REDUCING DIFFERENTIAL SETTLEMENTS UNDER A ROAD EMBANKMENT IN KOREA BY USE OF GEOSYNTHETICS: A FINITE ELEMENTS ANALYSIS

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ABSTRACT

The paper deals with the problem of reducing differential settlements under a road embankment over compressible soil through the use of Geogrids.

The technical solution, the geotechnical characterisation of the involved soil layers and the settlements calculation with a Finite Elements Analysis are reported. Moreover the calculation results, in terms of expected displacements and tension stresses in the reinforcements are presented and briefly commented.

INTRODUCTION

The road to Pusan airport is being enlarged from 14 m (7 m old road + 7 recently widened road) to 28 m. The road lays on a 10m-thick Silt layer and on a 10÷20m-thick Clay sublayer. About 2.0 meters of well compacted granular soil are being interposed between the asphalt layer and the ground level.

Both Silt and Clay layers showed remarkably high compressibility parameters, making it necessary to carefully regard at the possibility of localized settlements. In fact, due to the different loading history, a different mechanical behaviour of the underlying soil is expected.

First a 7 m wide road was built beneath a few meters high embankment: this conditions led to a good consolidation of the involved soil. Then a 7 m wide enlargement was recently built without any consolidation preloading: in this area larger settlements are therefore supposed to develop.

Finally a further 14 m enlargement, vertical drains and a consolidation preloading, lasting 9 months, were executed.

The designer expectations were to achieve up to 90% of consolidation at the end of the preloading stage: for this reason smaller settlements were expected in this area.

REINFORCING LAYER

When designing foundation structures that will have to sustain concentrated loading, as for a large road, two main problems should be focused.

First of all the possibility of unsustainable settlements has to be considered, a large amount of consolidation settlements should be expected under both the first and the second enlargement.

The execution of vertical drains and the application of preloading for nine months under the new enlargement will surely contribute to improve the situation; nevertheless the possibility of soil heterogeneity and the different loading history suggest the insertion of reinforcing tensile resistant elements such Geogrids.

The possibility of tension cracking of the asphalt pavement should be expected too: in fact, due to the high stiffness of the pavement compared with the high compressibility of the lower soil, localized cracking could generate. Also for this reason, high stiffness well compacted stabilized should be coupled with Geogrids.

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As it is well known in scientific literature, Geogrids can remarkably improve foundation structures behaviour (Cancelli et al., 1996; Cancelli et al. 1997; Cancelli and Montanelli, 1999) by means of their confining effect and, at the same time, of the tensioned membrane effect (Giroud et al., 1984; Koerner, 1986); thus reducing shear and tension actions on stiff foundation structures.

Their capability of spreading concentrated loads could be even more appreciated for every problem in which large loading are applied over small areas.

In this problem the behaviour of two extruded PP Geogrids, 40 kN/m peak tensile strength (type TENAX LBO 440) has been studied.

Furthermore a non woven geotextile (100 g/m²) should be inserted at the interface between stabilized soil and the Silt layer in order to prevent sinking of frictional material into lower Silt.

The proposed layout for the reinforcing elements is sketched in Figure 1.

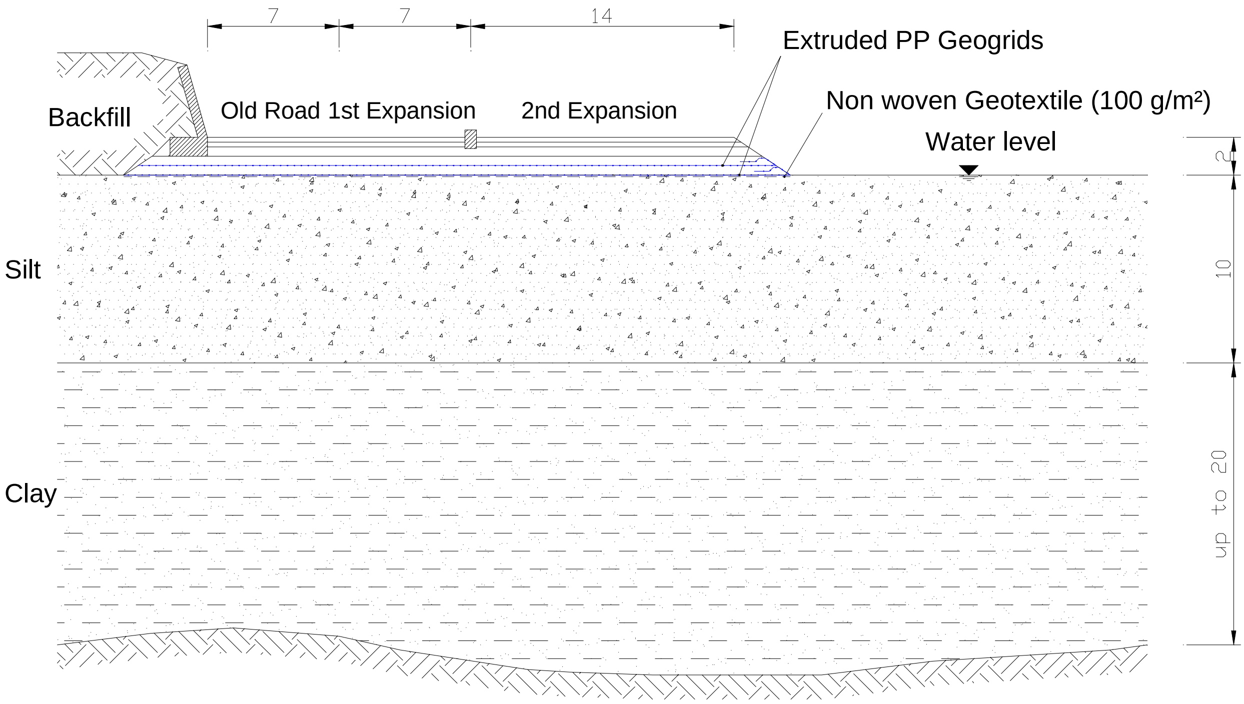

Figure 1: Design Layout

GEOTECHNICAL PROPERTIES OF THE SUBSOIL

The results of several lab tests (from boreholes n° 7 – Old Road area – and n° 14 – Recent and New expansions) were analysed, in order to achieve all the input data that were necessary for Finite Elements Analyses.

Basically, three kind of parameters were needed: Shear Strength parameters, Elastic deformability parameters and Consolidation deformability parameters, being this last group the most important in order to evaluate the final settlements of this road embankment.

The interpretations of tests results on Silt samples coming from boreholes n° 7 and n° 14 have been kept separate: the loading history of these two areas are remarkably different, being the old road area much more consolidated than the recent and new expansions area; this circumstance is clearly recognizable by comparing the consolidation tests results.

On the contrary, being the dimension of the Old Road cross section rather small (7 m), the Clay layer has been considered not influenced by the previous loading: for this reason in the following geotechnical characterisation it has been defined as a single material.

The determination of the required geotechnical parameters will be therefore related to the following materials: Silt from Borehole n° 7, Silt from Borehole n° 14, Clay and a single material representing at the same time the Stabilized soil and the pavement material.

For the last material no information was available; for this reason the input data that were used for the numerical analysis can be considered as typical values commonly used in road engineering calculations.

Shear Strength Parameters

Shear Strength parameters were determined directly from Triaxial Tests: Consolidated Undrained (CU) and Unconsolidated Undrained (UU), respectively giving parameters expressed in terms of effective stresses (c' and φ') and in terms of total stresses ($c_u \neq 0$ and $\varphi_u = 0$).

This assumption didn't lead to any additional complication since the Software used for calculation allows both the Input for independently total and effective parameters. The resulting Shear Strength parameters are resumed in Table 1:

Table 1: Shear Strength Parameters

Material	c_u (kPa)	c' (kPa)	φ' (°)
Silt 7	-	0	25
Silt 14	27	-	-
Clay	31	-	-
Stabilized + Pavement	-	0	38

Elastic Deformability Parameters

In order to evaluate the Elastic Deformability parameters results from Triaxial Tests: Consolidated Undrained (CU) and Unconsolidated Undrained (UU) were available. Nevertheless the resulting moduli were spread on a very wide range of values (i.e. for the Clay layer at about the same stress level E^u_{50} was ranging from 660 to 5995 kPa) : therefore they were considered to be unreliable.

As a rough approximation, in the numerical analysis for E'_{50}(ref) were adopted the same values of the corresponding M_{ref} (see after).

For every soil layer, as a typical value used in common Computational Geotechnics, a Poisson's Ratio $\nu = 0.33$ was assumed.

Consolidation Deformability Parameters

Consolidation Deformability parameters were, of course, determined directly from Consolidation Tests. The first consideration that was done, being all the soil layers more or less normally consolidated, is that the dominant parameter that rules the Deformability is the Primary Compression Index.

From the determined values of the Initial Void Ratio e_o and the Primary Compression Index Cc it has been possible to obtain the Compression Ratio CR being:

$$CR = Cc(1+e_0) \tag{1}$$

From the Compression Ratio it can then be obtained the Oedometer Modulus for whatever stress rate level:

$$M = 1/m_v = \sigma'_v / 0.435*CR \tag{2}$$

and, of course:

$$M_{ref} = 1/m_v = p_{ref} / 0.435*CR \tag{3}$$

being $p_{ref} = 100$ kPa.

In this way every Consolidation Deformability parameter could be determined as reported in Table 2.

Table 2: Consolidation Deformability Parameters

Material	CR	M (kPa)	M_{ref} (kPa)
Silt 7	0.2109	1257	1090
Silt 14	0.2622	906	877
Clay	0.2057	1520	1118

As it can be clearly seen from the table, the Consolidation Deformability parameters are remarkably higher in the Old Road area (Borehole n° 7) than in the Recent and New Expansions Area (Borehole n°

14), showing how the consolidation process strongly depends from the loading history, thus confirming the hypotheses previously done.

In order to simulate the stiffening of the soil due to the presence of an embankment on the left hand side of the Old Road area, a surcharge loading was applied by means of a fictitious value of the soil unit weight. A value of $\gamma^* = 42$ kN/m3 instead of 21 kN/m3 was therefore applied to the soil layer besides the stabilised base layer.

FINITE ELEMENTS ANALYSIS

The complexity of a Finite Elements Analysis requires a noticeable number of hypothesis to be done in order to completely characterize the problem: in this paragraph some of the most important ones are summarized.

As in common engineering practice, in order to evaluate the stability conditions of long strip structures, a plane-strain analysis is performed.

According to the options available with the software (PLAXIS – Holland) used in the analysis, every soil element has been described with quadratic 6-node triangular elements (the same type of elements were used for the stabilised base layer and the pavement).

In representing geogrid elements, 3-node tension elements were used. Since it's also necessary to model the interaction between soil and geogrid, special interface 3-node elements were adopted. As it's been demonstrated from experimental researches (Cancelli et al., 1992) reduced values of interface friction angle should therefore be assigned to these elements.

In order to represent the soil constitutive law the Hardening Soil model was used (Brinkgreve and Vermeer, 1998): this is an elastoplastic type of hyperbolic model (Duncan and Chang, 1970), formulated on the base of hardening plasticity. Furthermore the mechanical behaviour of both the stabilised base layer and the pavement was described as a single material with a Mohr-Coulomb model.

In order to correctly simulate the loading history some simplifying hypotheses were necessary.

The greatest problem was to simulate the initial conditions in order to describe the different loading history that brought to different levels of consolidation in the three areas.

The adopted calculation scheme is articulated in two phases, namely Stage 1 and Stage 2 (Fig. 2): the first stage was necessary to simulate the initial conditions, it was analysed as a long term analysis, in order to have any excess pressure fully dissipated. At the end of this stage the remaining parts of the embankment weight were applied and in the Stage 2 another long term analysis was performed, in order to evaluate the long term settlements that will develop at the end of the whole Consolidation process.

During Stage 1 the full consolidation of the Old Road area under the whole height of the embankment and the working surcharge load were achieved (because the entire consolidation process in this area is supposed to be finished). The first expansion has been consolidated only for one half of the embankment height (because the consolidation process is far from been concluded), while the new expansion has been consolidated under the 90% volume of the embankment (in this way the 90% preconsolidation has been roughly simulated).

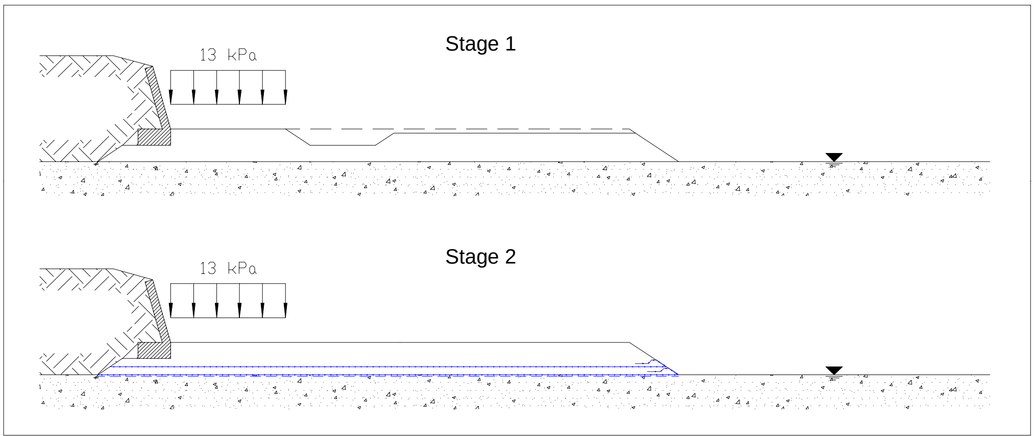

Figure 2: Calculation stages

During Stage 2 the remaining parts of the embankment height was added. Prior to running the calculation also the Geogrids were then inserted.

Since at the end of Stage 1 every displacement was set to zero, at the end of this phase of the calculation only the settlements after the construction could be obtained.

At last, in order to allow a conservative evaluation of the maximum stresses in the Geogrids, the total consolidation under the full surcharge loading was studied too.

RESULTS AND CONCLUSIONS

Some drawings, featuring the most relevant calculation results, are enclosed in order to allow a better understanding of the present chapter.

The calculation results fully confirmed the expectations: the Recent Expansion area is supposed to be subjected to the largest settlements (maximum displacement 119 mm) while the Old Road area and the New Expansion area should undergo to settlements of about 50 to 80 mm.

As it can clearly be seen in Figure 3 - Deformed Mesh (the displacements were amplified for a factor of 20) - the presence of Geogrids shows a satisfactory behaviour: the effect of the reinforcement shows a remarkable spreading of the expected settlements all over the section of the road, thus limiting shear and tension actions on the pavement.

The spreading of the deformations along the entire cross section results therefore in a strong reduction of the possibility of tension cracking of the pavement.

The resulting rotations of the pavement cross section can be therefore evaluated as 0.7% in the Old Road area and 0.45% in the New enlargement area: such rotations could be judged fully compliant with the required values by common road engineering.

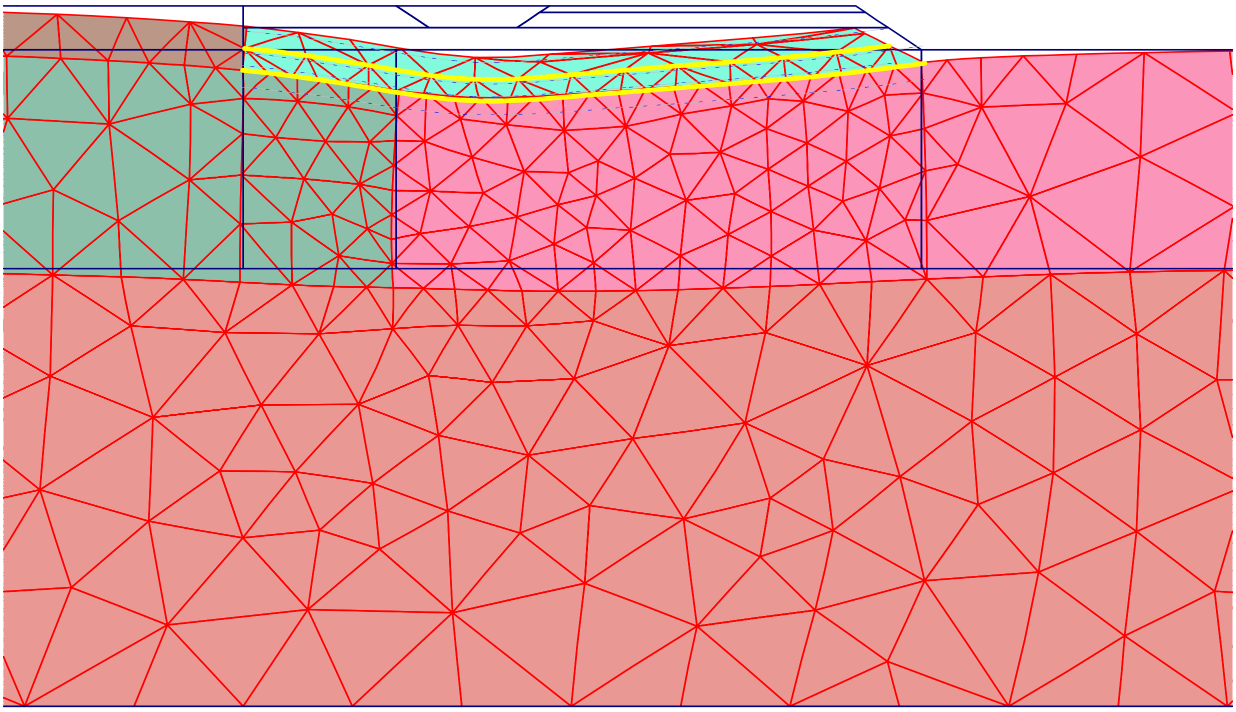

Figure 3: Deformed Mesh (scaled 20 times)

After evaluating the conditions related to Stage 2, one more calculation stage (Stage 3) was performed in order to analyse the stresses in the Geogrids for a definitively conservative condition: the full consolidation of the soil under a uniform surcharge loading of 13 kPa on the whole cross-section.

This condition is clearly unrealistic being the dimension of the loaded area large enough to involve the Clay layer which is much less permeable than Silt. Therefore very long time of permanent load would be necessary to consolidate, that is very difficult to occur under ordinary traffic loads.

In this way it has been possible to compare the expected tension forces in the Geogrids for both Stage 2 and Stage 3: the results are reported in Figure 4.

For conditions related to Stage 2 the Finite Elements Analysis allowed to predict a maximum tension force of about 2 kN/m while for Stage 3 a value of about 5 kN/m was determined. These results are fully

compliant with the long term serviceability conditions of the Geogrid, providing a Factor of Safety FS ≅ 2 since the Long Term Design Strength of the employed Geogrid is 10 kN/m (25 % the peak value).
As it can be clearly seen from the drawing, the maximum values are expected to occur in the same position, that is the first enlargement area where maximum displacements are expected (see Figure 3).

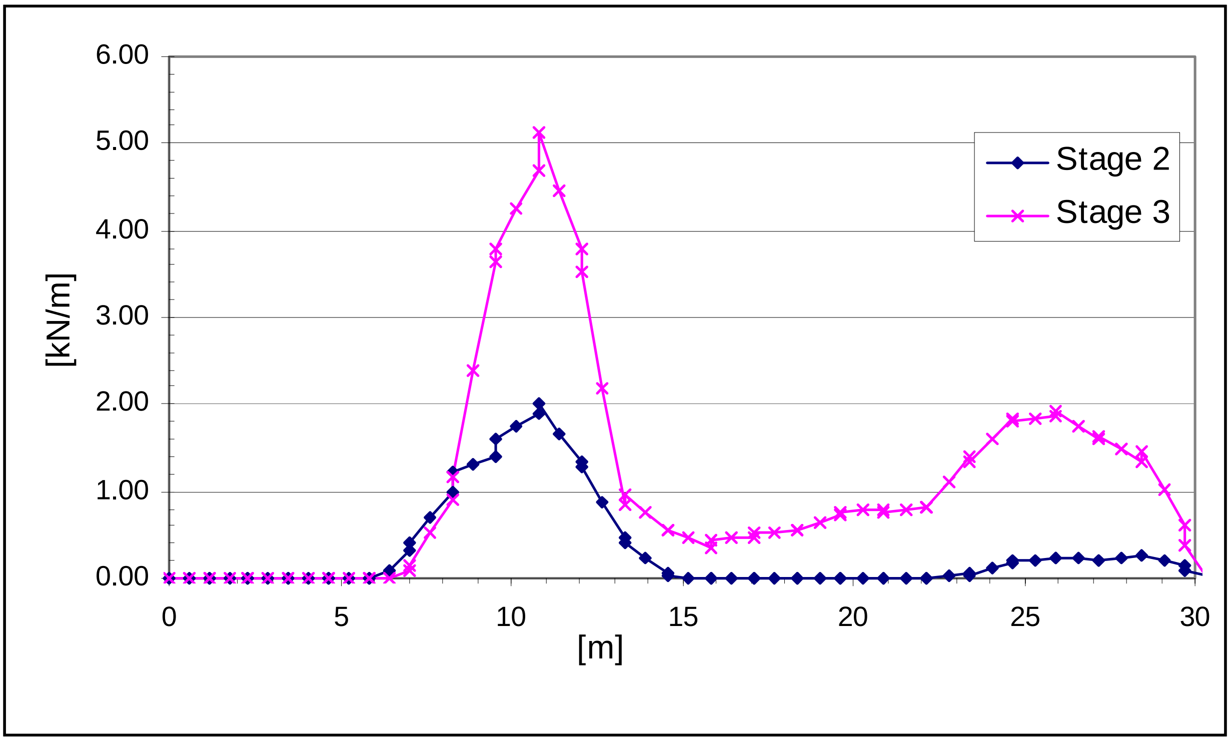

Figure 4: Tension forces in the lower Geogrid

It is worth noting how the tensile strength in the left side of the geogrid reduces almost to 0.00 kN/m. This left side part of the geogrid is the anchorage length, necessary to ensure the transmission of the foreseen tensile strength to the other part of the geogrid and to prevent pull-out of the geogrid.

REFERENCES

Brinkgreve, R. B. J. and Vermeer P. A. (1998) *Plaxis User's Manual*, Balkema, Rotterdam.

Cancelli, A.; Rimoldi, P.; Togni, A. (1992) "Frictional characteristics of geogrids by means of direct shear and pull-out tests", *Proc. of IS Kyushu 92 Int. Symposium on Earth Reinforcement Practice*, Fukuoka, Japan.

Cancelli, A., Montanelli, F., Rimoldi, P., and Zhao, A., (1996). "Full scale laboratory Testing on geosynthetics reinforced paved roads" *Proc. of the International Symposium on Earth Reinforcement*, IS KYUSHU 1996, Japan.

Cancelli, A., Cancelli, P., Recalcati, P., Rimoldi, P. (1997). "The use of Geosynthetics to Improve the Bearing Capacity of the Foundation of Road Embankments", *IGS International Conference Geosynthetics Asia '97*, Bangalore, India.

Cancelli, A., Montanelli, F. (1999). "In ground test for geosynthetic reinforced flexible paved roads", *Proc. Geosynthetics '99 Conference*, Boston, USA.

Duncan, J. M. and Chang C. Y., (1970). "Nonlinear Analysis of Stress and Strain in Soil." *ASCE J. of the Soil Mech. and Found. Div.*, Vol. 96.

Giroud, J.P., Ah-Line, C., Bonaparte, R. (1984). "Design of unpaved roads and trafficked areas with Geogrids" *Polymer grid reinforcement*. Thomas Telford Ltd., LONDON, UK

Koerner, R.M. (1986). *Designing with Geosynthetics*, Prentice-Hall, New Jersey.