walls were performed, with the following assumptions :

- quadrilateral soil elements (Fig.8)
- filling soil (γ_d=16,6 kN/m, ϕ=30°, c=2kPa, G=7.600 kPa, K=16.600 kPa)
- foundation soil (γ_d=20 kN/m, ϕ=45°, c=40 kPa, G=23.000 kPa, K=50.000 kPa)
- Geotextiles: cables elements with linear behaviour and interface law
- Cells facing: piled blocks (γ=21,4 kN/m^3, G=71.430 kPa, K=333,330 kPa) with interface (ϕ=30°, c=2 kPa, normal and shear rigidity 14.700 MN)
- cables and cells are linked (attached node)

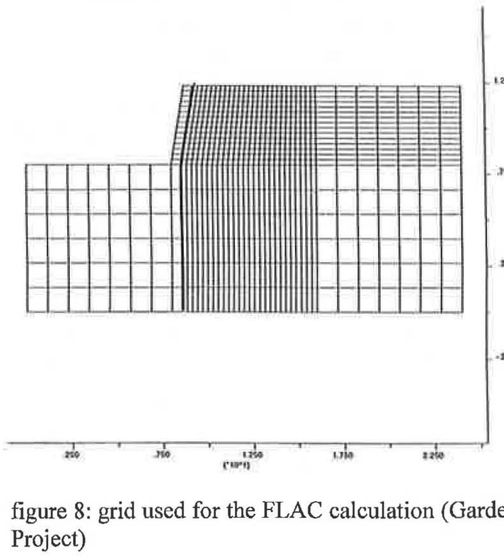
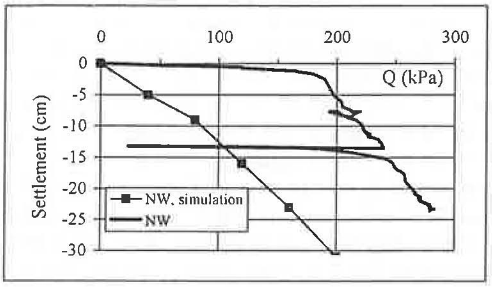


figure 8: grid used for the FLAC calculation (Garden Project)



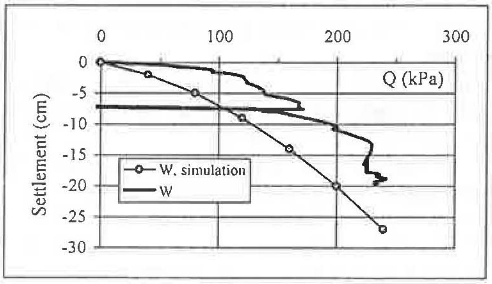


figure 9: Experimental and Simulated loading sequences for the two profiles (Garden Project)

We focus on the central slab load settlements (Fig.9) The diagrams of additional load Q (kPa) versus settlements show significant differences: the numeral simulation give relatively more important displacements. We can see that the evolution is relatively different for the low loading forces Q (Q <170kPa for NW reinforced wall and Q < 150 kPa for the W reinforced wall). but for high loading forces Q the curves have the same gradient. In our opinion, these differences result of the difficulty to simulate the constuction of such walls and particularly the effect of the compaction during the construction step by step. In order to simulate the compaction effect, one solution could be to load and unload with an uniform surcharge every soil layer (Arab et al 96).

6 CONCLUSIONS

The capability of numerical analyses to highlight some relevant aspects of the behaviour of geotextile reinforced earth walls is shown. The analyses may be performed in order to study the influence of the different mechanical and geometrical parameters on the behaviour of the structure. They provide a valuable tool in supporting the traditional design criteria.

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