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COMPARATIVE ANALYSIS OF DESIGN SOLUTIONS OF A REINFORCED RAILROAD EMBANKMENT USING VARIOUS CALCULATION METHODS

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ABSTRACT: This paper presented comparative analysis of design solutions of a reinforced railroad embankment using various calculation methods. To conduct a detailed comparative analysis, a remote part of the projected railway line in South Kazakhstan was selected. Embankment has five sections, four sections reinforced by geogrid, one of them without reinforced layer. The studies of geotechnical conditions concentrate on the top most silt or silty clay due to of upper soil is underlain by a stiff to very stiff clay till which is heavily over consolidated. As the main load case, railway loads were considered in accordance with the current regulatory documents SP RK EN 1991. For preliminary design of unreinforced embankment as many available calculation methods as possible were considered in order to obtain the most unfavorable values of safety factors. A detailed analysis using finite element method was performed to check the results of standard calculation methods to ensure safety of the embankment.

Keywords: Embankment, Geogrid, FEM, Soil Stability.

1. INTRODUCTION

Currently, the Republic of Kazakhstan is actively developing its railway transport system, which includes state programs for regional development. As part of these programs, the existing railway infrastructure is being overhauled, as well as the construction of new modern lines in promising directions. Improvement of transport infrastructure allows acceleration of economic growth and improves the quality of life and well-being of residents of the regions. It should also be noted that these development programs create new jobs and increase the interest of large companies in remote and hard-to-reach regions.

Considering the above factors, the Government of the Republic of Kazakhstan decided on the further development of the transport system of this region, including the construction of roads and railways with the modernization of the adjacent infrastructure. The combination of the above factors necessitates a detailed analysis of geotechnical conditions and the selection of optimal design solutions.

This scientific study is based on a comparative analysis of various geotechnical solutions within the framework of a working design for railway embankments on complex sections of a new railway line located in South Kazakhstan. Previously scientific analysis of different methods was performed in several studies [1-3]. Results in these studies showed correlation between the most popular analysis methods (Meyerhof's, Terzaghi's and SNiP methods). Information given in mentioned studies was used to peform a detailed analysis in the current research with addition of several methods and laboratory tests. Also it is necessary to apply results of the previous research to a design of a railway embankment due to specific parameters of loads and geometry of the embankment.

Based on the facts described above, it is necessary to carry out a detailed study, considering all possible calculation methods to determine the most unfavourable results of the bearing capacity, as well as to conduct laboratory tests of the soil embankment model for a comparative analysis of the results.

2. RESEARCH SIGNIFICANCE

Stability analysis is a significant part of the design process. The design must satisfy the results of the stability analysis therefore more accurate results should lead to a more stable structure and avoid many possible problems in the future. Different methods of embankment slope stability analysis, mechanisms of instability of slopes and their remedial measures were considered in this study**.** Mentioned models and methods might be very important for Kazakhstan and will have practical application in railroad construction taking into account modern changes in standards and usage of new materials.

3. OBJECT CHARACTERISTICS

3.1 Design Solutions

To conduct a detailed comparative analysis, a remote part of the projected railway line in South Kazakhstan with the highest designed embankment in this project was selected . This embankment has a height of 13.7 m and a slope of 1:2 (V:H).

The accepted dimensions of the embankment are based on the current standards of the Republic of Kazakhstan for the design of railway embankments and are due to the variable geological conditions along the length of the railway section. Also, new requirements in connection with the transition to European standards (such as SP RK EN 1991) were taken into account [1].

Various options were considered as the material of the embankment, depending on the feasibility study of their delivery to the construction site, as well as their physical and mechanical parameters.

The Embankment has five sections, four sections reinforced by geogrid, one of them without reinforced layer.

3.2 Geogrid Properties

After completing a detailed feasibility study for the use of geotextiles, the study selected an available material with characteristics, which are presented in Table 1.

Table 1 Properties of the geogrid

Properties	Value
Type of polymer	Polyethylene
Structure	Uniaxial grid
Junction Type	Planar
Weight, (g/m^2)	940
Open Area, (%)	58
Main aperture size (mm)	99.2
Thickness (mm)	1.31
Color	Black
Tensile Force (2% strain), kN/m	20 - 21

3.3 Geotechnical Conditions

The studies of geotechnical conditions concentrate on the top most silt or silty clay due to of upper soil being underlain' by a stiff to very stiff clay till which is heavily overconsolidated. In the depth of between 9 and 10 m from the ground surface the clay till is then underlain by a very dense sand. To determine the physic-mechanical properties of the upper foundation soil detailed laboratory tests were performed [2]. Table 2 shows properties of the upper foundation soil.

Table 2 Properties of the Upper Foundation soil

Properties	Index	Value	Unit
Atterberg Limits Tests			
Water content	W_n	35.2	%
Liquid Limit	L_l	45.8	%
Plastic Limit	L_p	22.3	%
Plasticity Index	I_P	23.4	%
Dry Unit Weight	γ_{dry}	18	kN/m^3
Saturated Unit Weight	γ_{sat}	20	kN/m^3
Grain Size Distribution Tests			
Sand	-	6	%
Clay	-	21	%
Silt	-	74	%
Consolidation Tests			
Coefficient of Consolidation	C_v	0.001	-
Compression Index	C_c	0.538	-
Recompression Index	C_r	0.055	-
Time factor	t_{90}	2.51	-
Coefficient of volume Compressibility	M_v	1.4E-4	m/kN
Coefficient of Permeability	k	1.03E-7	cm/s
Triaxial and Direct Shear Tests			
Cell Pressure	σ_1	518	kPa
Density	ρ_d	1.859	g/cm
Void Ratio	e	0.59	-
Saturation	S_r	85.0	%
($\sigma1 - \sigma3$)	-	435	kPa
Strain	ε	7.8	%
Friction Angle	φ	22	°
Cohesion	c	25	kN/m^2
Elastic Modulus	E	36000	kN/m^2
Poison`s Ratio	ν	0.4	-

3.4 Load Assumptions

As the main load case, railway loads were considered in accordance with the current regulatory documents SP RK EN 1991 (See Fig. 1) [4].

Also, to carry out control calculations and determine the safety factors of bearing capacity, calculations were performed according to the standard regulatory framework of previous years. The need to perform calculations for both standards is due to the transition period in the Republic of Kazakhstan, as well as the fact that the dominant existing railway infrastructure was designed on the basis of SNiP.

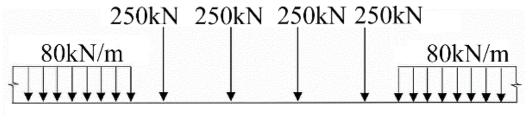

Fig.1 Load model LM 71 (Eurocode 1)

4. METHODS OF STABILITY ANALYSIS

Stability analysis is one of the significant design parts. By results of stability analysis, the future of designed structure is depends, therefore more accurate results lead to more accurate prediction future state of structure and avoiding the possible problems [3].

Different methods of embankment slopes stability analysis causes of instability of slopes and their remedial measures were considered in this paper work. Two main critical states were taken into account, first one is bearing capacity, and second is critical slip surface. The safety factor is based on type of soil (cohesive or cohesiveless), reliability of the soil parameters, structural information, and consultant caution [5].

Calculation methods are shown in Table 3.

Table 3 Calculation methods

The bearing capacity analysis
Terzaghi Method
Meyerhof Method
Hansen Method
Vesic Method
SNIP Method
The critical slip circle analysis
Method of slices
Bishop`s method
Fellenius construction for critical circle
Taylor`s slope stability number method
Morgenstern-Price slope stability method

4.1 Terzaghi`s Method of Bearing Capacity

One of the early sets of bearing-capacity equation was proposed by Terzaghi (1943) – Eq. (1). Terzaghi used shape factors. Terzaghi`s equations were produced from a slightly modified bearing-capacity theory developed by Prandtl (1920) from using the theory of plasticity to analyze the punching of a rigid base into a softer (soil) material [6].

$$q_{ult} = cN_c s_c + \overline{q}N_q + 0.5\Upsilon B N_\gamma s_\gamma \quad (1)$$

4.2 Meyerhof`s Method of Bearing Capacity

Meyerhof suggested using his equation similar to that of Terzaghi but included a shape factor sq with the depth term Nc – Eq. (2). He also included depth factor di and inclination factor ii for case where the footing load is inclined from the vertical [7].

$$q_{ult} = cN_c s_c d_c + \overline{q}N_q s_q d_q + 0.5\Upsilon B`N_\gamma s_\gamma d_y \quad (2)$$

4.3 Hansen`s Method of Bearing-Capacity

Hansen proposed the general bearing-capacity case and N factors equations in 1970 – Eq. (3). This equation is readily seen to be a further extension of the earlier Meyerhof work. Hansen`s shape factors include situation in which the footing is tilted from the horizontal bi and for the possibility of a slope β of the ground supporting the footing to give ground factors g_i [8].

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + \overline{q}N_q s_q d_q i_q g_q b_q + \\ + 0.5\Upsilon B`N_\gamma s_\gamma d_y i_y g_y b_y \quad (3)$$

4.4 Vesic`s Method of Bearing Capacity

The Vesic (1973) procedure is essentially the same as the method of Hansen with select changes. The Nc and Nq tems are those of Hansen but Ny is slightly different – Eq. (4).

$$N_y = 2(N_q + 1)\tan\varphi \quad (4)$$

4.5 SNIP Method of Bearing Capacity

According to SNIP bearing capacity of soil is determined by the equation of soil resistance R ($q_{allowable}$), kPa – Eq. (5).

$$R = \frac{\gamma_{c1}\gamma_{c2}}{k}\begin{bmatrix} M_\gamma k_z b\gamma_{II} + M_q d_1 \gamma'_{II} + \\ (M_q - 1)d_b \gamma'_{II} + M_c c_{II} \end{bmatrix} \quad (5)$$

4.6 Methods of Slice

In all the limit-equilibrium methods of stability analysis the factor of safety is determined by considering equilibrium of the potential sliding mass along assumed slip surfaces and locating by trial the slip surface that gives the lowest factor of safety [9].

The method of analysis should be such as to accommodate conditions wherein the slip surface is curved and the soil properties and pore pressures vary with location through the slope [10].

5. FEM ANALYSIS

Proper models of the test fill and material parameters were used in the finite element analysis. Nonlinear stress-strain relationship was used for modeling the behavior of the soil, and the model parameters were derived based on laboratory test results [11].

Some useful examples of laboratory tests are performed in articles related to soil embankments [12-14].

An elastic normal (axial) stiffness of grids is one of the significant parameters for analysis of reinforced embankment by Plaxis – Fig. 2-4. It can be found by Eq. (6).

$$EA = T \cdot t, kN/m \quad (6)$$

Young`s modulus of the geotextile (E) we obtained from having tensile force (T) by Eq. (7).

$$E = T \cdot t_s, kN/m^2 \quad (7)$$

Transformed thickness of the geogrid defined by Eq. (8).

$$t_s = t_g \frac{w_g}{s}, mm \quad (8)$$

Table 4 Material parameters for Plaxis

Parameters	Foundation	Fill
Material Model	M-C	M-C
Dry Soil Weight, kN/m3	18	17
Wet Soil Weight, kN/m^3	20	20
Elastic Modulus, kPa	35000	28000
Possion`s Ratio, -	0.4	0.35
Cohesion, kPa	23	20
Friction angle	24	28

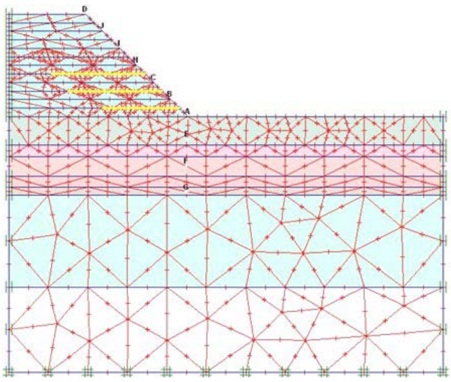

Fig.2 FEM model in Plaxis (General view)

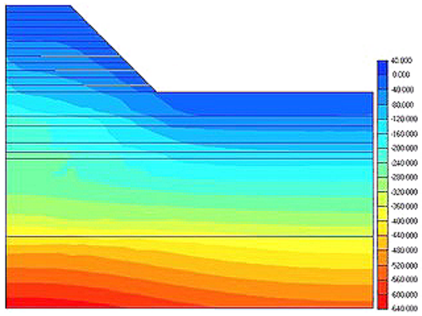

Fig.3 FEM model without reinforcement

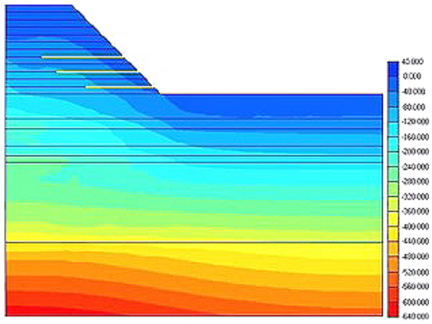

Fig.4 FEM model with reinforcement

6. RESULTS OF FEM AND ANALYTIC METHODS

The calculations were performed for various situations, including a reinforced embankment with several options for the location of the geogrid, as well as an unreinforced embankment. The list of performed design situations is presented in Table 5. The calculation results are presented in Tables 6-9 and Fig. 5-8.

Table 5 Design situations

Type	Type of geogrid reinforcement
A	Unreinforced embankment
B	3 layers with vertical spacing 2.6 m
C	4 layers with vertical spacing 2 m
D	5 layers with vertical spacing 1.8 m
E	6 layers with vertical spacing 1.5 m

Table 6 Results of calculations for unreinforced embankment (A) by standard methods

	Method of Calculation	Factor Nq	Factor Nc	Factor Ny	FOS
1	Terzaghi	11.7	24.07	8.58	11.6
2	Meyerhof	5.79	10.76	3.18	5.5
3	Hansen	5.79	10.76	3.2	5.6
4	Vesic	5.79	10.76	6.04	9.3
5	SNIP	3.87	6.45	0.72	1.9

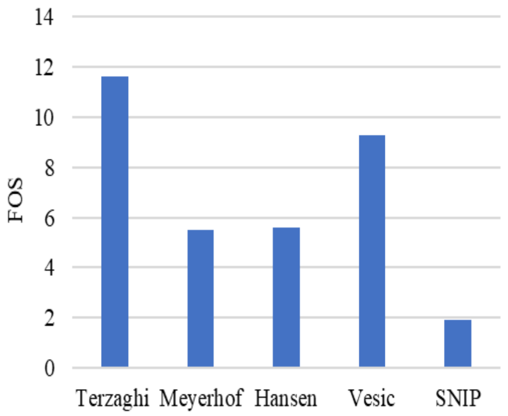

Fig.5 Results of calculations of bearing capacity for unreinforced embankment (A) by standard methods

Table 7 Results of calculations of critical slip for unreinforced embankment (A) by standard methods

Critical Circle		Factor of Safety
Slice Method	S1	0.894
	S2	0.891
	S3	0.862
Bishop`s Method	B1	1.075
	B1	1.081
	B1	1.168
Fellenius Method	F1	1.543
	F2	1.273
	F3	1.121
Taylor's Method	T1	1.253
	T2	1.204
	T3	1.110
Morgenstern-Price Method	M1	1.004
	M2	1.015
	M3	1.080

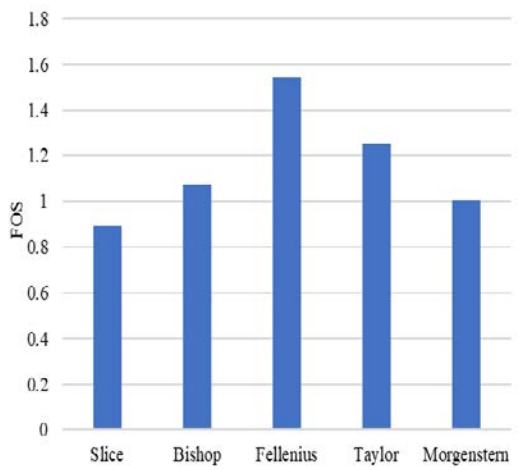

Fig.6 Results of calculations of critical slip for unreinforced embankment (A) by standard method

Table 8 Results of calculations of bearing capacity by FEM

Type	Factor of Safety
A	3.423
B	6.258
C	8.605
D	10.345
E	11.544

Table 9 Results of calculations of critical slip by FEM

Type	Factor of Safety
A	0.642
B	0.895
C	1.105
D	1.242
E	1.453

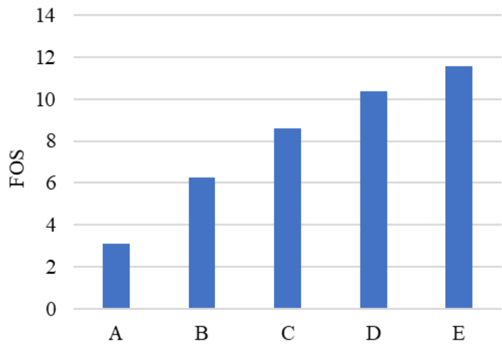

Fig.7 Results of calculations of bearing capacity by FEM

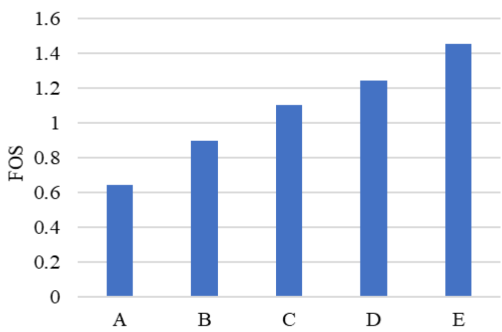

Fig.8 Results of calculations of critical slip by FEM

7. CONCLUSION

This paper presented comparative analysis of design solutions of a reinforced railroad embankment using various calculation methods.

To conduct a detailed comparative analysis, a remote part of the projected railway line in South

Kazakhstan was selected.

Embankment has five sections, four sections reinforced by geogrid, one of them without reinforced layer. As the main load case, railway loads were considered in accordance with the current regulatory documents SP RK EN 1991.

The following main conclusions may be drawn from this study:

1. For preliminary design of unreinforced embankment it is necessary to consider as many available calculation methods as possible in order to obtain the most unfavourable values of safety factors.
2. It is necessary to perform detailed analysis using finite element method to check the results of standard calculation methods in order to ensure safety of the enbankment.
3. The most unfavourable results for this particular case were obtained using FEM.
4. Bearing capacity analyses of test embankment reveal lack-of correspondence of the various analysis methods. The principal reason of that is great difference of bearing capacity factors. Unfortunately, this aspect exceeds the scopes of research work, but might be good prerequisite for the scientists.
5. Proper constitutive models of the analysed structure and parameters of each elements of model seems to be very important part of preparation to numerical analysis, becomes fundamental assignment to obtain good results.

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