Q290-CL0-GEO-REP-EAR-REV1

ROADS DEPARTMENT OF MINISTRY OF REGIONAL DEVELOPMENT AND INFRASTRUCTURE OF GEORGIA

Preparation of Detailed Design for the Upgrading of Tbilisi-Sagarejo and Sagarejo – Bakurtsikhe Road Sections

Geotechnical Design Earthworks

CONSTRUCTIONAL LOT 0

ACTIVITY 2

31/05/2021

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REVISION

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1 INTODUCTION

ILF Consulting Engineers Georgia LLC (The Concessionaire) has been assigned by the the Roads Department of the Ministry of Regional Development and Infrastructure of Georgia - RDMRDI to carry out the the Detailed Design of the project “Upgrading of Tbilisi – Sagarejo (Section 1) and Sagarejo - Bakurtsikhe (Section 2) Road ((EWHIP-4/CS/QCBS-06)”.
ILF Consulting Engineers has also to perform consulting services for preparation of Detail Engineering Design of Earthworks for the main alignment, interchanges and secondary roads that is a part of Consulting Services for the Detailed Design for the Upgrading of Tbilisi – Sagarejo (Section 1) and Sagarejo - Bakurtsikhe (Section 2) Road. The Consultant will assist the Client in the following assignments: Consulting services for preparation of Detailed Design for the Upgrading of Tbilisi-Sagarejo and Sagarejo-Bakurtsikhe Road – Detail Engineering Design of Earthworks for main alignment, interchanges and secondary roads.
Activities for the Section 1 and Section 2 are split in 6 consecutive Lots. The Chainage for the six consecutive constructional lots will be:

- 00+310 – 04+040 (Lochini Interchange to Vaziani Interchange.)
- 04+040 – 27+840 (Vaziani Interchange to Ninotsminda Interchange.
- 27+840 – 35+500 (Ninotsminda Interchange to Tokhliauri Interchange).
- 35+500 – 53+000 (Tokhliauri Interchange to Badiauri Interchange).
- 53+000 – 75+000 (Badiauri Interchange to Chalaubani Interchange).
- 75+000 – 84+000 (Chalaubani Interchange to Bakurtsikhe).

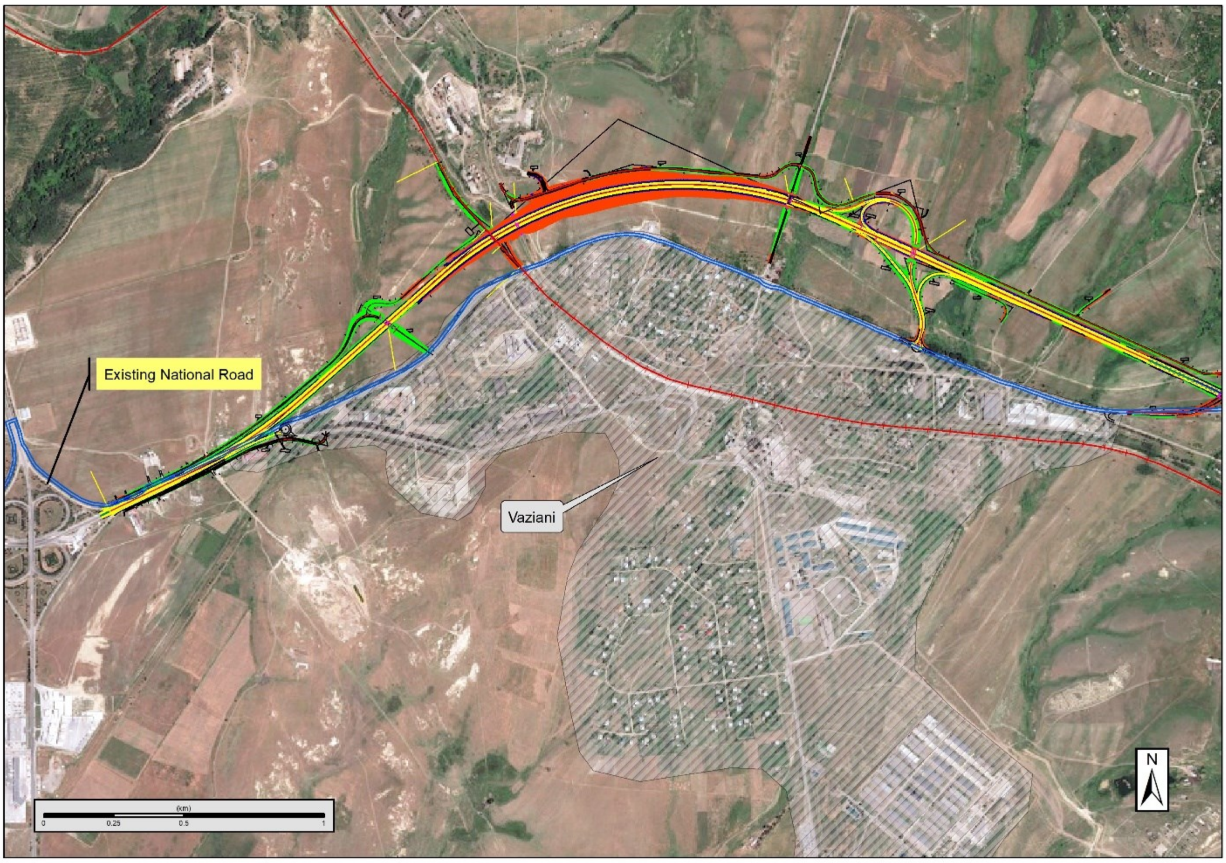


Figure 1. Overview Map of the Constructional Lot 0 from Lotchini I/C to Vaziani I/C, Including the IC.

This report presents the geotechnical design of the embankments and cut slopes along the Motorway between Ch. 0+310 and Ch. 04+040 (Section 1, Lot 0). In this report a review of the geological and geotechnical conditions along the section of the highway under design is presented. This review takes into account the ground investigations that have been carried out recently. For a full review of the findings of the ground investigation program that was carried out along the study area, as well as for the evaluation of the ground conditions, the reader could be further informed from the series of the relevant “Factual Geotechnical Reports”.

2 DESIGN ASUMPTIONS AND METHODS

2.1 APPLIED STANDARDS

Projects' design standards (DIS) were considered for the design of embankments along the motorway in addition to with EN 1997 Eurocode 7: Geotechnical Projects. Part 1: General Rules and Eurocode 8: Project guidelines for structural resistance to seismic events.

2.2 FACTOR OF SAFETY

Geotechnical design is carried out according to Eurocode 7, where control of the geotechnical (GEO) limit equilibrium global stability conditions for earthworks, with or without structural stability measures, will be done according to Design Approach 3 (DA-3) for static conditions. The stabilization actions of the structural stability elements are considered as favorable actions with individual actions factor $\gamma_F = 1$. Design Approach 3 (DA-3) is implemented in conjunction with the relationship (2.6a) for actions:

$Ed = E (F_d , X_d) = E (\gamma_F F_k , X_k / \gamma_M)$ (2.6a)

And the realationship (2.7a) for the resistances:

$Rd = R (F_d , X_d) = R (\gamma_F F_k , X_k / \gamma_M)$ (2.7a)

And so with the application of relationship (2.5):

$Ed \leq Rd \Rightarrow E(\gamma_F F_k, X_k/\gamma_M) \leq R(\gamma_F F_k, X_k/\gamma_M$ (2.5)

And the following groups of individual factors of actions and ground parameters (γF,γM) of Appendix A EN1997-1:

- (A1) for structural actions such as building loads and traffic loads in ground surface,
- (A2) for ground actions (geotechnical) including the weight of the ground,
- (M2) for ground parameters.

Design Approach 3 (DA-3) refers only to the check of global stability of geotechnical works. The design of all stabilization measures is calculated with Design Approach 2 (DA-2). The Factor of Safety depends on the assumptions for the hydraulic conditions and has the following values:

(1) For regular unfavorable conditions: $\gamma_m = 1.1$.

And so, the total Factor of Safety is:

- $FS = \gamma_M \gamma_m = 1.25 \times 1.1 = 1.38$ for analysis with effective stresses and use of effective shear strength parameters (c', φ΄).
- $FS = \gamma_M \gamma_m = 1.40 \times 1.1 = 1.54$ for analysis with total stresses and use of undrained strength parameters (c_u).

(2) For very unfavorable assumptions for the hydraulic conditions: $\gamma_m = 1$.

In this case the total Factor of safety (FS) for global stability is:

- $FS = \gamma_M \gamma_m = 1.25 \times 1 = 1.25$ for analysis with effective stresses and use of effective shear strength parameters (c', φ΄).
- $FS = \gamma_M \gamma_m = 1.40 \times 1 = 1.40$ for analysis with total stresses and use of undrained strength parameters

Analyses under seismic actions of geotechnical works that are studied based on Eurocode EN1997-1 is carried out according to Eurocode 8 - Part 5 (EN 1998-5), with the following remarks:

(1) The individual factors of seismic actions and their effect will be considered equal to 1.0 ($\gamma_F = \gamma_E = 1$).

(2) The individual factors of ground parameters (γ_M) and resistances (γ_R) will be considered equal to 1.0 i.e. $\gamma_M = \gamma_R = 1.0$.

(3) Design Approach 2 is applied in all analyses cases (DA-2*), including those which in static conditions are analysed according to Design Approach 3 (DA-3). This is done in order to simplify calculations, as in the analyses under seismic actions the individual factors are considered 1.0 and so Design Approach 2 is equal to Design Approach 3.

Based on the above, analyses under seismic actions according to Eurocode EN1997-1 and EN1998-5 can be done using individual factors values equal to 1.0, i.e.: $\gamma_M = \gamma_R = 1$ in order to achieve a total Factor of safety equal to 1,0 (FS=1). Exception to that is the analysis of total stability (e.g. Cut Slope Stability) under normal hydraulic conditions where FS=1.10.

According to DIS, the embankments are divided into two categories: those that are founded on soil and those that are founded on rock. The acceptable factors of safety are slightly different, depending on the foundation type. According to DIS, both generalized slope failure, and slope failure between the berms (if any) have to be checked. The acceptable factors of safety are slightly different only for rock cut slopes.

The minimum required factors of safety for the load combinations that have to be considered are shown in Table 1 to Table 4 and fulfill the applied Standards of the project. The general safety envelope that has been considered in this report, covering all available specifications, is shown in Table 5 In case that a minimum factor of safety is not fulfilled, additional measures will be required i.e. geosynthetic reinforcements, ground improvement etc., in order to increase stability to the required minimum level of safety for each load combination.

Load Combination		Specification	Required Factor of Safety
SG1	Short Term Conditions	Eurocode 7	**1.40**
SG2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7/8	**1.10**
SG3	Long Term Conditions +Water level (Y_{50})	Eurocode 7	**1.25**
SG4	Long term Conditions+Completely Dry conditions	Eurocode 7/8	**1.38**

Table 1: Minimum Factors of Safety for Embankments Founded on Soil Formations- Generalized Slope Failure

Where:

Short Term Conditions: Use of undrained shear strength of the foundation soil (if applicable)

Long Term Conditions: Use of drained shear strength of the foundation Soil Earthquake: Design earthquake according to Georgian seismic Resistant Code

Water Level: **Y50** Estimated Maximum Water Level for a Period of 50 Years.
A Estimated Maximum Annual Water Level.
- No Pore Pressures

*Considering the “average gradient” of the entire slope

**Safety factor obtained by using “minimum” / “average” parameters for shearing resistance.

Load Combination		Specification	Required Factor of Safety
SB1	Short Term Conditions	Eurocode 7	**1.40**
SB2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7	**1.10**
SB3	Long Term Conditions +Water level (Y_{50})	Eurocode 7	**1.25**
SB4	Long term Conditions+Completely Dry conditions	Eurocode 7	**1.38**

Table 2: Minimum Factors of Safety for Embankments Founded on Soil Formations- Slope Failure between Berms

Where:
Short Term Conditions: Use of undrained shear strength of the foundation soil (if applicable)
Long Term Conditions: Use of drained shear strength of the foundation Soil
Earthquake: Design earthquake according to Georgian seismic Resistant Code
Water Level: **Y50** Estimated Maximum Water Level for a Period of 50 Years.
A Estimated Maximum Annual Water Level.
- No Pore Pressures

* Safety factor obtained by using "minimum"/"average" parameters for shearing resistance.

Load Combination		Specification	Required Factor of Safety
RG1	Short Term Conditions	Eurocode 7	**1.40**
RG2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7	**1.10**
RG3	Long Term Conditions +Water level (Y_{50})	Eurocode 7	**1.25**
RG4	Long term Conditions+Earhquake+Completely Dry conditions	Eurocode 7	**1.38**

Table 3: Minimum Factors of Safety for Embankments Founded on Rock Formations- Generalized Slope Failure

Where:
Short Term Conditions: Use of undrained shear strength of the foundation soil (if applicable)
Long Term Conditions: Use of drained shear strength of the foundation Soil
Earthquake: Design earthquake according to Georgian seismic Resistant Code
Water Level: **Y50** Estimated Maximum Water Level for a Period of 50 Years.
A Estimated Maximum Annual Water Level.
- No Pore Pressures

*Considering the "average gradient" of the entire slope
**Safety factor obtained by using "minimum" / "average" parameters for shearing resistance.

Load Combination		Specification	Required Factor of Safety
RB1	Short Term Conditions	Eurocode 7	**1.40**
RB2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7	**1.10**
RB3	Long Term Conditions +Water level (Y_{50})	Eurocode 7	**1.25**
RB4	Long term Conditions+Earhquake+Completely Dry conditions	Eurocode 7	**1.38**

Table 4: Minimum Factors of Safety for Embankments Founded on Rock Formations- Slope Failure between Berms

Where:
Short Term Conditions: Use of undrained shear strength of the foundation soil (if applicable)
Long Term Conditions: Use of drained shear strength of the foundation Soil
Earthquake: Design earthquake according to Georgian seismic Resistant Code
Water Level: **Y50** Estimated Maximum Water Level for a Period of 50 Years.
A Estimated Maximum Annual Water Level.
- No Pore Pressures

* Safety factor obtained by using “minimum”/“average” parameters for shearing resistance.

Load Combination		Specification	Required Factor of Safety
FS1	Short Term Conditions	Eurocode 7	**1.40**
FS2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7/8	**1.10**
FS3	Long Term Conditions +Water level (Y_{50})	Eurocode 7	**1.25**
FS4	Long term Conditions +Completely Dry conditions	Eurocode 7/8	**1.38**

Table 5: Minimum Factors of Safety for Embankments

Where:
Short Term Conditions: Use of undrained shear strength of the foundation soil (if applicable)
Long Term Conditions: Use of drained shear strength of the foundation Soil
Earthquake: Design earthquake according to Georgian seismic Resistant Code
Water Level: **Y50** Estimated Maximum Water Level for a Period of 50 Years.
A Estimated Maximum Annual Water Level.
- No Pore Pressures

2.3 SETTLEMENT

Calculations of settlements include both immediate and delayed settlement. As per Eurocode 7, Part 1, Section 6.6:

> *The following three components of settlement should be considered for partially or fully saturated soils:*
> - *s0: immediate settlement; for fully-saturated soil due to shear deformation at constant volume, and for partially-saturated soil due to both shear deformation and volume reduction;*
> - *s1: settlement caused by consolidation;*
> - *s2: settlement caused by creep.*

Special consideration is given to soils such as organic soils and soft clays, in which settlement may be prolonged almost indefinitely due to creep. The depth of the compressible soil layer to be considered when calculating settlement depends on the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements. This depth is normally taken as the depth at which the effective vertical stress due to the foundation load is 20 % of the effective overburden stress. For many cases this depth is also roughly estimated as 1 to 2 times the foundation width, but may be reduced for lightly-loaded, wider foundation rafts. This approach is not valid for very soft soils.

Any possible additional settlement caused by self-weight compaction of the soil is assessed.

The following should be considered:

- the possible effects of self-weight, flooding and vibration on fill and collapsible soils;
- the effects of stress changes on crushable sands.

Either linear or non-linear models of the ground stiffness are adopted, as appropriate. To ensure the avoidance of a serviceability limit state, assessment of differential settlements and relative rotations take account of both the distribution of loads and the possible variability of the ground.

Differential settlement calculations that ignore the stiffness of the structure tend to be over-predictions. An analysis of ground-structure interaction may be used to justify reduced values of differential settlements. Allowance should be made for differential settlement caused by variability of the ground unless it is prevented by the stiffness of the structure.

2.4 DESIGN FOR SEISMIC ACTIONS

2.4.1 Seismicity of the Area

Based on the Georgian Seismic Resistant Design Code, the Peak Ground Acceleration for Section 1 (Lots 1 & 2) of the Lochini Interchange to Sartichala and Sartichala (Iori Railway Station) to Tokhliauri Interchange should be taken according to the values presented in **Error! Reference source not found.**.

Chainage From:	Chainage To:	PGA:
0+000	0+031	0.14
0+031	7+338	0.12
7+338	24+318	0.14

Table 6: Characteristic Values of PGA along Constructional Lot 1 (From Georgian Seismic codex 2014)

Ground Type	Description of stratigraphic profile	Vs_{30} (m/s)	NSPT (blows/30 cm)	Cu kPa
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800		
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with *v* s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v s > 800m/s.			
S_1	Deposits consisting, or containing a layer at least 10m thick, or soft clays/silts with a high plasticity index (PI > 40) and high water content	<100 (indicative)		10-20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A-E or *S* 1			

Table 7: Ground Types, according to Eurocode 8

Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 7 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

The site should be classified according to the value of the average shear wave velocity, Vs_{30}, if this is available. Otherwise the value of N_{SPT} should be used. For sites with ground conditions matching either one of the two special ground types S1 or S2, special studies for the definition of the seismic action are required. For these types, and particularly for S2, the possibility of soil failure under the seismic action shall be taken into account.

Note: Special attention should be paid if the deposit is of ground type S1. Such soils typically have very low values of Vs, low internal damping and an abnormally extended range of linear behavior *and can therefore produce anomalous seismic site amplification and soil-structure interaction effects (see EN 1998-5:2004, Section 6). In this case, a special study to define the seismic action should be carried out, in order to establish the dependence of the response spectrum on the thickness and vs value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.*

2.4.2 Seismic zones

For the purpose of EN 1998, national territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant. For most of the applications of EN 1998, the hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_gR eg. Table 6. Additional parameters required for specific types of structures are given in the relevant Parts of EN 1998.

Note 1: The reference peak ground acceleration on type A ground, a_gR, for use in a country or parts of the country, may be derived from zonation maps found in its National Annex.

The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period TNCR of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, PNCR) chosen by the National Authorities (see 2.1(1)P). An importance factor γI equal to 1.0 is assigned to this reference return period. For return periods other than the reference (see importance classes in 2.1(3) P and (4)), the design ground acceleration on type A ground ag is equal to a_gR times the importance factor γI (ag =γI x a_gR). (See Note to 2.1(4)). In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used.

Note 2: The selection of the categories of structures, ground types and seismic zones in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended to consider as low seismicity cases either those in which the design ground acceleration on type A ground, ag , is not greater than 0,08g (0,78 m/s^2), or those where the product ag x S is not greater than 0,1 g (0,98 m/s^2). The selection of whether the value of a_g,or that of the product ag x S will be used in a country to define the threshold for low seismicity cases, may be found in its National Annex.

In cases of very low seismicity, the provisions of EN 1998 need not be observed.

Note 3: The selection of the categories of structures, ground types and seismic zones in a country for which the EN 1998 provisions need not be observed (cases of very low seismicity) may be found in its National Annex. It is recommended to consider as very low seismicity cases either those in which the design ground acceleration on type A ground, ag, is not greater than 0,04g (0,39 m/s^2), or those where the product ag x S is not greater than 0,05g (0,49 m/s^2). The selection of whether the value of a_g or that of the product ag x S will be used in a country to define the threshold for very low seismicity cases, can be found in its National Annex.

2.4.3 Acceleration Spectrum

Within the scope of EN 1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum". The shape of the elastic response spectrum is taken as being the same for the two levels of seismic action introduced in 2.1(1) P and 2.2.1(1) P for the no-collapse requirement (ultimate limit state – design seismic action) and for the damage limitation requirement.

The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum. For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquake magnitudes generated from them.
For the horizontal components of the seismic action, the elastic response spectrum Se (T) is defined by the following expressions (see below):

$$0 \le T \le T_B: S_c(T)=a_g*S*[1+\frac{T}{TB}*(n*2{,}5-1)]$$

$$T_B \le T \le T_C: S_c(T)=a_g*S*n*2{,}5$$

$$T_C \le T \le T_D: S_c(T)=a_g*S*n*2{,}5*[\frac{TC}{T}]$$

$$T_D \le T \le 4_s: S_c(T)=a_g*S*n*2{,}5*[\frac{TC*TD}{T}]$$

Where:

Se (T)	is the elastic response spectrum
T	is the vibration period of a linear single-degree-of-freedom system
a_g	is the design ground acceleration on type A ground (ag =γIa_gR)
T_B	is the lower limit of the period of the constant spectral acceleration branch
T_C	is the upper limit of the period of the constant spectral acceleration branch
T_D	is the value defining the beginning of the constant displacement response range of the spectrum
S	is the soil factor
η	is the damping correction factor with a reference value of η= 1 for 5% viscous damping.

The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type (Table 8).

Ground type	S	T_B	T_C	T_D
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 8: Values of the parameters describing the recommended Type 1 elastic response spectra

2.4.4 Design based on Pseudostatic Analysis

The response of ground slopes to the design earthquake is calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods subject to the limitations of (3) and (8) of Eurocode 8. In modelling the mechanical behavior of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading is taken into account.

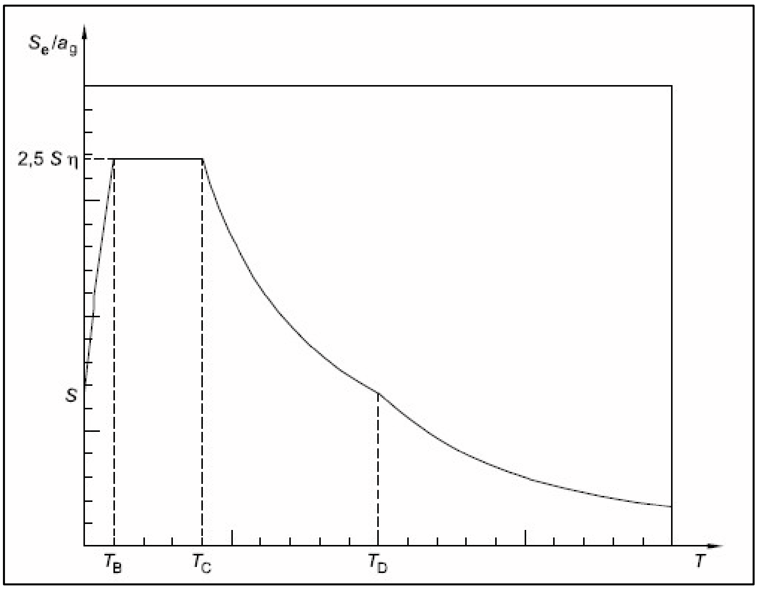


Figure 2: Shape of the Elastic Response Spectrum

The stability verification may be carried out by means of simplified pseudostatic methods where the surface topography and soil stratigraphy do not present very abrupt irregularities. The pseudo-static methods of stability analysis are similar to those indicated in EN 1997-1:2004; 11.5, except for the inclusion of horizontal and vertical inertia forces applied to every portion of the soil mass and to any gravity loads acting on top of the slope. The design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

$F_H = 0,5\ \alpha\ SW$

$F_V = \pm\ 0,5\ F_H$ if the ratio a_vg/ag is greater than 0,6

$F_V = \pm\ 0,33\ F_H$ if the ratio a_vg/ag is not greater than 0,6.

Where:

α is the ratio of the design ground acceleration, ag, to the gravity acceleration g;

a_vg is the design ground acceleration in the vertical direction;

a_g is the design ground acceleration;

S is the soil parameter of EN 1998-1:2004, 3.2.2.2; (**Error! Reference source not found.**)

W is the weight of the sliding mass.

A topographic amplification factor for a_g is taken into account according to 4.1.3.2 (2). A limit state condition shall then be checked for the least safe potential slip surface.

The horizontal pseudostatic acceleration coefficient can be given by the following equation considering the height of the embankment: $\alpha_h = \alpha_\pi = (\alpha_B + \alpha_K)/2$

The vertical pseudostatic acceleration coefficient is given by: $\alpha_v = 0.5\alpha_h$

Where:

α_B: the horizontal acceleration at the base of the embankment αK: the horizontal acceleration at the crest of the embankment

The acceleration at the base is given by: $\alpha_B = 0.5 * PGA$ the acceleration at the crest is given by: $\alpha_K = \alpha_B * \beta(T)$

Where:

β(T): the spectral magnification (Figure 3)

T: the fundamental period of the embankment, which may be estimated by: T = 2.5 H /Vs

Vs: the average shear wave velocity of the embankment material, assumed to be Vs = 300m/sec

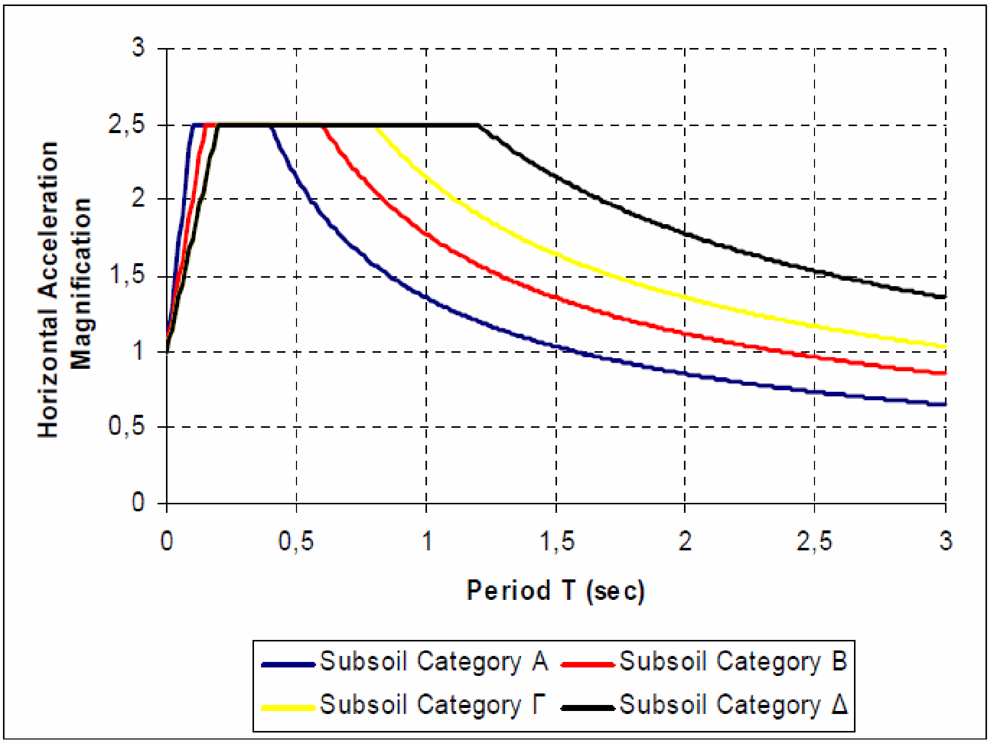


Figure 3: Horizontal Acceleration Magnification

The procedure outlined above is valid for embankments of trapezoidal shape, resting on the subsoil, which is the case of the embankments of the current report. An example of estimation of horizontal and vertical pseudostatic acceleration coefficients for ground type C, equivalent to the fill material used in the project, according to the height of the embankment (PGA=0.14g) is given in Table 9.

Height (m)	Period T (sec)	Soil Class C			
		αB (g)	αK (g)	αh(g)	av(g)
5	0.04	0.07	0.093	0.082	0.041
7.5	0.06	0.07	0.105	0.088	0.044
10	0.08	0.07	0.116	0.093	0.047
12.5	0.10	0.07	0.123	0.096	0.048
15	0.13	0.07	0.134	0.102	0.051
17.5	0.15	0.07	0.146	0.108	0.054
20	0.17	0.07	0.158	0.114	0.057

Table 9: Example of estimation of horizontal and vertical pseudostatic acceleration coefficients for ground type C according to the height of the embankment

The serviceability limit state condition may be checked by calculating the permanent displacement of the sliding mass by using a simplified dynamic model consisting of a rigid block sliding against a friction force on the slope. In this model the seismic action should be a time history representation in accordance with 2.2 and based on the design acceleration without reductions.
The pore pressure increment should be evaluated using appropriate tests. In the absence of such tests, and for the purpose of preliminary design, it may be estimated through empirical correlations.

2.4.5 Liquefaction Susceptibility

Liquefaction during and after an earthquake may affect an embankment in the following two ways: Firstly, liquefaction may occur within the embankment's body itself. This is taken care of, specifying appropriate material and compaction standards. Further examination is not needed.

2.4.6 Earthquake Volumetric Compression

During an earthquake event, volumetric compression may occur, if the embankment material is in a loose state. This is taken care of, specifying appropriate material and compaction standards. Further examination is not needed.

2.5 PROCEDURES FOR SLOPE STABILITY ANALYSES

Assuming that the geotechnical conditions are uniform along a given embankment, slope stability analyses are carried out considering all probable failure surfaces for the highest section. For this section a representative geotechnical profile is considered. Both static and seismic conditions are analyzed, considering appropriate water levels according to the specifications.

The effect of the highest water level (50year return period) and the annual water level, is considered by making appropriate assumptions regarding the position of the piezometric surface. This is done on an area by area basis, considering mainly the results from the geotechnical investigation.

A traffic load equal to 20 kPa is assumed in the analyses.

The stability of the embankment slopes, is assessed using the software Slide Ver. 5.0, which is available from RocScience Inc. The software has the ability to calculate the factor of safety for circular and non-circular failure surfaces, using various limit equilibrium methods (methods of BISHOP, JANBU etc.), always in the context of the method of slices.

2.6 ALLOWABLE SETTLEMENTS

In a general note, the acceptable long-term post construction settlement for the pavement is set to 15cm in cases of approach embankments the allowable settlements are set to 5cm in a distance of 50m from the structure. This refers only to the consolidation settlements estimated in the analysis. The immediate settlements that will occur in coarse grained materials or in the unsaturated zone (until saturation) maybe significant but will be concluded during or short after the end of embankments' construction, and so they can be treated from construction procedure (e.g. construction of a higher embankment than design equivalent to the estimated immediate settlements), with no additional measures of treatment.

Settle3D v2.0 by RocScience Inc was used for the consolidation analyses and the calculation of total settlements. *Settle3D* is a 3-dimensional program for the analysis of vertical settlement and consolidation under surface loads such as foundations, embankments and surface excavations.

There are several important assumptions and limitations that must be considered when using *Settle3D*:

- *Settle3D* calculates three-dimensional stresses due to surface loads. However, displacements (settlement) and pore pressures are computed in one-dimension, assuming only vertical displacements can occur. This is in keeping with general geotechnical engineering practice and material parameters are specified to reflect the one-dimensional nature of the analysis.
- Loads may be either flexible or rigid. For a uniform flexible load, the stress at the surface directly below the load is constant, but the displacement is not. For a uniform rigid load, the displacement directly below the load is constant, but the stress is not.
- The ground surface is at depth = 0 (by default), depth is positive downwards, and compressive stress is positive.
- *Settle3D* does not consider the interaction between rigid loads or between rigid and flexible loads. Only interaction between flexible and flexible loads is considered. Excavations and embankments are considered to be flexible loads.

Consolidation settlement solves Terzaghi's 1-D differential equation using the finite difference approach. The consolidation solution utilizes the vertical stress increment due to the embankment 'immediate' loading as the initial condition (i.e., the initial excess pore water pressure is equal to the induced stress increment at the point of interest).

It is noted that in order to compute total consolidation settlement and the rate of consolidation it is necessary to define the following parameters for the consolidating soil layers (fine grained alluvial deposits):

(a) Overconsolidation ratio OCR

(b) Initial Void ratio e_o

(c) Compression index C_c

(d) Recompression index $C_r = C_c/5$

(e) Coefficient of consolidation in the vertical direction C_v (m^2/day)

(f) Coefficient of consolidation in the horizontal direction C_h (m^2/day) which has been conservatively assumed to be 2x C_v

Compressibility characteristics for the ground were obtained from the lab consolidation test results, taking also into account empirical formulas for defining compressibility characteristics. The field coefficient of consolidation is a parameter extremely difficult to define simply by carrying out laboratory tests and can only be accurately defined by constructing and monitoring trial embankments. A conservative value of the mean estimate was taken into account throughout the analyses.

2.7 FILL MATERIAL

2.7.1.1 General

In Lot 1 the fill Material is to be obtained from the road excavations. More precisely, great percentage of the material will be derived from the cut slope excavations at 2+000 – 3+100 and 5+000 – 10+000. In both sections, The strata that will be excavated in order to construct the cut slopes, could provide appropriate material for embankment construction. The specifications for the selection and proper treatment of the materials, the compaction process, the layering of the materials and the quality control should be included in the relevant construction standards of the project (DIS). Trial placement may be necessary in order to decide upon the detailed construction procedure.

2.7.1.2 Classification

The fill material derived from the excavation of cut slopes is described as silty to sandy Gravel containing 50%-75% gravels, 13%-23% sand and 11%-25% fines according to the results of Grain Size Distribution tests. It exhibits a well to moderate grading with corresponding values of C_C =5-7. It contains large size round cobbles in a proportion of about 25% of total mass grains larger than 20mm and in in cases 5% larger than 37,5mm. According to USCS it can be classified as GP-GM/GC.

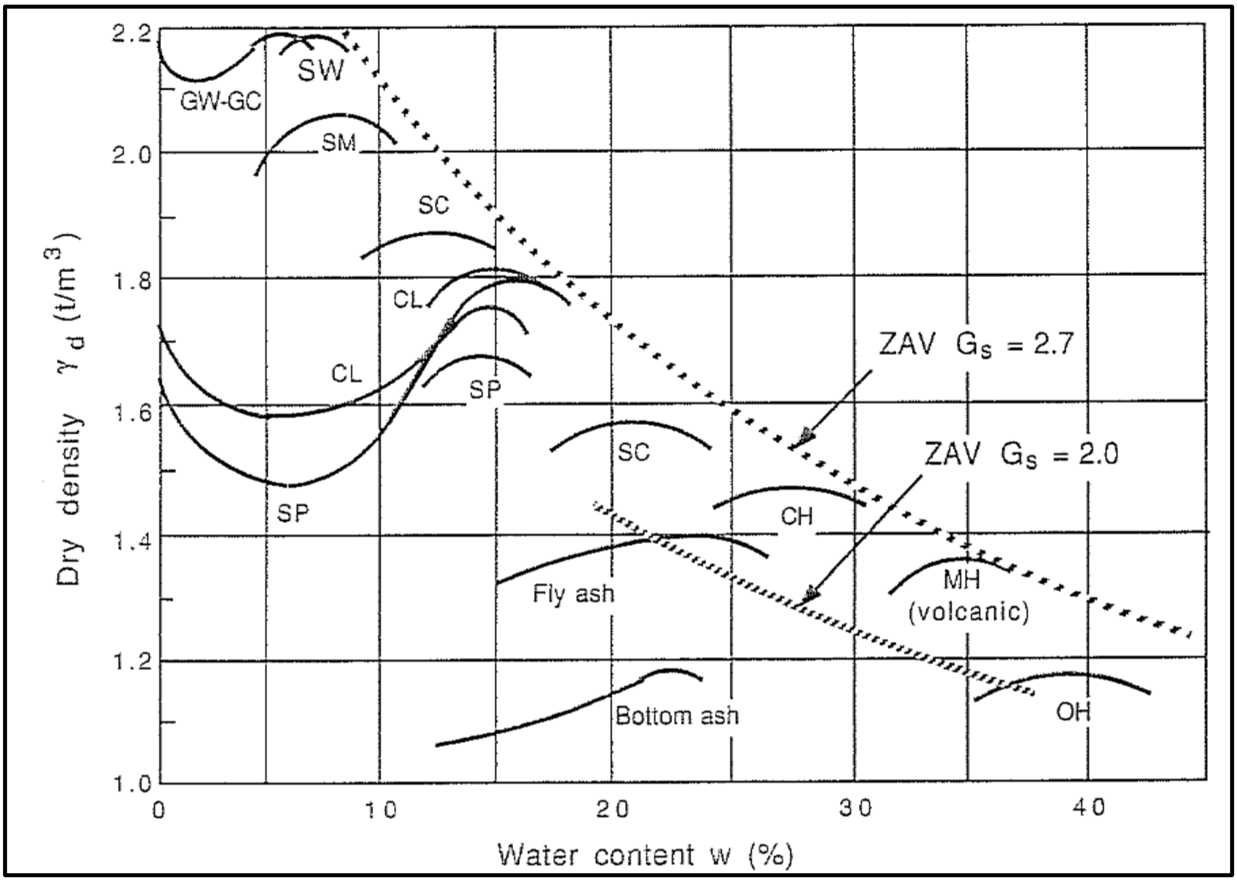


Figure 4: Typical results of compaction tests for soil types (Hausmann, 1990)

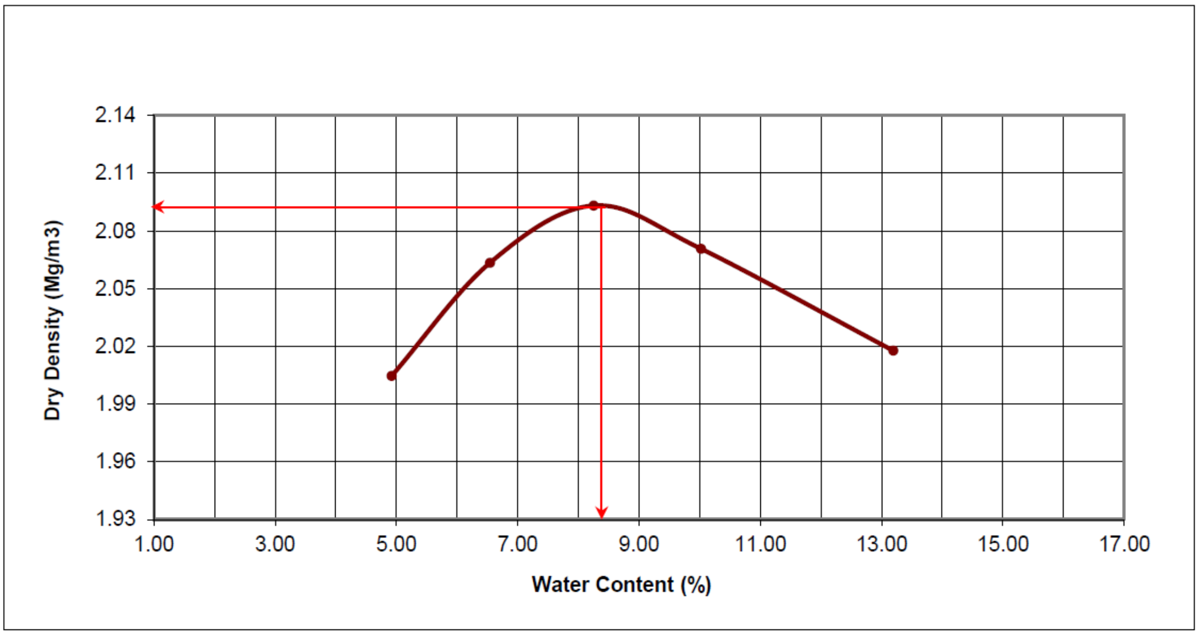
In respect to the compaction properties of the fill material, the maximum dry density and optimum water content derived from Proctor tests in two characteristic samples are ρd_{max}=2.09Mg/m^3 at w_{opt}=8.3% and ρd_{max}=1.86Mg/m^3 at w_{opt}=13.7%. According to Figure 5 the fill material tested has equivalent compaction properties to a SW-SC material and it can be classified as such when compacted.

Group Symbol	Soil type	Range of max. dry unit weight, t/m^3	Range of optimum moisture , %	Typical value of compression		**Typical strength characteristics**						
				At about 140 kPa, % orig. height	At about 350 kPa, % orig. height	Cohesion (as compacted), kPa	Cohesion (saturated) , kPa	ϕ' (effective stress envelope), degrees	tan ϕ'	Typical coeff. of permeability, m/s	Range of CBR values	Range of subgrade modulus k_1x1000 kN/m^3
GW	Well-graded clean gravels, gravel-sand mix	2.0-2.2	11-8	0.3	0.6	0	0	>38	>0.79	10^{-5}	40-80	80-140
GP	Poorly-graded clean gravels, gravel-sand mix	1.8-2.0	14-11	0.4	0.9	0	0	>37	>0.74	$5X10^{-5}$	30-60	70-110
GM	Silty gravels, Poorly-graded gravel-sand silt	1.9-2.2	12-8	0.5	1.1	-	-	>34	>0.67	$>5X10^{-10}$	20-60	30-110
GC	Clayey graels, poorly graded gravel-sand clay	1.8-2.1	14-9	0.7	1.6	-	-	>31	>0.60	$>5X10^{-11}$	20-40	30-80
SW	Well-graded clean sands, gravelly sands	1.8-2.1	16-9	0.6	1.2	0	0	38	0.79	$>5X10^{-7}$	20-40	55-80
SP	Poorly-graded clean sands, sand-gravel mix	1.6-1.9	21-12	0.8	1.4	0	0	37	0.74	$>5X10^{-7}$	10-40	55-80
SM	Silty sabds, poorly graded sand-silt mix	1.8-2.0	16-11	0.8	1.6	50	20	34	0.67	$>10^{-8}$	10-40	30-80
SM-SC	Sand-silt clay mix with slightly plastic fines	1.8-2.1	15-11	0.8	1.4	50	14	33	0.66	$>10^{-9}$	5-30	30-80
SC	clayey sands, poorly graded sand-clay mix	1.7-2.0	19-11	1.1	2.2	75	11	31	0.60	$>10^{-10}$	5-20	30-80
ML	Inorganic silts andd clayey silts	1.5-1.9	24-12	0.9	1.7	65	9	32	0.62	$>5X10^{-9}$ or less	15	30-55
ML-CL	Mixture of inorganic silt and clay	1.6-1.9	22-12	1.0	2.2	65	22	32	0.62	$>10^{-10}$	-	
CL	Inorganic clays of low to medium plasticity	1.5-1.9	24-12	1.3	2.5	85	13	28	0.54	$>5X10^{-11}$ or less	15	15-55
OL	Organic silts and silt-clays, low plasticity	1.3-1.6	33-21	-	-	-	-	-	-	-	5 or less	15-30
MH	Inorganic clayey silts, elastic silts	1.1-1.5	40-24	2.0	3.8	70	20	25	0.47	$>10^{-10}$	10 or less	15-30
CH	Inorganic clays of high plasticity	1.2-.7	36-19	2.6	3.9	105	11	19	0.35	$>5X10^{-11}$	15 or less	15-40
OH	Organic clays and silty clays	1.0-1.6	45-21	-	-	-	-	-	-	-	5 or less	5-30

Source: Adapted from "Design Manual 7.2.U.S. Navy, 1982.
Notes:
1. All properties are for condition of standard proctor maximum density, except values of k1 and CBR which are for modified proctor maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from U.S.S.R. data.
3. Compression values are for vertical loading with complete lateral confinement.
4. ">" indicates that the typical property is greater than the value shown.
5. "-" indicates that insufficient data is available for an estimate.

Table 10: Typical properties of compacted soils (From Hausman M., 1990)

Overall it can be defined as a granular fill (Specification for Highway Works, 1998) pointing that cobbles (60-200mm) and boulders (larger than 200mm) need to be removed before placement and compaction



Results:

Optimum moisture content (w_{opt}):	***8.3 %***
Maximum dry density (ρ_{dmax}):	***2.09 Mg/m^3***

Figure 5: Results from Standard proctor test from BH_L1_S1_S10

2.7.1.3 Material Properties

One big problem with fill material is to determine design strength properties. There are a lot of qualitative and quantitative methods of determining strength properties, especially for granular fills.

Typical properties of compacted soils are given in Table 10 based on soil type classification. For a soil type classified as GP values of ρd_{max}=1.8-2.0Mg/m^3, w_{opt}=11.0-14.0%, c'=0 kPa and φ'>37° are proposed. For ground type SW values of ρd_{max}=1.8-2.1Mg/m^3, w_{opt}=9.0-16.0%, c'=0 kPa and φ'=38° are proposed and for ground type SC ρd_{max}=1.7-2.0 Mg/m^3, w_{opt}=11.0-19.0%, c'=11 kPa (saturated) and φ'=31°. Comparing the proposed typical properties for compacted soils with the laboratory test results it can be said that the in site fill material when compacted is better simulated as a SW-SC compacted soil.

Eurocode 7 also suggests simple rules-of-thumb from well-established experience for determining the peak (φ) and constant volume (φ_{cv}) angles of shearing resistance of sands and gravels (Decoding Eurocode 7, Bond & Harris, 2008) shown in **Error! Reference source not found.**. For a rounded moderate graded compacted sandy-gravelly fill material the determined values are φ=38° and φ_{cv}=32°.

$$\varphi = 30^\circ + \begin{Bmatrix} 0,\ \text{rounded} \\ 2,\ \text{sub-angular} \\ 4,\ \text{angular} \end{Bmatrix} + \begin{Bmatrix} 0,\ \text{uniform} \\ 2,\ \text{moderate grading} \\ 4,\ \text{well-graded} \end{Bmatrix} + \begin{Bmatrix} 0,\ N < 10 \\ 2,\ N = 20 \\ 6,\ N = 40 \\ 9,\ N = 60 \end{Bmatrix}$$

$$\varphi_{cv} = 30^\circ + \begin{Bmatrix} 0,\ \text{rounded} \\ 2,\ \text{sub-angular} \\ 4,\ \text{angular} \end{Bmatrix} + \begin{Bmatrix} 0,\ \text{uniform} \\ 2,\ \text{moderate grading} \\ 4,\ \text{well-graded} \end{Bmatrix}$$

Figure 6. Determination of peak (φ) and constant volume (φcv) angles of shearing resistance of sands and gravels

Based on the above, the design strength parameters of the fill material can be taken as ***φ'=35°*** conservatively and ***c'=5 kPa***. The existence of cohesion can be justified by the presence of significant fine content (up to 25%) and by the fact that an apparent cohesion is developed when the soil is compacted in unsaturated conditions (i.e. optimum water content), with the precondition that the constructed embankment is protected from water content changes.

2.8 ADDITIONAL CONSIDERATIONS

1. Foundation Preparation

The embankments will be constructed either on alluvial or swamp deposits, after the removal of the top soil (including organics or other soft material). The depth of the material that will be removed has been estimated and is shown in **Error! Reference source not found.**. Material that h as been removed will be replaced with appropriate fill material, as described in the following paragraph.

Embankment	Chainage		Road	Foundation Layer (m)	Drains		
	From	To			Grid	(Q)	Total Depth (m)
#1	01+020	01+300	LR01	0.5	1	8808.00	88080.00
#2	00+210	00+410	VAZ-S1	0.6	1	5685.00	45480.00
SUM:						14493.00	133560.00

Table 11. Foundation of the Embankments

2. Drainage Layer of the Foundation of the Embankment

A drainage layer of coarse-grained material will be placed at the base of the embankment.

3. Measures against Slope Erosion

The embankment slopes will be constructed generally at an inclination 1.75:3 to 2:3 (ver:hor). A layer of top soil, 30 cm thick will be placed on all embankment slopes and vegetated. The method of vegetation establishment, that will define the type and density of plants and means of application, will be the subject to the construction.

	Instrumentation					
Embankment	Piezometer (Q)	Piezometer (m)	Wire Settlement cell (Q)	Extensiometer (Q)	Extensiometer (m)	Settlement plates
#1	2	50	2	2	60	2
#2	2	33	2	2	53	2
SUM:	4	83	4	4	113	4

Table 12. Instrumentation of the embankments

4. Anchor Berms

Where the embankment is to be founded on sloping ground with an inclination more than 20%, anchor berms (steps, terraces) will be constructed. For the construction of the embankments of the present design there is no requirement for anchor berms.

3 GEOTECHNICAL DESIGN OF EMBANKMENTS

3.1 EMBANKMENT ON LR01 BETWEEN CH. 1+020 – CH. 1+300

A. GENERAL

At motorway chainage 1+514, a motorway overpass is being foreseen, in order to maintain the local traffic and connect the areas astride of the motorway. Considering the fact that an overpass bridge structure is going to be constructed and the LR01 is paved, the approach embankments of the bridge should be designed with limited allowable long-term settlements. In contrast to the generally accepted level of the allowable settlements of 15cm in embankments, in this approach embankments the long-term consolidation settlements should not exceed 5cm.

The overpass bridge is going to be constructed in between the Chs. 1+182 - 1+242. The approach embankment for the Northwestern abutment extends from 1+020 to the abutment of the bridge at 1+182.From Southeastern side the approach embankment extends from the abutment of the bridge at 1+242 to 1+300, for length where the embankment of the LR01 exceeds the height of 4m. The total length of the designed approach embankment is 220m.

The proposed inclination of the slopes of the fill, is 2:3 (ver:hor). The maximum height of the slopes (from toe to crest) is about 9.50m (Ch. 1+242).

In this area, one Borehole and one Trial Pit were carried as summarized in Table 13. From the borehole it was found that the foundation soil consists of fine-grained Deposits and more precisely, impermeable hard silty Clays. At depth of 9m permeable medium dense sandy deposits are encountered.

With regards to the water table, water level was encountered at 7.00m, however due to Y50 it is expected at the ground level.

Mot. Chainage	BH-TP	Depth (m)	Coordinates			Ground Water Level BGL. (m)
			E(m)	N(m)	Z(m)	
1+514	BH-L1-S1-S01	15.00	4615945.55	503038.926	584.595	7.00
1+560	TR-L1-S1-02	3.40	5031106.53	4615950.97		-

Table 13:List of Boreholes and Trial Pits in the area between Ch. 1+020 – Ch. 1+300 of LR01

B. GROUND PROFILE

Based on engineering geological mapping and the correlation of the carried-out investigation, in the wider area, it can be concluded that the geological conditions are similar and the stratigraphic layers are horizontal, in the area of the embankment and the bridge. Therefore, in cases where the embankment presents its maximum height, the most unfavorable conditions are expected. So, the geotechnical analyses are focused at that cases and more precisely cross sections. Slope stability analyses and settlement calculations were carried out on one typical analysis section, at Ch. 1+242 of LR01, where this maximum embankment height of 9.5m is encountered. The typical embankment cross section has a top width of about 9.8 m and side slopes with inclination 2:3 (ver: hor).

The Ground conditions were taken in accordance to the interpretation of BH-L1-S1-S01 and TR-L1-S1-02, presented in Table 14. Details with regards the investigations are presented in the Factual reports. It should be pointed out that the encountered top soil, thickness indicated in Table 14 should be removed. The Annual maximum water level is assumed to be at 7.0m below the ground level and the maximum water table for a return period of 50 years is conservatively assumed to be at ground surface. It is also assumed that the ground at 35m below ground level is practically incompressible.

LR01 Embankment Ch. 1+020 – CH. 1+300					Laboratory Tests - ლაბორატორიის ტესტირების შედეგები				
					Organic Content- ირგანული შემცველობა	Grain Size Distrubution ინერტული მასალის ზომის განაწილება			
Layers	Depth (m) სიღრმე (მ)	Description /აღწერილობა				Gravel (%) ღორღი (%)	Sand (%) ქვიშა (%)	Silt (%) შლამი /ნატანი	Clay (%) თიხა (%)
Layer 1	0-0.50	Very soft, deep brown organic CLAY. Presence of vegetation and angular gravel (Top Soil)			N/A	-	-	-	-
Layer 2	0.50-9.00	Very hard, light brown to brown, overconsolidated silty CLAY of high plasticity. Presence of oxides as well as crystals from gypsum and calcite. Thin layers, less than 10cm, of very dense Sands within a matrix composed of calcite are presented. Also, greenish layers of highly plastic Clay are frequent. (Neogene deposits).			-	0.00%	9.04%	35.50%	55.46%
Layer 3	9.00-25.00	Medium dense, brown silty SAND with gravel and clay			-	21.21%	44.76%	21.18%	12.85%
Layers	Depth (m) სიღრმე (მ)	Atterberg Limits ატერბერგის ლიმიტი			Liquidity Index (I_L) - კონსისტენციის ინდექსი	Deformation Modulus According to SNIP 2.02.01.83* დეფორმაციის მოდული SNIP 2.02.01.83*-ს შესაბამისად (Mpa)	Mechanical properties / მექანიკური მახასიათებლები		
		Plastic Limit PL(%) პლასტიური ზღვარი (%)	Liquid Limit LL(%) დენადობის ზღვარი (%)	Plasticity Index PI (%) პლასტიურობის რიცხვი (%)			Unconfined Compression Strength UCS (kPa) - განუსაზღვრელი სიმტკიცე კუმშვისა (kPa)	Undrained Shear Strength Cu(kPa) - კონსოლიდირებული სიმტკიცე ძვრაზე	Deformation Modulus E (mPa) - დეფორმაციის მოდული
Layer 1	0-0.50	N/P			N/A				
Layer 2	0.50-9.00	21.70%	44.20%	22.50%	0.24	18.00	N/A		
Layer 3	9.00-25.00	-			0.2*	-			
Layers	Depth (m) სიღრმე (მ)	Physical Properties / ფიზიკური მახასიათებლები						Mechanical properties / მექანიკური მახასიათებლები	
		Moisture Content- ტენიანობა	Bulk Density kN/m³ - მოცულობითი წონა	Dry Density kN/m³- მშრალი სიმკვრივე	Specific Gravity kN/m³ - კუთრი წონა	Void Ratio e_0 ფორიანობის კოეფიციენტი	Saturation Degree S (%) გაკერების ხარისხი (%)	Cohesion (kPa) შეჭიდულობა(kPa)	Friction Angle (°) - შინაგანი ხახუნის კუთხე
Layer 1	0-0.50	N/A							
Layer 2	0.50-9.00	29.57%	19.27	14.87	26.97	0.81	100.00%	44.37	16.94
Layer 3	9.00-25.00	N/A							
Layers	Depth (m) სიღრმე (მ)	Standard Penentation Test Based Data /მონაცემები და გამკვრივების მონაცემები							Water Table / წყლის დონე
		$N_{SPT\ Evaluated}$	Type of Soil- გრუნტის ტიპი	Fricton Angle φ´(degrees) - ხახუნის კუთხე	Unconfined Compression Strength - qu (kPa) - განუსაზღვრელი სიმტკიცე კუმშვისას	Undrained Shear Strength Cu(kPa) - არადრენირებული სიმტკიცე ძვრაზე	Deformation Modulus Es (mPa) - დეფორმაციის მოდული	Compressibility Index კუმშვადობის ინდექსი Ic	
Layer 1	0-0.50								
Layer 2	0.50-9.00	39	CLAY	N/A	468.00	234.00	17.28	0.01	7m
Layer 3	9.00-25.00	14	SAND	31.20	168.00	-	12.00	0.04	
Layers	Depth (m) სიღრმე (მ)	Consolidation Based Data /გამკვრივების მონაცემები							
		Pressure (kPa)	e	Mv (MPa^{-1})	E_{soed}(MPa)	a_V (MPa^{-1})	Cc	Cv (m^2/year)	k (m/s)
Layer 1	0-0.50	N/A							
Layer 2	0.50-9.00	25.00	0.80	0.07	14.29	1.87	0.00	3.52	7.66E-11
		50.00	0.80	0.13	7.69	1.93	0.00	2.13	8.61E-11
		100.00	0.79	0.12	8.33	1.91	0.03	1.66	6.20E-11
		200.00	0.77	0.10	10.00	1.87	0.07	1.98	6.15E-11
		400.00	0.73	0.11	9.09	1.84	0.13	1.24	4.24E-11
		800.00	0.66	0.10	10.00	1.76	0.23	2.52	7.85E-11
		200.00	0.67	0.01	100.00	1.68	0.02	5.66	1.76E-11
Layer 3	9.00-25.00	N/A							
* Assumed Values / სავარაჟდო მონაცემები									

Table 14: Typical geotechnical unit properties for LR01 local road embankment between Ch. 1+020 – CH. 1+300

C. SLOPE STABILITY ANALYSIS

Slope stability analysis were carried out for the analysis section described previously. The fundamental period of the embankment for the analysis section is T = 0.05 sec. Using PGA = 0.12 g, which is representative for the area of Ch.0+031 - 7+338 of the motorway. For this case the horizontal pseudostatic acceleration coefficients α_H = 0.108 g, and the vertical α_V = 0.054 g for subsoil category C corresponding to an embankment height of 9.50m (Table 15)

Height (m)	Period T(sec)	Soil Class C			
		αB (g)	αK (g)	αh(g)	av(g)
5	0.04	0.0805	0.107	0.094	0.047
7.5	0.06	0.0805	0.121	0.101	0.050
10	0.08	0.0805	0.134	0.107	0.054
12.5	0.10	0.0805	0.141	0.111	0.055
15	0.13	0.0805	0.155	0.118	0.059
17.5	0.15	0.0805	0.167	0.124	0.062
20	0.17	0.0805	0.181	0.131	0.065

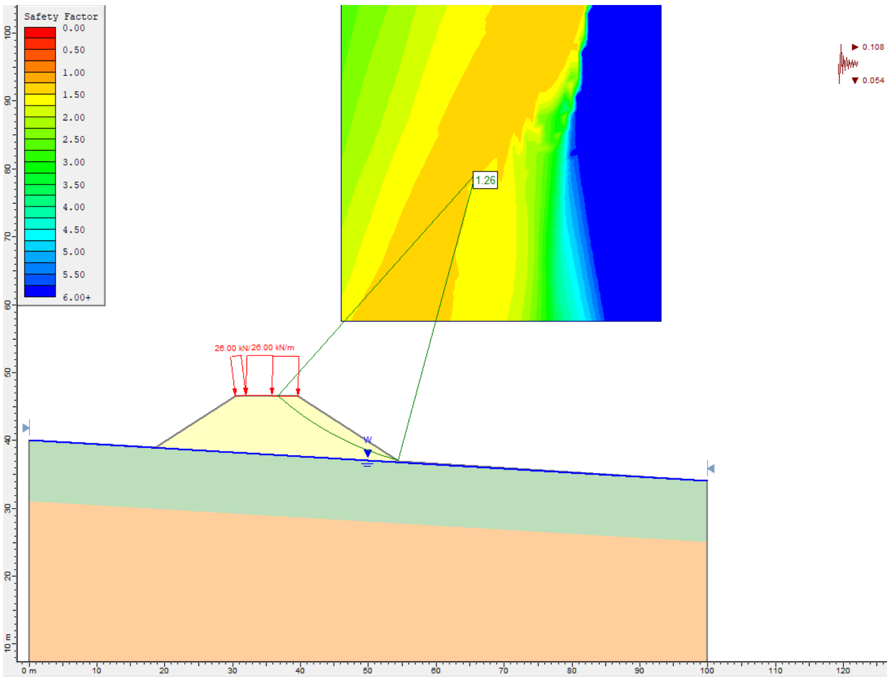
Table 15: Calculation of horizontal and vertical pseudostatic acceleration coefficients for ground type C according to the height of the embankment

For the analysis section the depth of the maximum annual water table (A) is assumed at 7.00m deep and the maximum water table for a return period of 50 years (Y50) at ground surface.
The embankment material has a unit weight of γ=20.6 KN/m^3 and design strength parameters c'=5 kPa, φ=35° or better.
The traffic load is q=20 kPa and according to Eurocode 7 it is an unfavorable geotechnical action with a partial factor of 1.3 (§2.4.7.3.4.4 of EN 1997-1), and thus the design traffic load is q_d=26 kPa.
The calculated factors of safety for the analysis section is presented in Table 16.
The calculated factors of safety, for all the load combinations examined are above the minimum values specified by the relevant regulations. Therefore, we can conclude that the embankment design is acceptable.

Load Combination		Specification	Required Factor of Safety	Janbu cor.	Bishop
FS1	Short Term Conditions	Eurocode 7	**1.40**	**1.53**	**1.52**
FS2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7/8	**1.10**	**1.25**	**1.26**
FS3	Long Term Conditions +Water level (Y50)	Eurocode 7	**1.25**	**1.53**	**1.52**
FS4	Long term Conditions +Completely Dry conditions	Eurocode 7/8	**1.38**	**1.53**	**1.52**

Table 16: Calculated factors of Safety for Analysis section (LR01 CH. 1+242)

Chainage LR01 1+242	
Load combination FS2 - Long Term Conditions+ Earthquake + Water level (A)	



Embankment Slope Data	
Height:	9.50m
Inclination	2 / 3
Geological Conditions	
Total Layers:	2
Layer 2 (green):	Very stiff Clay
γ (kN/m^3):	19.27
γ_d(kN/m^3):	14.87
C(kPa):	43
Φ(°):	18
Layer 3 (brown):	Clayey Gravel
γ (kN/m^3):	19.27
γ_d(kN/m^3):	14.87
C(kPa):	0
Φ(°):	31
Water Level:	Yes
PGA:	0.12
Kh:	0.108
Kv:	0.054
FS (Bishop):	1.26>1.10
FS (Janbu Cor):	1.25>1.1

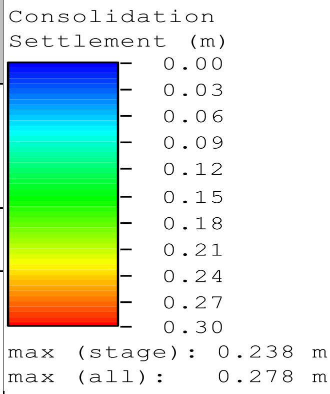
Table 17: Stability analysis results for FS2 Load combination for analysis section I (Ch. 1+242)

D. CALCULATION OF SETTLEMENTS

Due to the presence of fine-grained deposits, it can be assumed that the under-design embankment beside the immediate settlements will also suffer consolidation settlements due to the presence of these deposits. In addition, it is understood that since the Clayey ground presents very low permeability, the rates of the consolidation settlements will be very low. Which means that the consolidation settlements will occur during the operation period. In order to avoid the deferential settlements and the relative failures in the transmission of the Bridge to the embankment the design foresees specific counter measures.

As pointed out in Table 14, the ground consists of 9m of clay, which will suffer consolidation settlements. This formation overlays on a Medium dense, brown silty Sand with gravel and clay. Beside the fact that the loads on this formation are limited, it is expected that this material to experience immediate settlements. In addition, for the foundation of the embankment should have thickness of 0.5m and to be Class 6-B compacted coarse granular material. In the analysis the material is characterized with 50MPa deformation modulus and unit weight of γ=20.6kN/m^3. In addition, a traffic load of 26kPa is being takin into construction after the construction of the pavement. It should be pointed out that the impact of this additional load is limited.

Settlements and preloading.	Immediate settlements (cm)	Consolidation settlements (cm)	Total settlements (cm)
Total Settlements without measures On 9 months of pre-loading	21.5	5.76	27.26
Total Settlements without measures	21.5	18.4	41.8
Total Settlements with Strip Drains on 9 months of pre-loading	21.5	23.8	45.3
Total Settlements with Strip Drains	21.5	27.8	51.2



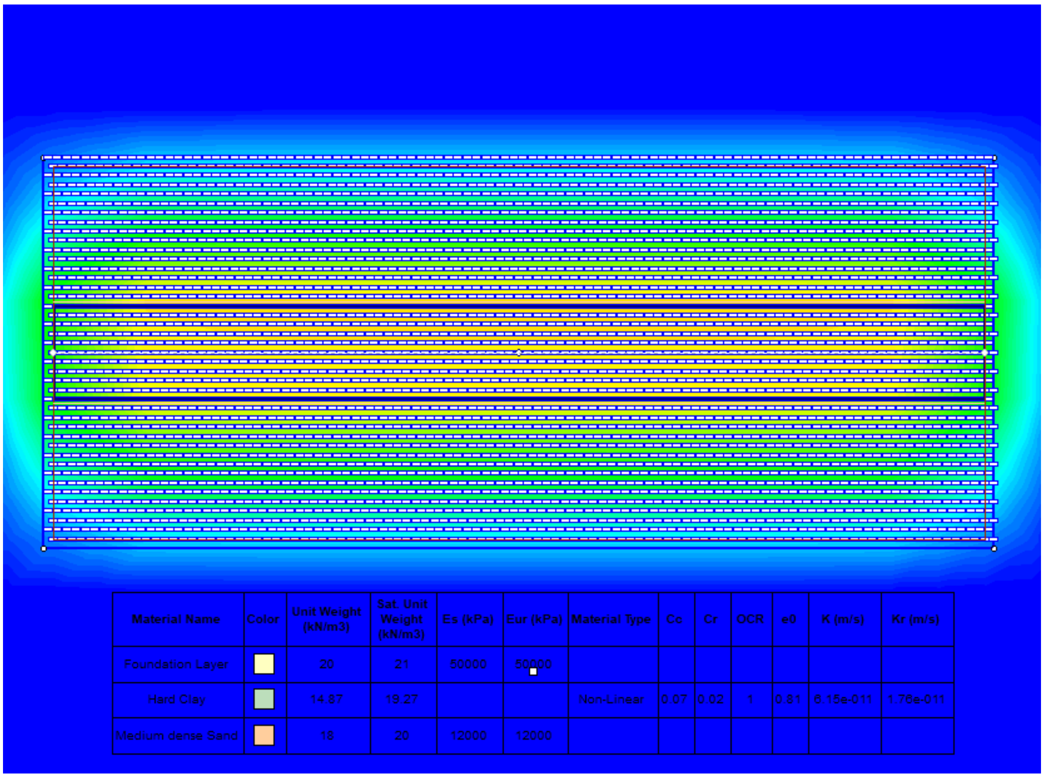


Table 18. Summary with the immediate and consolidation settlements with and without drains.

The results from the settlement calculation, carried out in Computer Program Settle3D v2.0 (RocScience Inc.), are presented in Table 18. More precisely at 9 months of preloading without counter measures the consolidation settlements reach 5.76cm from the total 18.4cm. This means that in long term the pavement is going to receive 18.4-5.76=12.64 cm. In case that there was no bridge, this settlement would have been acceptable. However, considering the construction of the bridge, which is a rigid structure, the differential settlement on the pavement of the bridge and the pavement of the embankment in high. Therefore, counter measures are being foreseen in order to reduce these long-term settlements. More precisely, in order to reduce the long-term settlements below the level of 5cm, Triangular Strip Drains in a grid of 1.0X1.0, with depth of 10m, are recommended. With the drains the total consolidation settlements at preloading period of 9

months are 23.8cm from the total 27.8. This means that in long term the pavement is going to receive 27.8-23.8=4.0cm<5cm which means that it is acceptable.
In addition to that a post construction creep of the embankment materials can be estimated based on empirical relationships and construction experience as follows:
-0.2%*Hi, for the upper 10m
-0.4%*Hi for height 10-20m from crest
-0.6%*Hi for height >20m from crest, where Hi the width of each layer.
Based on the above a post construction creep of 1.90cm is estimated.

E. CONSTRUCTION GUIDELINES

Prior to placing the foundation layer any topsoil should be stripped and the embankment foundation checked to ensure that all weak, weathered or otherwise unsuitable materials have been removed during excavation and that design formation level has been reached.
drain pattern is installed, with geosynthetic drain strips of 10cm width, 0.4cm thickness, 10m length and a drain spacing of 1.0m. The drains will be installed with pre-drilling.
The foundation (starter) layer is then placed with a thickness of 0.5m consisting of a Class 6-B compacted coarse granular material. This layer behaves also as a basal drainage blanket in conjunction with the vertical drain pattern.
On top of the foundation layer a stabilizing-separation geotextile wrapped under the starting layer should be placed with a typical tensile strength of 30kN/m. The embankment fill material should be of Class 6-I or 6-J and it should be compacted in layers of 0.30m thickness.
Resting Period of nine months before the construction of the pavement is proposed.

F. INSTRUMENTATION

Because of the drainage installed and in order to certify the goals that should be reached (no excess pore pressures generated during construction) it is crucial to monitor the evolution of the settlements and of pore pressures via specifically installed instruments as follows (Table 19):
(a) Pore pressure monitoring devices (Pi).
Two instruments will be installed in order to measure the built up and subsequent dissipation of pore water pressures. The piezometer tip will be installed at a depth of 10.0m below foundation level at Ch. 1+200 and at a depth of 10.0m below foundation level at Ch.1+160.
(b) Instruments to measure the evolution of settlements (Li+Si)
Two types of instruments will be installed in order to measure the evolution of settlements both at depth and at the foundation level. The instruments will be comprised of a borehole extensometer or settlement gauges (LI) installed down to a depth of 30m below foundation and of a foundation settlement plate or digital hydrostatic profile gauge or vibrating wire settlement cell or other equivalent monitoring device (Si). In addition to that two settlement plates (Si) will be installed on top of the embankment when completed, in order to monitor the post construction creep settlement of the fill material
Details regarding the type of the monitoring devices will be proposed by the Constructor and will be approved by the Supervision.

Instrument	Location	Depth below foundation (m)
S1+L1+P1+S2	Ch.1+160	0.0+30.0+20.0-15.00
S3+L2+P2+S4	Ch.1+200	0.0+30.0+20.0-15.00

Table 19: Location and Depth of Instruments

The frequency of the measurements is as follow:

- All instruments will be installed and measured after the surface stripping of the top soil.
- All instruments will be measured before the placement of embankment material (1 measurement).
- All instruments will be measured once every two meters (1 m) of placed embankment material (approximately 2 measurements).
- All instruments will be measured at the end of construction of the embankment stage (1 measurement).
- All instruments will be measured 0.5, 1, 3, 5, 9, 12 months after completion of the full embankment (6 measurements).

Further measurements will be decided if necessary after the evaluation of the measurements at the end of 270 days of preloading. Depending on the evolution of settlements final decisions will be taken for the feasibility of the construction of the pavement.

G. CONCLUSIONS

In the present report the highway embankments between Ch. 1+020 - 1+182 and 1+242 – 1+300, have been studied, based on the Final Design. Based on the results of the ground investigation, in this the area it was assumed that similar geological and geotechnical conditions prevail. Then, a representative embankment cross section was selected (Ch. 1+242) where the embankment reaches maximum height and both slope stability and settlement calculations were carried out.

Taking into account all available data, the earthworks layout is acceptable. Furthermore, a triangular wick drain pattern is proposed, comprising of geosynthetic drain strips of 10cm width, 0.4cm thickness, 10m length and a drain spacing of 1.0m, in order to reduce the preloading time for the foundation layers to be consolidated in an acceptable degree. Wick drains are suggested in the areas where the embankment height is >4.0m, i.e from Ch. 1+020 - 1+182 and 1+242 to 1+300,

A rest period before the construction of the pavement of minimum 270 days (9 months) is proposed.

3.2 EMBANKMENT ON VAZ-S1 BETWEEN CH. 0+210 – CH. 0+410

A. GENERAL

At motorway chainage 3+578, a motorway overpass is being foreseen, for the construction of an interchange. Considering the fact that an overpass bridge structure is going to be constructed and the VAZ-S1 is paved, the approach embankments of the bridge should be designed with limited allowable long-term settlements. In contrast to the generally accepted level of the allowable settlements of 15cm in embankments, in this approach embankments the long-term consolidation settlements should not exceed 5cm.

The overpass bridge is going to be constructed in between the Chs. 0+310 - 0+370. The approach embankment for the Southern abutment extends from 0+210 to the abutment of the bridge at 0+310. From northern side the approach embankment extends from the abutment of the bridge at 0+370 to 0+410 for length where the embankment of the VAZ-S1 exceeds the height of 4m. The total length of the designed approach embankment is 210m.

The proposed inclination of the slopes of the fill, is 2:3 (ver:hor). The maximum height of the slopes (from toe to crest) is about 6.50m (Ch. 0+300).

In this area, one Borehole and one Trial Pit were carried as summarized inTable 20. From the borehole it was found that the foundation soil consists of fine-grained Deposits and more precisely, impermeable hard silty Clays.

With regards to the water table, water level was encountered at 5.00m, however due to Y50 it is expected at the ground level.

Mot. Chainage	BH-TP	Depth (m)	Coordinates			Ground Water Level BGL. (m)
			E(m)	N(m)	Z(m)	
3+578	BH-L1-S1-S06	15.00	4616196.57	504907.70	649.342	5.00
3+620	TR-L1-S1-04	2.80	504928.03	4615951.99	-	-

Table 20: List of Boreholes and Trial Pits in the area between Ch. 1+020 – Ch. 1+300 of LR01

B. GROUND PROFILE

Based on engineering geological mapping and the correlation of the carried-out investigation, in the wider area, it can be concluded that the geological conditions are similar and the stratigraphic layers are horizontal, in the area of the embankment and the bridge. Therefore, in cases where the embankment presents its maximum height, the most unfavorable conditions are expected. So, the geotechnical analyses are focused at that cases and more precisely cross sections. Slope stability analyses and settlement calculations were carried out on one typical analysis section, at Ch. 0+300 of VAZ-S1, where this maximum embankment height of 6.5m is encountered. The typical embankment cross section has a top width of about 12.00 m and side slopes with inclination 2:3 (ver: hor).

The Ground conditions were taken in accordance to the interpretation of BH-L1-S1-S06 and TR-L1-S1-04, presented in Table 21 Details with regards the investigations are presented in the Factual reports. It should be pointed out that the encountered top soil, thickness indicated in Table 21 should be removed. The Annual maximum water level is assumed to be at 5.0m below the ground level and the maximum water table for a return period of 50 years is conservatively assumed to be at ground surface. It is also assumed that the ground at 35m below ground level is practically incompressible.

Vaz - S1 Embankment from 0+210 - 0+420 Mot Ch.3+578.80			Laboratory Tests - ლაბორატორიის ტესტირების შედეგები				
			Organic Content- ირგანული შემცველობა	Grain Size Distrubution ინერტული მასალის ზომის განაწილება			
Layers	Depth (m) სიღრმე (მ)	Description /აღწერილობა		Gravel (%) ღორღი (%)	Sand (%) ქვიშა (%)	Silt (%) შლამი /ნატანი	Clay (%) თიხა (%)
Layer 1	0-0.70	Very soft, deep brown organic CLAY. Presence of vegetation and angular gravel (Top Soil)	N/A				
Layer 2	0.70-8.30	Medium stiff to stiff, dark colored, silty CLAY with rounded gravel and sand. Calcite veins are observed. (Neogene deposits)	-	2.86%	14.20%	30.66%	52.27%
Layer 2	8.30-20.00	Very hard, light brown to brown, overconsolidated silty CLAY of high plasticity. Presence of oxides as well as crystals from gypsum and calcite. Thin layers, less than 10cm, of very dense Sands within a matrix composed of calcite are presented. Also, greenish layers of highly plastic Clay are frequent (Neogene deposits).	-	0.00%	4.16%	36.46%	59.38%

Layers	Depth (m) სიღრმე (მ)	Atterberg Limits ატერბერგის ლიმიტი			Liquidity Index (I_L) - კონსისტენციის ინდექსი	Deformation Modulus According to SNIP 2.02.01.83* დეფორმაციის მოდული SNIP 2.02.01.83*-ს შესაბამისად (Mpa)	Mechanical properties / მექანიკური მახასიათებლები		
		Plastic Limit PL(%) პლასტიური ზღვარი (%)	Liquid Limit LL(%) დენადობის ზღვარი (%)	Plasticity Index PI (%) პლასტიურობის რიცხვი (%)			Unconfined Compression Strength UCS (kPa) - განუსაზღვრელი სიმტკიცე კუმშვისას (kPa)	Undrained Shear Strength Cu(kPa) - კონსოლიდირებული სიმტკიცე ძვრაზე	Deformation Modulus E (mPa) - დეფორმაციის მოდული
Layer 1	0-0.70	N/A							
Layer 2	0.70-8.30	25.60%	48.30%	22.70%	0.10	18.00	N/A		
Layer 3	8.30-20.00	23.80%	46.20%	23.80%	0.54	16.00			

Layers	Depth (m) სიღრმე (მ)	Physical Properties / ფიზიკური მახასიათებლები						Mechanical properties / მექანიკური მახასიათებლები	
		Moisture Content- ტენიანობა	Bulk Density kN/m³ - მოცულობითი წონა	Dry Density kN/m³- მშრალი სიმკვრივე	Specific Gravity kN/m³ - კუთრი წონა	Void Ratio e_0 ფორიანობის კოეფიციენტი	Saturation Degree S (%) გაჯერების ხარისხი (%)	Cohesion (kPa) შეჭიდულობა(kPa)	Friction Angle (°) - შინაგანი ხახუნის კუთხე
Layer 1	0-0.70	N/A							
Layer 2	0.70-8.30	30.30%	19.17	14.71	26.97	0.83	100%	43.71	16.45
Layer 3	8.30-20.00	39.02%	18.08	13.01	26.97	1.07	100%	44.59	16.36

Layers	Depth (m) სიღრმე (მ)	Standard Penentation Test Based Data /მონაცემები და გამკვრივების მონაცემები							Water Table / წყლის დონე
		N_{SPT} Evaluated	Type of Soil- გრუნტის ტიპი	Fricton Angle φ´(degrees) - ხახუნის კუთხე	Unconfined Compression Strength - qu (kPa) - განუსაზღვრელი სიმტკიცე კუმშვისას	Undrained Shear Strength Cu(kPa) - არადრენირებული სიმტკიცე ძვრაზე	Deformation Modulus Es (mPa) - დეფორმაციის მოდული	Compressibility Index კუმშვადობის ინდექსი Ic	
Layer 1	0-0.70	N/A							5.00m
Layer 2	0.70-8.30	21	CLAY	N/A	246.00	123.00	11.36	0.02	
Layer 3	8.30-20.00	49	CLAY	N/A	585.00	292.50	20.40	0.01	

Layers	Depth (m) სიღრმე (მ)	Consolidation Based Data /გამკვრივების მონაცემები							
		Pressure (kPa)	e	Mv (MPa^{-1})	E_{soed}(MPa)	a_V (MPa^{-1})	Cc	Cv (m^2/year)	k (m/s)
Layer 1	0-0.70	N/A							
Layer 2	0.70-8.30	25	0.80	0.36	2.78	2.16		3.15	3.53E-10
		50	0.80	0.12	8.33	1.92	0.00	2.27	8.48E-11
		100	0.79	0.14	7.14	1.93	0.03	4.24	1.85E-10
		200	0.77	0.11	9.09	1.88	0.07	4.25	1.45E-10
		400	0.74	0.07	14.29	1.81	0.10	2.93	6.38E-11
		800	0.70	0.05	20.00	1.75	0.13	5.38	8.36E-11
		200	0.72	0.02	50.00	1.74	0.03	3.04	1.89E-11
Layer 3	8.30-20.00	25	1.06	0.12	8.33	2.18	0.00	1.93	7.20E-11
		50	1.06	0.10	10.00	2.16	0.00	1.25	3.90E-11
		100	1.05	0.10	10.00	2.15	0.03	0.81	2.53E-11
		200	1.03	0.09	11.11	2.12	0.07	0.73	2.05E-11
		400	1.00	0.05	20.00	2.05	0.10	1.56	2.43E-11
		800	0.98	0.03	33.33	2.01	0.07	0.81	7.59E-12
		200	1.00	0.02	50.00	2.02	0.03	6.21	3.86E-11

* Assumed Values / სავარაუდო მონაცემები

Table 21: Typical geotechnical unit properties for LR01 local road embankment between Ch. 0+210-0+410

C. SLOPE STABILITY ANALYSIS

Slope stability analysis were carried out for the analysis section described previously. The fundamental period of the embankment for the analysis section is T = 0.05 sec. Using PGA = 0.12 g, which is representative for the area of Ch.0+031 - 7+338 of the motorway. For this case the horizontal pseudostatic acceleration coefficients $α_H$ = 0.101 g, and the vertical $α_V$ = 0.05 g for subsoil category C corresponding to an embankment height of 6.50m (Table 22)

Height (m)	Period T(sec)	Soil Class C			
		αB (g)	αK (g)	αh(g)	av(g)
5	0,04	0,0805	0,107	0,094	0,047
7,5	0,06	0,0805	0,121	0,101	0,050
10	0,08	0,0805	0,134	0,107	0,054
12,5	0,10	0,0805	0,141	0,111	0,055
15	0,13	0,0805	0,155	0,118	0,059
17,5	0,15	0,0805	0,167	0,124	0,062
20	0,17	0,0805	0,181	0,131	0,065

Table 22: Calculation of horizontal and vertical pseudostatic acceleration coefficients for ground type C according to the height of the embankment

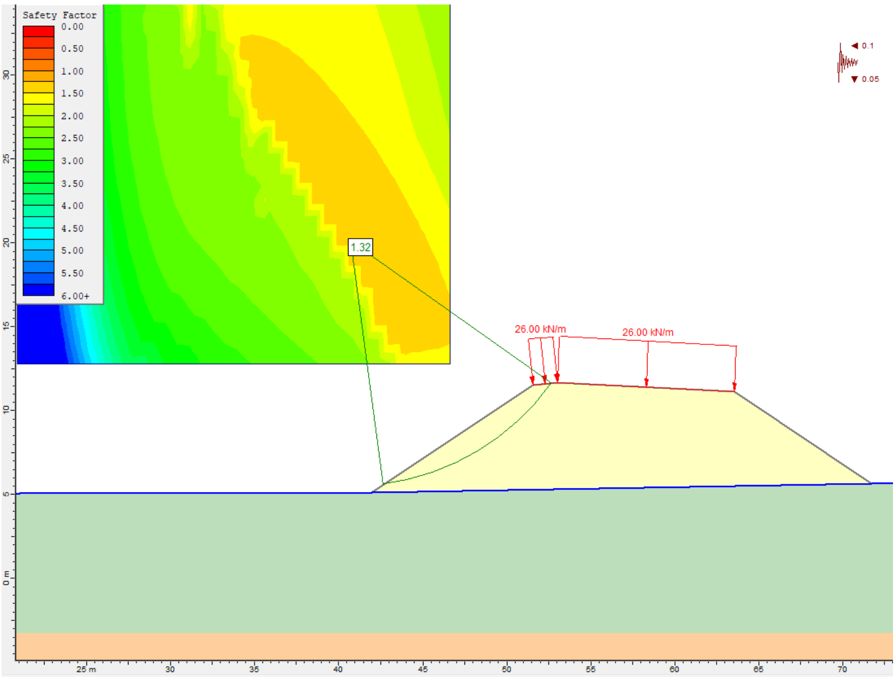
For the analysis section the depth of the maximum annual water table (A) is assumed at 7.00m deep and the maximum water table for a return period of 50 years (Y50) at ground surface.
The embankment material has a unit weight of γ=20.6 KN/m^3 and design strength parameters c'=5 kPa, φ=35^o or better.
The traffic load is q=20 kPa and according to Eurocode 7 it is an unfavorable geotechnical action with a partial factor of 1.3 (§2.4.7.3.4.4 of EN 1997-1), and thus the design traffic load is q_d=26 kPa.
The calculated factors of safety for the analysis section is presented inTable 23.

The calculated factors of safety, for all the load combinations examined are above the minimum values specified by the relevant regulations. Therefore, we can conclude that the embankment design is acceptable.

Load Combination		Specification	Required Factor of Safety	Janbu cor.	Bishop
FS1	Short Term Conditions	Eurocode 7	**1.40**	**1.53**	**1.52**
FS2	Long Term Conditions+Earhquake+Water level (A)	Eurocode 7/8	**1.10**	**1.30**	**1.32**
FS3	Long Term Conditions +Water level (Y50)	Eurocode 7	**1.25**	**1.54**	**1.56**
FS4	Long term Conditions +Completely Dry conditions	Eurocode 7/8	**1.38**	**1.56**	**1.54**

Table 23: Calculated factors of Safety for Analysis section (LR01 CH. 1+242)

Chainage LR01 1+242	
Load combination FS2 - Long Term Conditions+ Earthquake + Water level (A)	



Embankment Slope Data	
Height:	6.50m
Inclination	2 / 3
Geological Conditions	
Total Layers:	2
Layer 2 (green):	Very stiff Clay
γ (kN/m^3):	19.17
γ_d(kN/m^3):	14.71
C(kPa):	43.71
Φ(°):	16.45
Layer 3 (brown):	Hard Clay
γ (kN/m^3):	18.08
γ_d(kN/m^3):	18.08
C(kPa):	44.59
Φ(°):	16.36
Water Level:	Yes
PGA:	0.12
Kh:	0.10
Kv:	0.05
FS (Bishop):	1.32>1.1
FS (Janbu Cor):	1.30>1.1

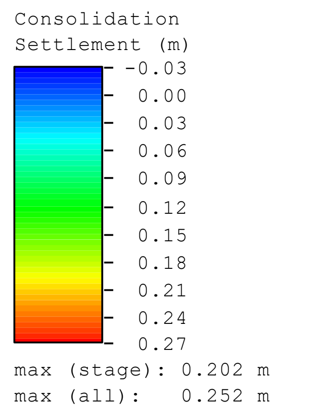
Table 24: Stability analysis results for FS2 Load combination for analysis section (Ch. 1+242)

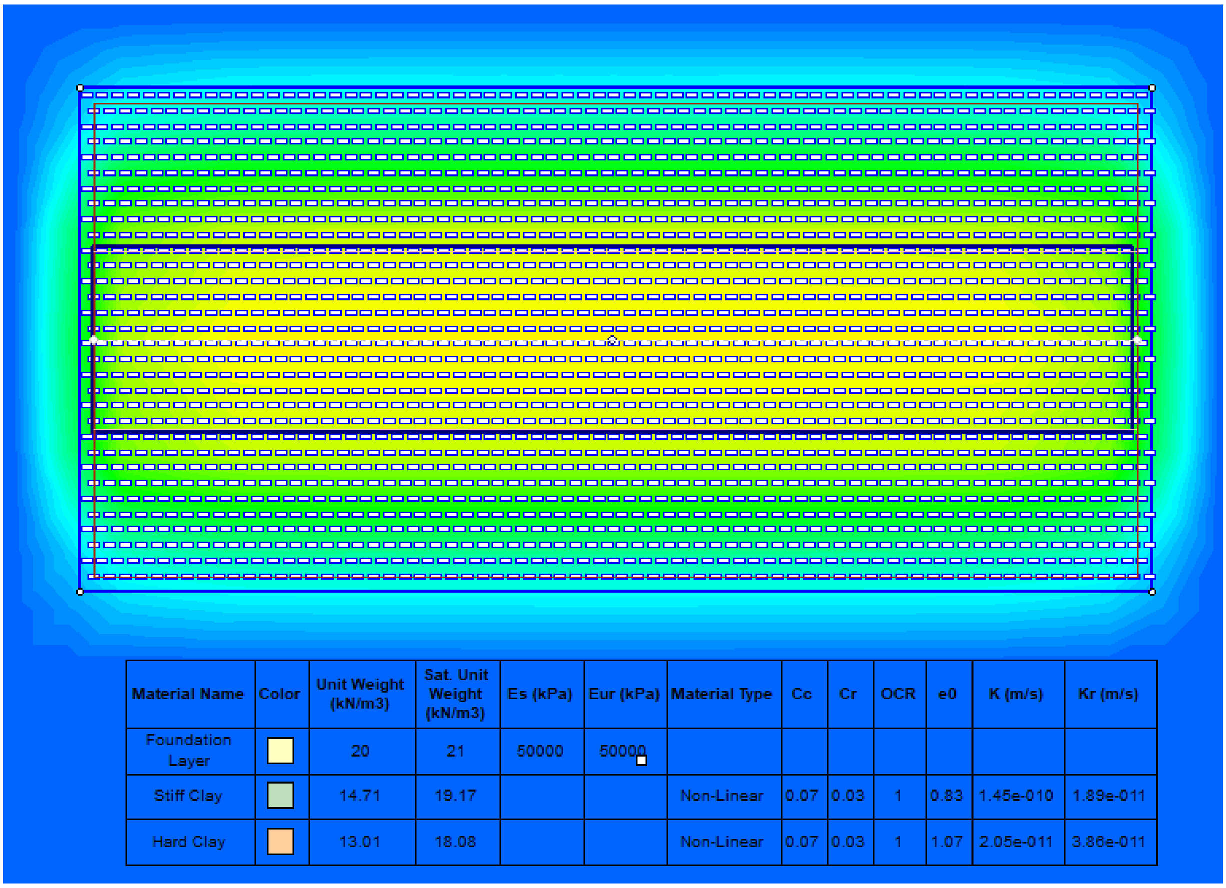
D. CALCULATION OF SETTLEMENTS

Due to the presence of fine-grained deposits, it can be assumed that the under-design embankment will suffer consolidation settlements due to the presence of claye deposits. In addition, it is understood that since the Clayey ground presents very low permeability, the rates of the consolidation settlements will be very low. Which means that the consolidation settlements will occur during the operation period. In order to avoid the deferential settlements and the relative failures in the transmission of the Bridge to the embankment the design foresees specific counter measures.

As pointed out in Table 21 the ground consists of 8.5m of Stiff clay, which will suffer consolidation settlements. This formation overlays on hard brown silty Clay which will also suffer consolidation settlements. In addition, the embankment should be found on a 0.5m thick layer of compacted coarse grained, granular material of Class 6-B. In the analysis the material is characterized with 50MPa deformation modulus and unit weight of γ=20.6kN/m^3. In addition, a traffic load of 26kPa is being takin into construction after the construction of the pavement. It should be pointed out that the impact of this additional load is limited.

Settlements and preloading.	Immediate settlements (cm)	Consolidation settlements (cm)	Total settlements (cm)
Total Settlements without measures On 9 months of pre-loading	19.1	3.99	23.09
Total Settlements without measures	19.1	13.1	34.10
Total Settlements with Strip Drains on 9 months of pre-loading	19.1	20.2	41.20
Total Settlements with Strip Drains	19.1	25.2	46.20





Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Es (kPa)	Eur (kPa)	Material Type	Cc	Cr	OCR	e0	K (m/s)	Kr (m/s)
Foundation Layer		20	21	50000	50000							
Stiff Clay		14.71	19.17			Non-Linear	0.07	0.03	1	0.83	1.45e-010	1.89e-011
Hard Clay		13.01	18.08			Non-Linear	0.07	0.03	1	1.07	2.05e-011	3.86e-011

Table 25. Summary with the immediate and consolidation settlements with and without drains.

The results from the settlement calculation, carried out in Computer Program Settle3D v2.0 (RocScience Inc.), are presented in Table 25. More precisely at 9 months of preloading without counter measures the consolidation settlements reach 3.99cm from the total 13.1cm. This means that in long term the pavement is going to receive 13.1 - 3.99 =9.11 cm. In case that there was no bridge, this settlement would have been acceptable. However, considering the construction of the bridge, which is a rigid structure, the differential settlement on the pavement of the bridge and the pavement of the embankment in high. Therefore, counter measures are being foreseen in order to reduce these long-term settlements. More precisely, in order to reduce the long-term

settlements below the level of 5cm, Triangular Strip Drains in a grid of 1.0X1.0, with depth of 8m, are recommended. With the drains the total consolidation settlements at preloading period of 9 months are 20.2 cm from the total 25.2. This means that in long term the pavement is going to receive 25.2-20.2=5.0cm which is acceptable.
In addition to that a post construction creep of the embankment materials can be estimated based on empirical relationships and construction experience as follows:
-0.2%*Hi, for the upper 10m
-0.4%*Hi for height 10-20m from crest
-0.6%*Hi for height >20m from crest, where Hi the width of each layer.
Based on the above a post construction creep of 1.25cm is estimated.

E. CONSTRUCTION GUIDELINES

Prior to placing the foundation layer any topsoil should be stripped and the embankment foundation checked to ensure that all weak, weathered or otherwise unsuitable materials have been removed during excavation and that design formation level has been reached.
drain pattern is installed, with geosynthetic drain strips of 10cm width, 0.4cm thickness, 8m length and a drain spacing of 1.0m. The drains will be installed with pre-drilling.
The foundation (starter) layer is then placed with a thickness of 0.5m consisting of a Class 6-B compacted coarse granular material. This layer behaves also as a basal drainage blanket in conjunction with the vertical drain pattern.
On top of the foundation layer a stabilizing-separation geotextile wrapped under the starting layer should be placed with a typical tensile strength of 30kN/m. The embankment fill material should be of Class 6-I or 6-J and it should be compacted in layers of 0.30m thickness.
Resting Period of Nine months before the construction of the pavement is proposed.

F. INSTRUMENTATION

Because of the drainage installed and in order to certify the goals that should be reached (no excess pore pressures generated during construction) it is crucial to monitor the evolution of the settlements and of pore pressures via specifically installed instruments as follows (Table 26):
(a) Pore pressure monitoring devices (Pi).
Two instruments will be installed in order to measure the built up and subsequent dissipation of pore water pressures. The piezometer tip will be installed at a depth of 10.0m below foundation level at Ch. 0+300 and at a depth of 10.0m below foundation level at Ch.0+380.
(b) Instruments to measure the evolution of settlements (Li+Si)
Two types of instruments will be installed in order to measure the evolution of settlements both at depth and at the foundation level. The instruments will be comprised of a borehole extensometer or settlement gauges (LI) installed down to a depth of 20m below foundation and of a foundation settlement plate or digital hydrostatic profile gauge or vibrating wire settlement cell or other equivalent monitoring device (Si). In addition to that two settlement plates (Si) will be installed on top of the embankment when completed, in order to monitor the post construction creep settlement of the fill material.Details regarding the type of the monitoring devices will be proposed by the Constructor and will be approved by the Supervision.

Instrument	Location	Depth below foundation (m)
S1+L1+P1+S2	Ch.0+300	0.0+20.0+20.0-20.00
S3+L2+P2+S4	Ch.0+380	0.0+20.0+20.0-20.00

Table 26: Location and Depth of Instruments

The frequency of the measurements is as follow:

- All instruments will be installed and measured after the surface stripping of the top soil.
- All instruments will be measured before the placement of embankment material (1 measurement).
- All instruments will be measured once every two meters (1m) of placed embankment material (approximately 2 measurements).
- All instruments will be measured at the end of construction of the embankment stage (1 measurement).
- All instruments will be measured 0.5, 1, 3, 5, 9, 12 months after completion of the full embankment (6 measurements).

Further measurements will be decided if necessary after the evaluation of the measurements at the end of 270 days of preloading. Depending on the evolution of settlements final decisions will be taken for the feasibility of the construction of the pavement.

G. CONCLUSIONS

In the present report the IC VAZ S1 road approach embankments between Ch. 0+210 – 0+310 and 0+370 – 0+410, have been studied, based on the Final Design. Based on the results of the ground investigation, in this the area it was assumed that similar geological and geotechnical conditions prevail. Then, a representative embankment cross section was selected (Ch. 0+300) where the embankment reaches maximum height and both slope stability and settlement calculations were carried out.

Taking into account all available data, the earthworks layout is acceptable. Furthermore, a triangular wick drain pattern is proposed, comprising of geosynthetic drain strips of 10cm width, 0.4cm thickness, 8m length and a drain spacing of 1.0m, in order to reduce the preloading time for the foundation layers to be consolidated in an acceptable degree. Wick drains are suggested in the areas where the embankment height is >4.0m, i.e. from Ch. 0+210 – 0+310 and 0+370 – 0+410. A rest period before the construction of the pavement of minimum 270 days (9 months) is proposed.

4 GEOTECHNICAL DESIGN OF CUT SLOPES

4.1 MOTORWAY CUT SLOPE 1 FROM 1+980 -3+040

A. SUMMARY

The First Cut slope extends from Ch: 1+980 to 3+040. For the Designed of the Cut Slope, three boreholes and one trial pit were carried out.

A/A	NAME	DEPTH (m)
1	BH_L1_S1_S03	15.00
2	BH_L1_S1_S04	15.00
3	BH_L1_S1_S07	20.13
4	TR_L1_S1_03	2.70

Table 27: Geotechnical Investigations in the area of 1-st Cut Slope.

The Cut Slope Encounters Hard Clays to Mudstones from 1+980 to 2+140 where the material changes into dense Gravel to medium strong Conglomerates. Therefore, the stability will be checked in the most unfavourable cross section from 1+980 to 2+140 which is at 2+060, and at the most unfavourable cross section from 2+140 – 3+040 which is at 2+520. By means of most unfavourable cross section, there are the deepest – highest cuts.

CUT SLOPE 1 CHs: 1+980 - 3+040							
Analysis Summary							
Load Combination		**Specification**	**Required Factor of Safety**	Ch. 2+060		Ch. 2+520	
				Janbu Cor	Bishop	Janbu Cor	Bishop
FS1	Short Term Conditions	Eurocode 7	**1.40**	**9.37**	**7.85**	**4.07**	**3.54**
FS2	Long Term Conditions+ Earthquake +Water level (A)	Eurocode 7/8	**1.10**	**1.61**	**1.55**	**1.35**	**1.39**
FS3	Long Term Conditions +Water level (Y50)	Eurocode 7	**1.25**	**1.79**	**1.89**	**1.57**	**1.59**
FS4	Long term Conditions +Completely Dry conditions	Eurocode 7/8	**1.38**	**2.17**	**2.09**	**1.65**	**1.64**

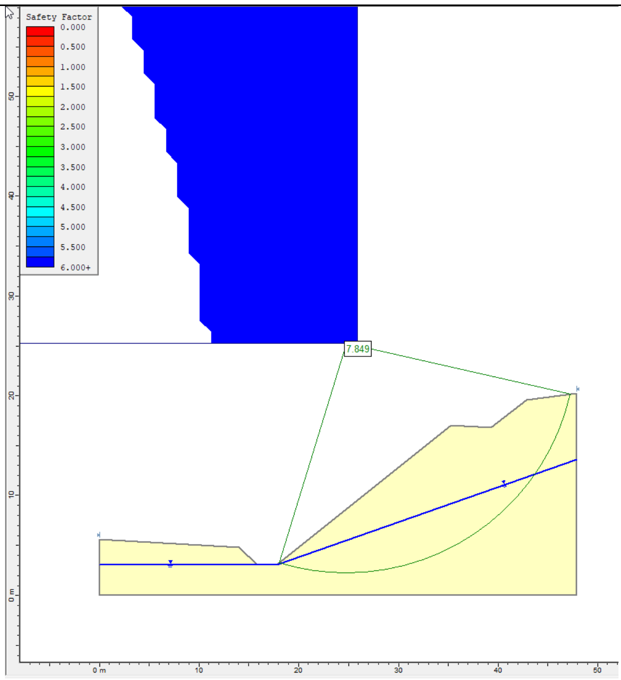
Table 28: Summary with the resulted Safety Factors fort he Cut Slope 1.

According with the design the Cut slope is to be excavated with inclination of 4/5, which based on the analysis table above, is acceptable.

B. FS1 ANALYSIS FOR 2+060

Unfavourable Location 1 Chainage 2+060

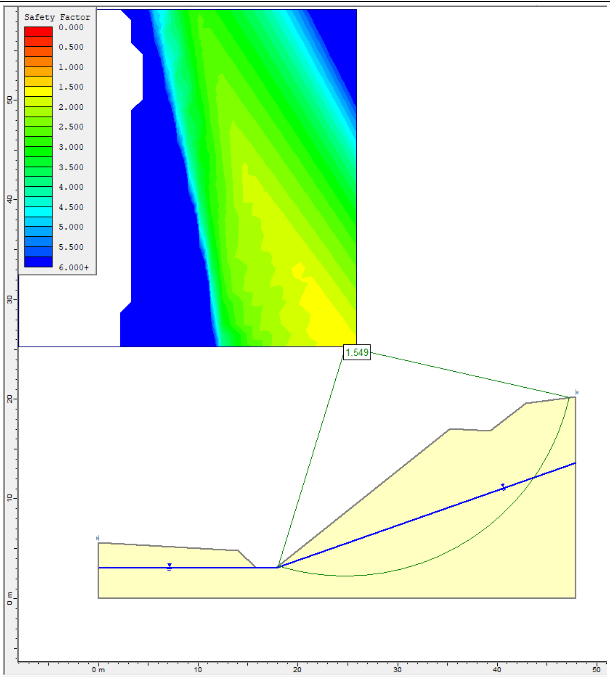
Load combination FS1 - Short Term Conditions



Cut Slope Data	
Height:	17m
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Hard, Brown Clay
γ (kN/m^3):	18.47
γ$_d$(kN/m^3):	13.54
Cu(kPa):	276.67
Water Level:	Yes
FS (Bishop):	7.85>1.4
FS (Janbu Cor):	9.37>1.4

C. FS2 ANALYSIS FOR 2+060

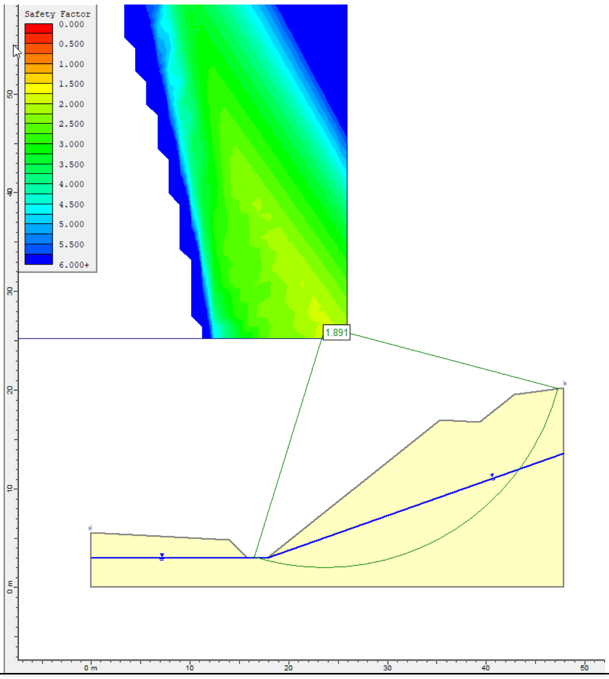
Load combination FS2 - Long Term Conditions+ Earthquake + Water level (A)



Cut Slope Data	
Height:	17m
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Hard, Brown Clay
γ (kN/m^3):	18.47
γ$_d$(kN/m^3):	13.54
C(kPa):	44.37
Φ(°):	16.94
Water Level:	yes
PGA	0.14
kh:	0.07
kv:	0.035
FS (Bishop):	1.55>1.1
FS (Janbu Cor):	1.61>1.1

D. FS3 ANALYSIS FOR 2+060

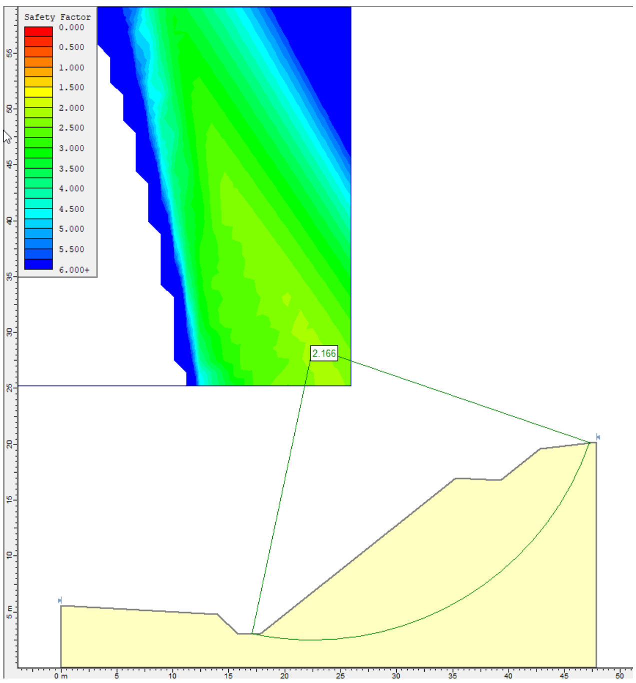
Load combination FS3 - Long Term Conditions +Water level (Y50)



Cut Slope Data	
Height:	17m
Inclination	4:5
Geological Conditions	
Total Layers:	1
Layer 1:	Hard, Brown Clay
γ (kN/m^3):	18.47
γ_d(kN/m^3):	13.54
C(kPa):	44.37
Φ(°):	16.94
Water Level:	Yes
FS (Bishop):	1.79>1.25
FS (Janbu Cor):	1.89>1.25

E. FS4 ANALYSIS FOR 2+060

Load combination FS4 - Long term Conditions +Completely Dry conditions

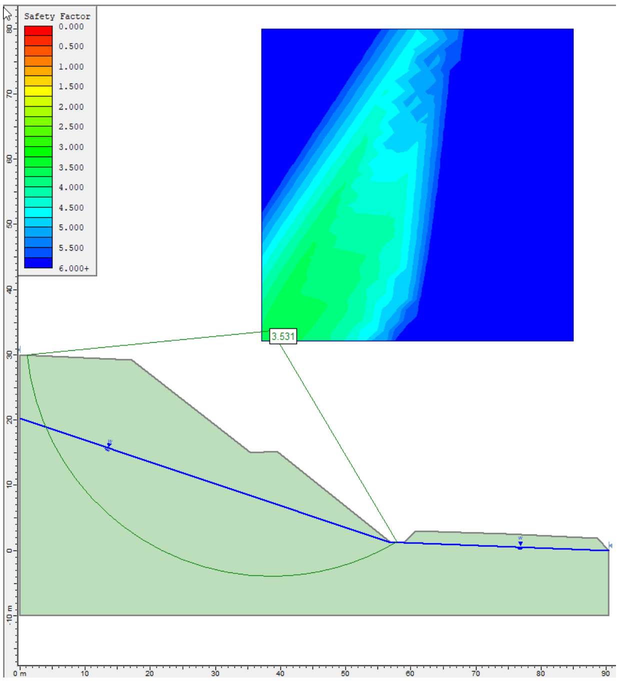


Cut Slope Data	
Height:	17.0
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Hard, Brown Clay
γ (kN/m^3):	18.47
γ_d(kN/m^3):	13.54
C(kPa):	44.37
Φ(°):	16.94
Water Level:	No
PGA	0.0
kh:	0.0
kv:	0.0
FS (Bishop):	2.09>1.38
FS (Janbu Cor):	2.17>1.38

F. FS1 ANALYSIS FOR 2+520

Unfavourable Location 2 Chainage 2+520

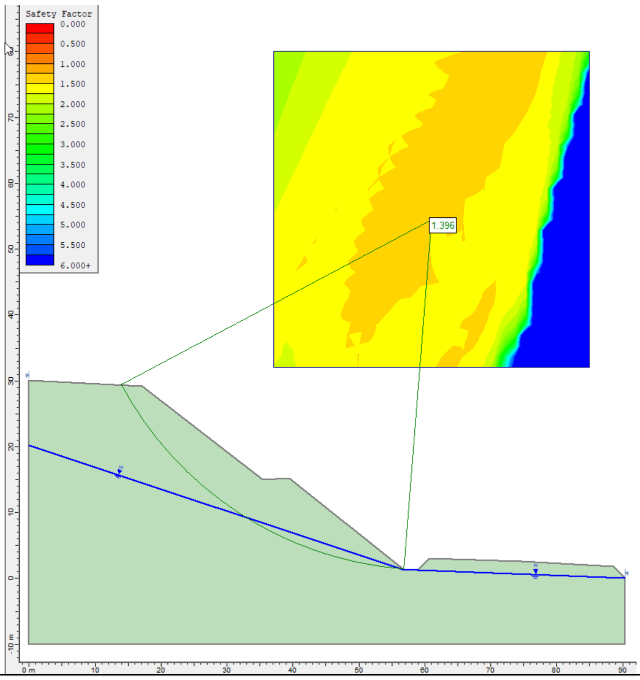
Load combination FS1 - Short Term Conditions



Cut Slope Data	
Height:	28m
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Very Dense Gravel
γ (kN/m^3):	21.05
$γ_d$(kN/m^3):	19.06
Cu(kPa):	300
Water Level:	Yes
FS (Bishop):	3.53>1.4
FS (Janbu Cor):	4.07>1.4

G. FS2 ANALYSIS FOR 2+520

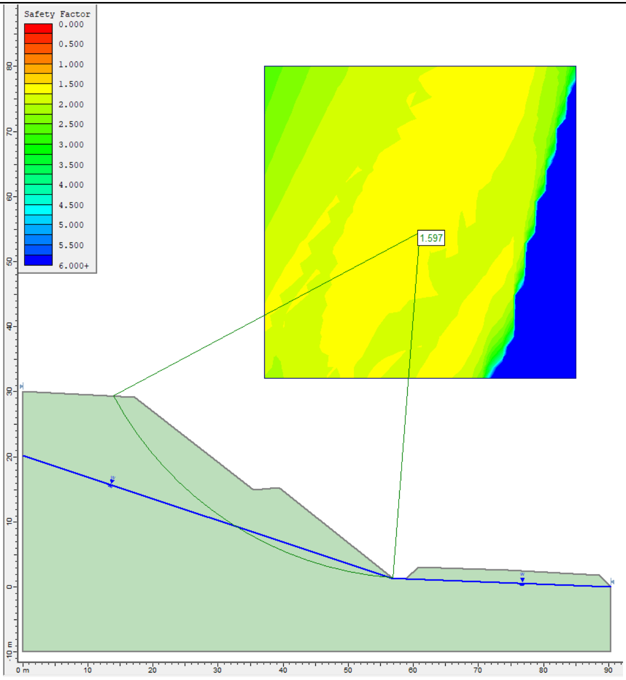
Load combination FS2 - Long Term Conditions+ Earthquake + Water level (A)



Cut Slope Data	
Height:	28m
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Very Dense Gravel
γ (kN/m^3):	21.05
$γ_d$(kN/m^3):	19.06
C(kPa):	10
Φ(°):	40
Water Level:	Yes
PGA	0.14
kh:	0.07
kv:	0.035
FS (Bishop):	1.39>1.1
FS (Janbu Cor):	1.35>1.1

H. FS3 ANALYSIS FOR 2+520

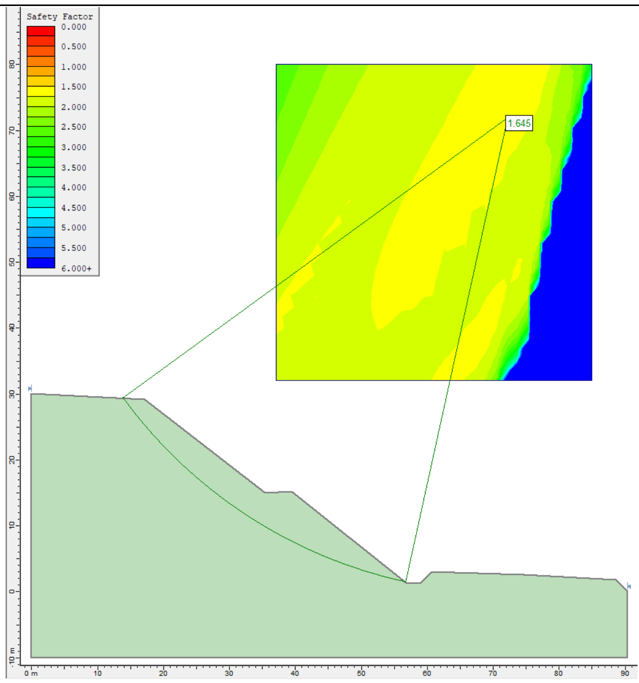
Load combination FS3 - Long Term Conditions +Water level (Y50)



Cut Slope Data	
Height:	28m
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Very Dense Gravel
γ (kN/m^3):	21.05
γ_d(kN/m^3):	19.06
C(kPa):	10
Φ(°):	40
Water Level:	Yes
FS (Bishop):	1.59>1.25
FS (Janbu Cor):	1.57>1.25

I. FS4 ANALYSIS FOR 2+520

Load combination FS4 - Long term Conditions +Completely Dry conditions



Cut Slope Data	
Height:	28m
Inclination	4/5
Geological Conditions	
Total Layers:	1
Layer 1:	Very Dense Gravel
γ (kN/m^3):	21.05
γ_d(kN/m^3):	19.06
C(kPa):	10
Φ(°):	40
Water Level:	No
FS (Bishop):	1.64>1.38
FS (Janbu Cor):	1.65>1.38