ANALYSIS OF LONG-TERM PERFORMANCE EMBANKMENT REINFORCED BY GEOGRID

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ABSTRACT

Applications of the geosynthetic materials as reinforced elements to strengthen of the embankments slope has a great world practice. The purpose of the numerical analysis was research of the stress-strain conditions of the long-term performance embankment reinforced by geogrid. The comparison of numerical and monitoring results of horizontal and vertical deformation of reinforced and unreinforced section of tested embankment was presented in paper as well as results of stress and strain development, consolidation process of tested embankment after elapse a long term. The obtained results of the researching work corroborated the hypotheses that reinforcement is one of the reliable and effective soil improvement models.
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Keywords: Geogrid, Plaxis 2D, settlement monitoring

INTRODUCTION

The concept of earth reinforcement can be traced back to the ancient history. First primitive application of reinforcement was using sticks or branches for the mad dwelling. Modern conception of reinforcement as one of engineering technology has origin since the 1960`s. Technology of reinforcement is developed through development of mankind. Nowadays there are a lot of different types of reinforcement materials are used in a world engineering practice. In spite of that people try to find new technology and materials for reinforcement which might be more effective and practically feasible. That the earth reinforcement has prevalence on the other earth improvement technology is undoubted fact, but what will happen with reinforced construction after essential elapsing of time? What the role of the reinforcement for the long-term performance of construction? What the influence for the stress-strain behavior of reinforcement?

For that purpose and full understanding of the long term performance of reinforced embankment behaviour in 1986 year was constructed artificial embankment by fine-grained coherent soil reinforced by geogrid (Zhussupbekov 2012).

REINFORCED EMBUNKMENT BACKGROUND

The researching work and design of the reinforced embankment was carried out by engineers of Alberta University. Testing embankment is 12 m of high, inclination of 1:1. Embankment consists of four sections, three of them reinforced by different type of geogrid and fourth section non reinforced, see Fig. 1.

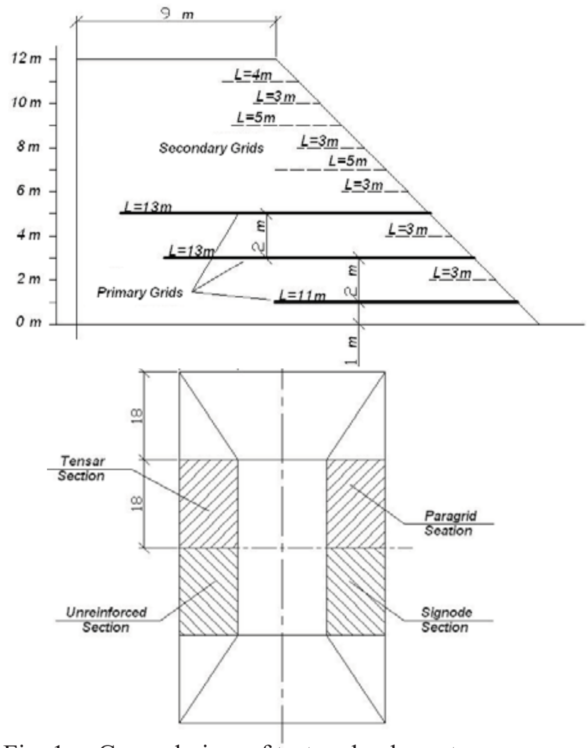

Fig. 1 General view of test embankment.

The foundation soil was studied by Hofmann (1989). Four boreholes were drilled at the test fill site using a wet rotary drill rig which allowed Shelby tube samples, 73 mm in diameter by 610 mm long, to be taken. The shelby tubes were taken by Hofmann about depth of 10 m in borehole 1, 3 and

4. In borehole 2, however, drilling was continued down to clay shale bedrock at 27 m, without sampling.

Examination of the material removed as drilling progressed showed that the grey sand was continuous to the bedrock. Alberta Transportation and Utilities drilled and logger a total of 70 boreholes at various locations along the new alignment of Highway 60 to the depth at 20 m. A typical borehole profile is shown in Fig.2 and results of laboratory tests of the foundation and filling soils obtained by Hofmann and Alberta Transportation is shown in Table 1 and Table 2. The groundwater table is 5 m below the ground surface (Hofman 1989).

Table 1 Properties of upper foundation soil and filling soil

Properties of soil	Index	Value	Unit
Atterberg Limits Tests			
Water contents	W_n	36,4	%
Liquid Limit	Ll	46,7	%
Plastic Limit	Lp	21,5	%
Plasticity Index	IP	25,2	%
Dry Unit Weight	γ_{drv}	18	kN/m
Saturated Unit Weight	γ_{sat}	20	kN/m
Grain Size Distribution Tests			
Sand	-	5	%
Clay	-	20	%
Silt	-	75	%
Consolidation Tests (Stress Range 800kPa, P_c` - 458			
Coefficient of Consolidation	C_v	0,001	cm/s
Compression Index	C_c	0,535	-
Recompression Index	C_r	0,053	-
Time factor	t_{90}	2.43	min
Coefficient of Volume	M_v	1.41	m/кН
Coefficient of Permeability	k	1.03	cm/s
Triaxial and Direct Shear Tests			
Sell Pressure	σ_1	518	kPa
Density	ρ_d	1.859	g/cm
Void Ratio	e	1.894	-
Degree of Saturation	S_r	84.0	%
$(\sigma_1 - \sigma_3)$	$(\sigma_1 -$	427	kPa
Strain	ε	8.1	%
Friction Angle	φ	24	°
Cohesion	c	23	kN/m
Elastic Modulus	E	3500	kN/m
Poison`s Ratio	ν	0,4	-

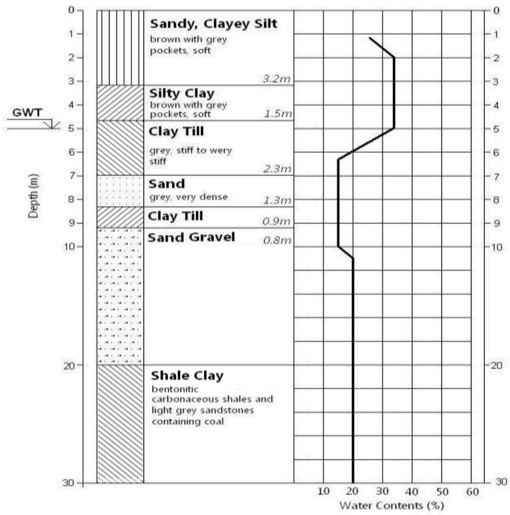

Fig. 2 Profile of foundation soil.

Table 2 Properties of filling soil

Properties of soil	Index	Value	Unit
Atterberg Limits Tests			
Water contents	W_n	20 -	%
Liquid Limit	Ll	37.4	%
Plastic Limit	Lp	20.9	%
Plasticity Index	IP	16.5	%
Dry Unit Weight	γ_{drv}	17	kN/m
Saturated Unit Weight	γ_{sat}	20	kN/m
Grain Size Distribution Tests			
Sand	-	20	%
Clay	-	18	%
Silt	-	62	%
Consolidation Tests (Stress Range 800kPa, P_c` - 458			
Coefficient of Consolidation	C_v	54 -	cm/s
Time factor	t_{90}	23 -	min
Normal Stress	σ	200	kPa
Triaxial and Direct Shear Tests			
Sell Pressure	σ_1	80 -	kPa
Density	ρ_d	1.7 -	g/cm
Void Ratio	e	0.59 -	-
Degree of Saturation	S_r	91 –	%
$(\sigma_1 - \sigma_3)$	$(\sigma_1 -$	129-	kPa
Strain	ε	15.0	%
Friction Angle	φ	28	°
Cohesion	c	20	kN/m
Elastic Modulus	E	28000	kN/m
Coefficient LEP (σ_1 / σ_3)	k	2.3 –	-
Poison`s Ratio	ν	0.35	-

For the reinforcing of the test embankment it was chosen to use three types of high tensile strength geogrids: Tensar SR2, Signode TNX5001 and Paragrid 50S. Their physical properties provided by the manufacturers are summarized in Table 3 (Liu 1992). Before the geogrids were used in the test fill, load-extention prorerties of reinforcement materials were obtained from unconfined tests such as the grab tensile test, the steip tensile test and wide width tensile test.

There were defects in the Paragrid material supplied and placed in the test fill. It was found during laboratory tests that some of the high strength fibers in the tension members were weakened or damaged at the intersections of the grids. This damage was most likely caused by overheating of the polypropylene sheath during the welding process. During test of Signode section it was found that most instrumentation damaged and not good for interpretation of the instrumentations results, therefore it was chosen to use Tensar section for the FEM analysis by Plaxis (Liu 1992).

Tabel 3 Physical properties of geogrid

Type of material		Tensar	Signode	Paragrid
Type of polymer		Polyethylene	Polyester	Polyester
Structure		Uniaxial greed	Rectangular Greed	Quadrangle
Junction Type		Planar	Welded	Welded
Weight, (g/m)		930	544	530
Open Area, (%)		55	58	78
Aperture size (mm)	MD	99.1	89.7	66.2
	CMD	15.2	26.2	66.2
Thickness (mm)		T 1.27 A 4.57	T 0.75 J 1.50	T 2.05 J 3.75
Color		Black	Black	Yellow
Tensile Force (2% strain), kN/m		19 - 20	32 – 34	---

MD – Mashine direction, CMD – Cross machine direction
T – Tension member, A – Anchor mamber
J - Joint

2D SIMULATION OF REINFORCED EMBANKMENT BY PLAXIS

Proper constructiv models of the test fill and material parameters should be used in the finite element analysis. Nonlinear stress-strain relationship were used for modeling the behavior of the soil, and the model parameters were derived based on labaratory test results. The load-strain behavoiur of geosynthetic under test conditions of 20^0C and rate 2% per minute were used to model the reinforcement (Chang-Tok Yi 1995).

For analysis of reinforced embankment by Plaxis we need to know only one perameter of geogrid is an elastic normal (axial) stiffness of grids, which can be defined:

$$EA = E \cdot t, kN/m \tag{1}$$

where *EA* is elastic normal (axial) stiffeness of grids, *kN/m*; *E* is Young`s modulus of the geotextile, kN/m^2; *t is* thickness of the fabric, *m*.

Young`s modulus of the geotextile is obtained from having tensile force:

$$E = T \cdot t_s, kN/m^2 \tag{2}$$

where T is tensile force, *kN/m*; t_s is trasformed thickness of geogrid, *mm*.

Transformed thickness of the gegrid defined by next equation:

$$t_s = t_g \frac{w_g}{s}, mm \tag{3}$$

where t_g is ribs thickness of geogrid, *mm*; w_g is ribs width of geogrid, *mm*; s is space between rigs of geogrid (Figure 3), *mm*.

Properties of the foundation soil, fill soil and reinforcements accepted for FEM analisys by Plaxis are shown on Table 4.

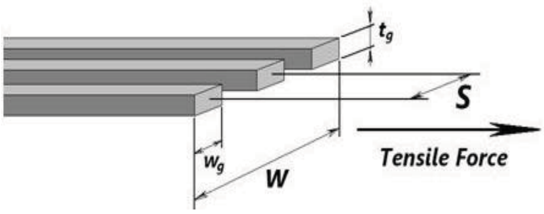

Fig. 3 Geometrical parameters of geogrids

Tabel 4 Parameters of materials

Parameters	Foundation Soil							
	Sandy Clayey Silt	Silty Clay	Clay Till	Sand	Clay	Sand & Gravel	Clay Shale	Fill Soil
Material Model	M-C							
Behaviour	Drain							
Dry Soil Weight,	18	18	17	15	19	18	19	17
Wet Soil Weight,	20	20	19	20	21	22	21	20
Hor. perm.,	1E-7	1E-7	1E-7	1E-4	1E-8	1E-2	1E-9	1E-7
Vert. perm.,	1E-7	1E-7	1E-7	1E-4	1E-8	1E-2	1E-9	1E-7
Elastic Modulus,	35000	24000	90000	1,2 E+5	26000	24 E+4	42000	28000
Possion`s Ratio, -	0.4	0,4	0.42	0.38	0.4	0.35	0.4	0.35
Cohesion,	23	25	30	1	35	0,1	48	20
Friction , ^{0}	24	24	25	35	25	40	21	28
Dilatancy, ^{0}	24	24	25	35	25	40	21	28

Full assignment consist of 33 steps, firs of all we need to know initial condition of embankment, next steps are compactions of intermediate soils stratum and reinforcement layers instalations, that is lead to increase pore pressure of ground water, and decrease of the φ and c parameters.

It was chosen several interested us points for the analysis of embankment settlement. Taken points are corresponds to positions of instrumentations.

According results full settlement due to of full consolidation will be in 2602 year, but for point D (12 m) the settlement will be neglected small at 2055, with rate of 0.5 mm per year. For point A (0 m) same rate of settlement occur after 2001, for point B (2m) – 2012, point C (4 m) – 2025, point F (-6 m) after two years. Predictable settlement curve is shown on Fig. 4.

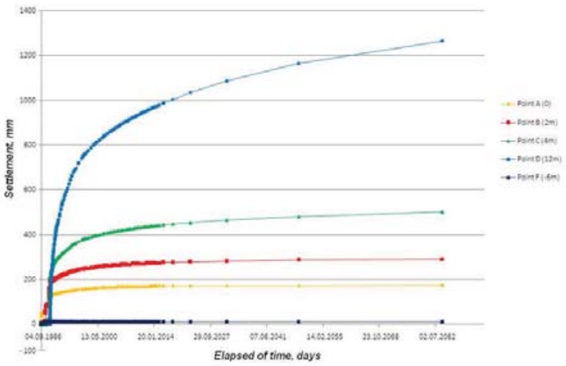

Fig. 4 Predictable settlement curve

The settlement of unreinforced section almost has the same value as reinforced section. As in-situ test results shown the magnitude of point D settlement (12 m high) is 820 mm for unreinforced section whereas reinforced section settlement is 750 mm. For the curiously Plaxis results has the same picture. Unreinforced and reinforced section settlements obtained by in-situ observation and Plaxis simulations are shown on Figure 5 and 6.

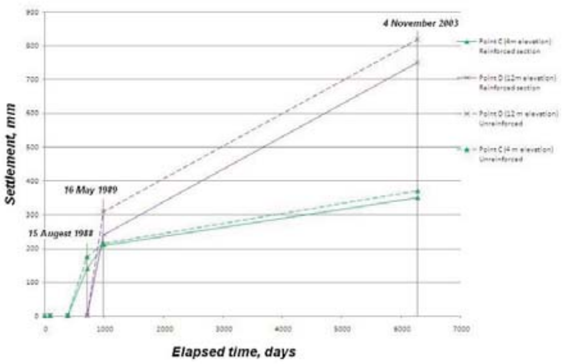

Fig. 5 Unreinforced and reinforced section settlements difference by in-situ

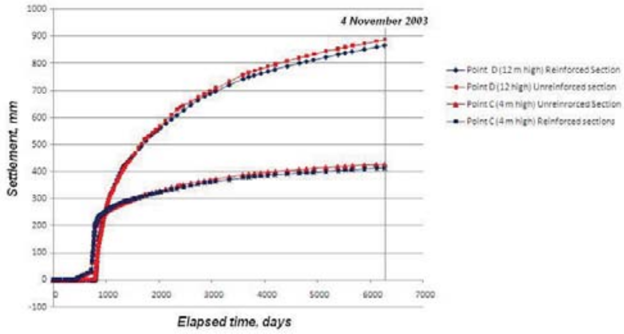

Fig. 6 Unreinforced and reinforced section settlements difference by Plaxis

The settlement of reinforced section slightly higher than unreinforced section settlement. This may also raise the doubt that there are no effects of reinforced application, but real observation can help us to understand the mechanism of stability and failure redistribution load among construction parts and transmission the stress from the overloading zone to the adjacent underloading zone, lead to smoothly deformation of reinforced section and failure effects of unreinforced section.

CONCLUSIONS

By obtained results of settlements we can conclude that reinforcement of earth embankment play a very big role for the embankment stability for a long time. As results of Plaxis simulation of soft clay embankment showed that settlement of the high point (12 m) conventionally stops at 2055 and predictable magnitude of full settlement is 1150 mm. Following settlement is neglected small with rate 0.5 mm per year. The settlements value various from 914 to 920 mm at 2008 and still continue with rate 9 – 10 mm per year.

Insignificant settlement difference of reinforced and unreinforced section can mislead us of application necessity of reinforcement. From the aforesaid we can make wrong conclusion that there is no effect of reinforcement, but real embankment observation corroborated reinforced model as long as unreinforced section had a failure effects of the shallow slope, whereas reinforced section smoothly deformed without any rapture, collapse or crumble effects.

In fine fulfilled researching work approved reinforcement model as one of the effective earth improvement technology concrete.

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