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Contribution of a finite element model to analysis of geosynthetic reinforced earth retaining walls

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ABSTRACT: This paper presents conventional design, based on slope stability calculation, and Finite Element analysis of a 7 m high reinforced earth retaining wall, constructed by connecting extensible HDPE geogrids to a semi-rigid vertical facing. According to the FE analysis, the maximum tensile force occurs in the fourth geogrid from the base of the wall and is 22 % lower than the value based on conventional design and assuming that the shear strength of soil was fully mobilized. Because the relative stiffness of the reinforcements is lower than the horizontal stiffness of the compacted soil, the soil is close to failure in most of the reinforced earth structure. Therefore, FE analyses of geosynthetics reinforced earth structures mainly depend on the model used to represent the stress-strain relationship of the soil for large strain, close to failure, rather than on the model of interface between soil and reinforcements.

1 INTRODUCTION

Most conventional methods for the design of geosynthetics reinforced earth retaining walls are based on slope stability calculations. Tensile forces in the reinforcements are included in the global moment equilibrium equation. For each potential slip surface, the design tensile forces are assumed to be the values producing global equilibrium of the soil mass above the slip surface, for the factor of safety required on the shear strength of soil. In these analyses, the behavior of soil is assumed to be rigid-plastic and interaction between soil and reinforcements is ignored.

This approach has proven to be safe for most common retaining walls. Nevertheless, in some cases, a deeper understanding of the behavior of geosynthetic reinforced earth structures may appear to be necessary to the design engineer. In those cases, the compatibility of strain in the soil and the in reinforcements, which is the condition for simultaneous mobilization of strength in these materials, may be questioned.

The SUMÔ™ construction process was devised to construct geosynthetic reinforced earth retaining walls with vertical facing. Fig. 1 shows (a) a front view of the wall facing, (b) a horizontal section of the wall and (c) a cross section. The facing is composed of concrete panels. In the manufacturing process of the panels, elements of high density polyethylene (HDPE) geogrid are cast into the concrete. During construction of the wall, the actual reinforcing HDPE geogrids are connected to the elements of geogrid cast into the concrete panels using "Bodkin" connections, a HDPE bar knitting together the two ends of the grids.

This technique was developed in 1995 by a joint venture of three companies: G.T.S. S.A., a contractor constructing earth structures, P.B.M. S.A., the manufacturer of concrete facing panels and NETLON Ltd., producing the TENSAR HDPE geogrids used as soil reinforcements in the SUMÔ technique. After a few experimental SUMÔ walls were built, these companies concluded that a deeper understanding of the behavior of earth reinforced with extensible grids would be useful to improve the design and the construction process of geosynthetic reinforced earth retaining walls with vertical facings.

The "Laboratoire Régional des Ponts et Chaussées" at Lyon (France) was requested to perform such an investigation, focusing on the following three main questions:

1. How to improve construction of planar vertical facings ?
2. What are the tensile forces in the geogrids at the facing and the earth pressure distribution against the facing ?
3. What are the tensile forces distributions along the geogrids ?

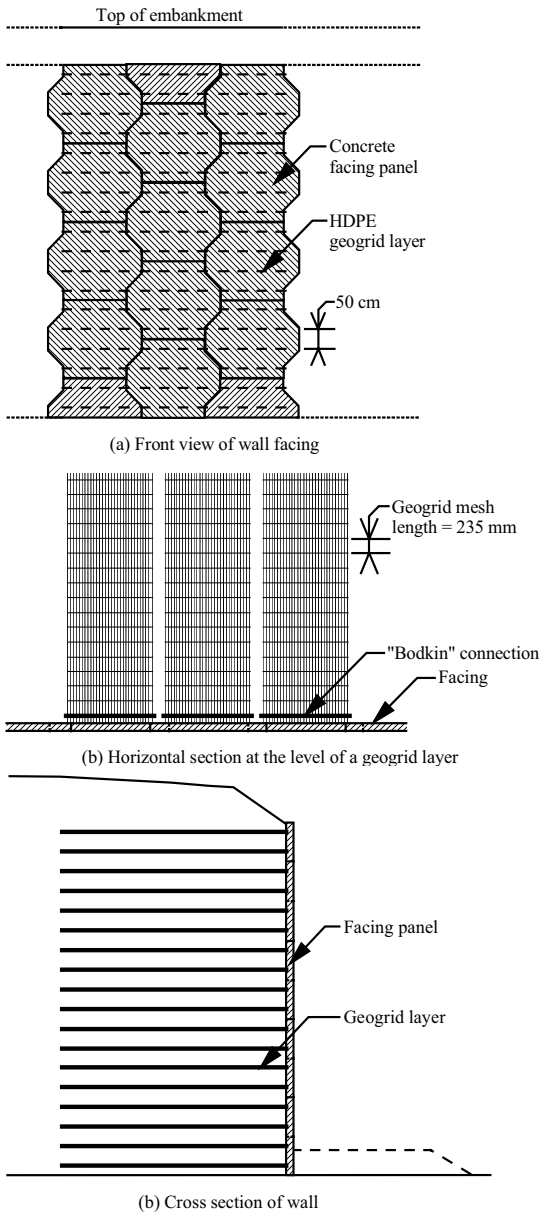

Figure 1. Description of SUMÔ reinforced earth retaining wall.

Clearly, these issues relate to interaction between earth reinforced by extensible geogrids and semi-rigid facing structure. This paper presents this investigation and the information gained from the deformation analysis performed for that purpose.

It must be emphasized that such a study was made possible because SUMÔ walls were well suited for Finite Element analysis. The main reason is that HDPE geogrids are employed in the construction instead of woven geosynthetics, for which displacement is required to start mobilize traction. Values of these displacements in the field are inaccurate and simulation of these displacements in Finite Element analysis would introduce significant uncertainties in the results compared to actual behavior. Similarly, the same reason holds for the connection between the reinforcement and the facing, in which slack must be limited.

Finally, since strains in geosynthetic reinforced earth may be quite large, compared to other reinforced earth structures including steel reinforcements, the model used to represent the stress-strain relationship in soil must account for non-linear and stress-dependent behavior of soils at large strains. This point is also discussed in detail in the paper.

The following is organized in four Sections. Section 2 presents the conventional analysis of a 7 m high SUMÔ retaining wall. Section 3 describes in detail the Finite Element model and results for the same wall. Finally, conclusions are presented in Section 4, including a summary of additional information gained from Finite Element analysis on the behavior of geosynthetic reinforced earth retaining walls.

2 CONVENTIONAL DESIGN ANALYSIS

The ultimate objective of this study was to compare prediction to observed behavior of an instrumented retaining wall to be constructed in an actual road project. Thus, the 7 m high retaining wall was first analyzed according to conventional practice, to define and select agreeable products for an actual wall. This Section presents the major results from the analysis of internal stability of the wall. External and global stability were studied for the geotechnical profile at the site, and are not presented here.

Internal design of the retaining wall was performed using the conventional method implemented in the computer program "CARTAGE" (Delmas et al. (1986)). In this program, tensile forces from the geosynthetics are introduced, for circular potential slip surfaces, in the global moment equilibrium equation of the modified Bishop method and, for non-circular potential slip surfaces, in the global moment equilibrium equation of the Perturbation method (Bell (1968), Raulin & al. (1974)).

Tensile forces in the geosynthetics layers are evaluated as follows. A uniform horizontal displacement of the soil mass above slip surface is first assumed. Given the in-plane stiffness of the geosynthetics reinforcement, the tensile force at intersection with slip surface may then be related to displacement, assuming that shear stress at the interface between soil and geosynthetics increases linearly with relative displacement until failure. In the CARTAGE program, displacement of the soil mass above slip surface is then iterated until the global moment equilibrium equation is satisfied, for the required value of the factor of safety on the shear strength of soil.

Fig. 2 shows the cross section of the retaining wall for the conventional analysis. The wall is 7 m high, the geogrids are 4.50 m long and the spacing between geogrid layers is equal to 0.50 m. The unit weight of the soil used to construct the wall (γ) was assumed to be equal to 20 kN/m^3, and the shear strength was defined by a friction angle (φ') of 38°.

In this design, the geogrids have been considered attached to the facing. The in-plane stiffness of the geogrids (k_{tex}) has been assumed to be equal to 1000 kN/m. To define the interface between the soil and the geogrids, the maximum relative displacement was taken equal to 2 cm and the friction angle defined according to equation 1:

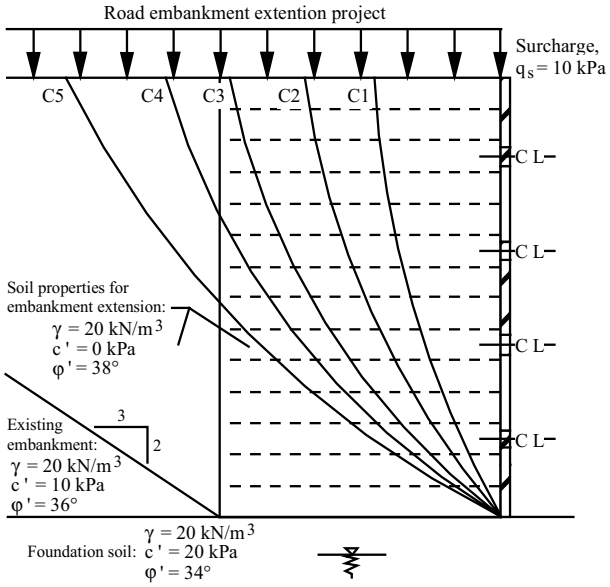

Figure 2. Cross section of retaining wall project for internal design.

$$\tan(\varphi_{soil/tex}) = 0.7 \times \tan(\varphi') \quad (1)$$

where $\varphi_{soil/tex}$ = friction angle between soil and geogrid.

A representative subset of five slip surfaces considered in this analysis, C1 to C5, are shown in the calculation profile in fig. 2. Partial results of this conventional analysis are presented in Table 1. Each entry in the table corresponds to a potential slip surface, C1 to C5. For each slip surface, the second column gives the maximum value of the tensile forces in the geogrids resisting to the failure for a required factor of safety on the shear strength of soil equal to 1.5 (F_{soil}=1.5), the third column to 1.35 and the fourth column to 1.

Table 1. Maximum tractions (kN/m) for different values of F_{soil}.

Slip surface	F_{soil} = 1.5	F_{soil} = 1.35	F_{soil} = 1
C1	21.7	19.8	**14.6**
C2	22.5	**20.3**	13.5
C3	21.7	18.9	10.9
C4	22.9	19.5	8.6
C5	**23.8**	17.9	n/a

No partial weighting was applied to the parameters in this conventional model. For a factor of safety equal to 1.35 on the shear strength of soil, the design tensile force in the geogrids would be of the order of 20.3 kN/m, at least in the lower part of the wall, where the load is maximum. It may also be noted that, as far as tensile forces in the geogrids are concerned, the critical slip surface depends on the required value of the factor of safety on the shear strength of the fill material.

The long term strength of the actual geosynthetics product used for building the wall will further depend on the creep, aging and construction damage properties of the product. TENSAR SR110 HDPE geogrids were selected to build this instrumented wall. Properties of these geogrids are used in the analysis below.

3 FINITE ELEMENT ANALYSIS

This section presents the prediction of the deformation behavior of the geosynthetics reinforced earth retaining wall described in Section 2. The Finite Element (FE) code used to simulate the wall construction was the program "SAGE", developed by Morrison & Duncan (1995), for Static Analysis of Geotechnical Engineering problems.

3.1 *Model*

Fig. 3 is a schematic cross section assumed for the future road embankment extension project. A plane strain model was developed corresponding to that cross section. Fig. 4 shows the detail of the FE mesh about the reinforced earth retaining wall, in the last construction step. The complete mesh, corresponding to an 80 m long model centered about the wall facing, was composed of 920 nodes and 1081 elements.

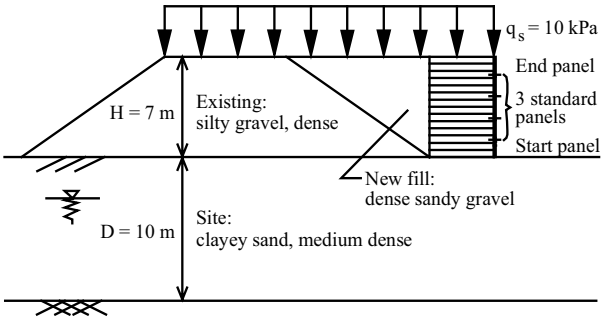

Figure 3. Soil types in schematic cross section of project.

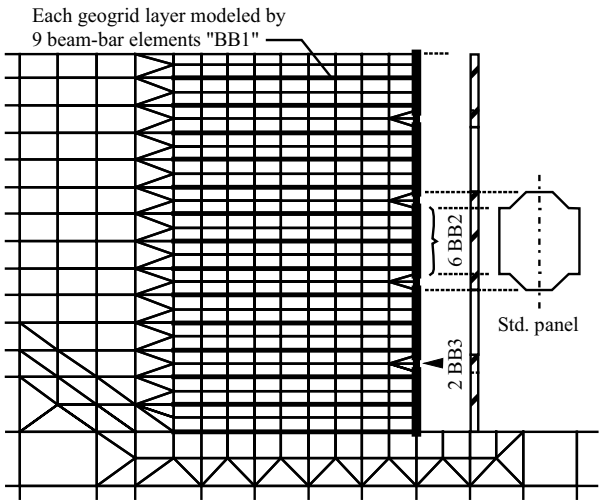

Figure 4. Detail of Finite Element mesh about the wall.

The hyperbolic model (Duncan & al. (1980)) was used to represent the behavior of soils in the profile. This model was developed to represent the behavior of compacted soils in deformation analyses of large earth dams, where significant strain may occur. Therefore, the hyperbolic model appeared to be particularly well suited for analyses of geosynthetics reinforced earth structures.

Typical values of hyperbolic model parameters for different soil types, like those identified in fig. 3, are given by Duncan & al. (1980). Table 2 gives the values of the soil properties and hyperbolic models parameters for the three soil types in the profile. In the table, γ represents the total unit weight of the soil and K_0 the at rest earth pressure coefficient. These two parameters are used to evaluate initial stresses, in particular when soil elements are placed during a construction step. Parameter c' is the soil effective cohesion, φ' the effective friction angle and T_S a possible tensile strength in the soil.

The next six columns in Table 2 contain the values of the six parameters of the hyperbolic model. The first four (R_f, K, K_{ur} and n) define the hyperbolic equation used to represent the relationship between stress and strain, depending on confining pressure and deviator stress. The last two (K_b and m) are used to describe non linear volumetric strain in soil, depending on confining pressure.

Table 2. Parameter values of hyperbolic models representing soils.

Soil\prop. (-)	γ (kN/m^3)	K_0 (-)	c' (kPa)	φ' (°)	T_S (kPa)	R_f (-)	K (-)	K_{ur} (-)	n (-)	K_b (-)	m (-)
Site	19.5	0.5	20	34	10	0.7	200	400	0.6	150	0.5
Existing	20	0.7	10	36	5	0.7	400	600	0.4	100	0.2
New fill	20.5	0.7	0	38	0	0.7	700	1000	0.4	200	0.2

Table 3. Properties of beam-bar elements in model (per meter of wall).

Location (-)	WL ((kN/m)/m)	EA (kN/m)	EI ($kN{\cdot}m^2$/m)
Geogrids (BB1)	0.01	950	1
Facing panels (BB2)	3.5	3.36×10^6	5500
Interlocking (BB3)	3.5	3.36×10^6	55

Detail of the mesh in fig. 4 shows the three types of beam-bar elements, BB1 to BB3, used in the model to represent the geogrids, the facing panels and the interlocking between two successive panels. The properties of beam-bar elements, unit weight (WL), axial stiffness (EA) and bending stiffness (EI), are summarized in Table 3.

The axial stiffness of geogrids was derived based on results of creep tests at constant temperature on the SR110 HDPE geogrids, employed in the instrumented wall project. Fig. 5 shows the one week, one year and 10 year traction-deformation isochrones, derived from the creep tests. Axial stiffness was assumed to be the slope of the one week curve for tensile forces lower than 25 kN/m. Since one week was the expected duration of the wall construction, at the end of construction, strain in the geogrids in the lower part of the wall would be of the order of those corresponding to the one week isochrone. Post construction deformation of the facing would relate to strain between the one week curve and the curves for larger values of time.

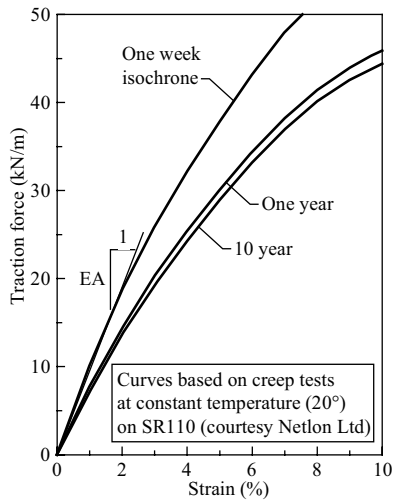

Figure 5. Traction-deformation isochrones for geogrids.

For the facing panels, made of reinforced concrete, axial and bending stiffness were derived considering the rectangular shape of their horizontal cross section. For the part corresponding to interlocking of panels, axial stiffness was assumed to be the same as that of panels and bending stiffness 1/100 of that of panels.

Although interfaces exist between soil and geogrids and between soil and facing, the FE model used in this analysis did not include interface elements. This approach assumes that the relative stiffness of the reinforcements (the axial stiffness divided by the vertical spacing of the geogrids) was lower than the horizontal stiffness of the compacted soil. This assumption is discussed in the next section, presenting the results of the calculations.

The numerical model consisted of 19 calculation steps. The first four were used to calculate initial stresses in the foundation

soil and in the existing embankment. The last 15 simulated progressive construction of the wall and final loading at the top.

3.2 *Results*

The displacements of the facing panels during construction are shown in fig. 6. When new elements are activated during a construction step, the program sets to zero the initial displacements of new nodes. To draw this figure, each panel has been assumed to be installed vertically on top of the preceding. Except at the base of the wall, where the foundation soil impedes the horizontal displacements, the facing is moving outwards rather uniformly during construction, up to about 7 cm at the top of the wall. This value is in good agreement with displacements observed by the companies in several similar experimental walls.

The distributions of tractions mobilized in the geogrids have also been represented in fig. 6. According to the model, tensile force along each geogrid is rather uniform. Maximum traction occurs in the fourth geogrid from the base of the wall, rather than at the base, because foundation soil reduces horizontal displacements near the base. The maximum tensile force is of the order of 11.5 kN/m, 22 % lower than the value calculated using CARTAGE for F_{soil}=1 (14.7 kN/m) and 43 % lower than the value for F_{soil}=1.35 (20.3 kN/m).

Finally, fig. 7 shows the stress level at the end of construction in soil elements composing the wall. In the hyperbolic model, the stress level is defined as the ratio of the current shear stress divided by the shear strength at failure, the asymptote to stress-strain hyperbola related to current confining pressure. The stress level ranges from about 100 % near the facing to about 85 % near the end of the geogrids inside the reinforced structure.

This results shows that the modeling approach, without interface element between the soil and the geogrids, was appropriate. Slippage between soil and geogrids would reduce the tractions in the reinforcements and thus increase the stress level in the soil. The soil being close to failure in the whole structure, this would not be possible.

It must therefore be emphasized that the results from FE analyses of geosynthetics reinforced earth structures mainly depend on the model used to represent the stress-strain relationship of the soil, in particular for large strain, close to failure.

Deformation analysis also indicates that earth pressure against the facing increases linearly with depth, to 23 kPa at the base of the wall, corresponding to an "active reinforced earth pressure coefficient" approximately equal to 0.16. Tractions in the geogrids at the facing are significant, with the maximum value of the order of 9 kN/m.

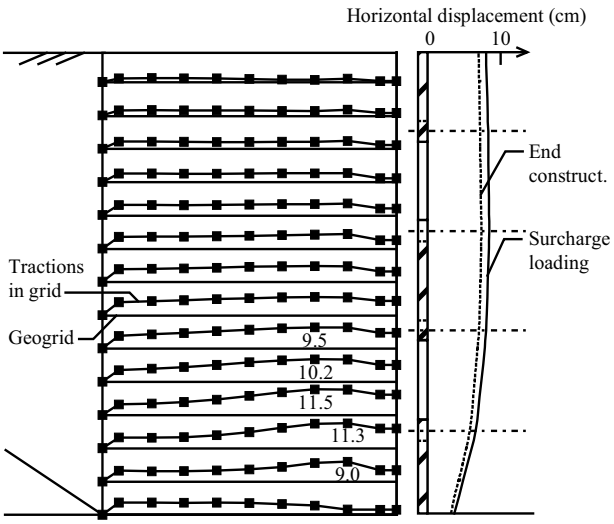

Figure 6. Calculated displacements of facing and tractions in geogrids.

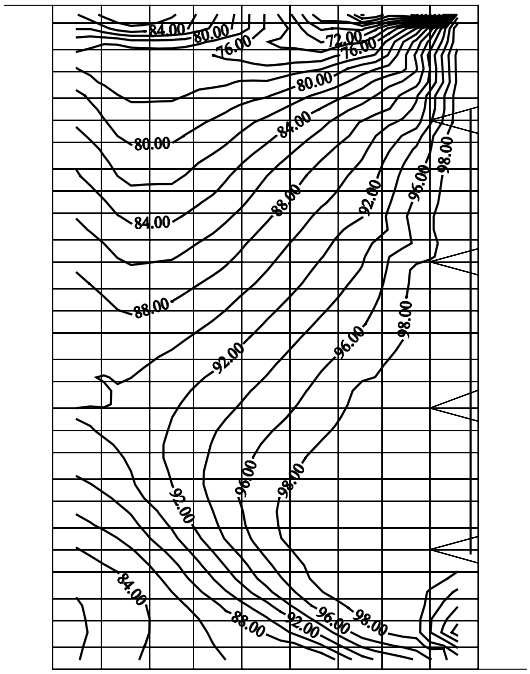

Figure 7. Stress level in soil elements at the end of construction.

4 CONCLUSION

A 7 m high reinforced earth retaining wall, constructed by connecting extensible geogrids to a semi-rigid vertical facing structure, has been studied. Conventional design, using CARTAGE program, and analysis based on a FE model were performed. The main results and conclusions are presented below.

According to the FE analysis, distributions of tensile forces in the geogrids are rather uniform. The maximum tensile force occurred in the fourth geogrid from the base of the wall and was 22 % lower than the value calculated using CARTAGE and assuming that the shear strength of soil was fully mobilized.

During construction, the facing is moving outwards rather uniformly, except near the base of the wall, where the foundation soil reduces horizontal displacements.

The relative stiffness of the reinforcements is lower than the horizontal stiffness of the compacted soil. As a consequence, the soil is close to failure in most of the reinforced earth structure. Therefore, FE analyses of geosynthetics reinforced earth structures mainly depend on the model used to represent the stress-strain relationship of the soil for large strain, close to failure.

Finally, as FE analyses of geosynthetics reinforced earth structures probably will become more frequent, it must be emphasized that models should fit the construction processes.

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