INSTYTUT KOLEJNICTWA	TECHNICAL STANDARDS DETAILED TECHNICAL CONDITIONS FOR THE CONSTRUCTION OF THE RAILWAY INFRASTRUCTURE OF THE SOLIDARITY TRANSPORT HUB – DESIGN GUIDELINES	CENTRALNY PORT KOMUNIKACYJNY – SOLIDARITY TRANSPORT HUB POLAND
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TECHNICAL STANDARDS
DETAILED TECHNICAL CONDITIONS FOR THE CONSTRUCTION OF THE RAILWAY INFRASTRUCTURE OF THE SOLIDARITY TRANSPORT HUB – DESIGN GUIDELINES

VOLUME I.5

RAILWAY TRACK – GEOTECHNICAL INVESTIGATIONS AND DESIGN

Version 1.3.0, closed on 5.08.2021

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The list of volumes constituting the detailed technical conditions for the construction of the railway infrastructure of the Solidarity Transport Hub:

Volume A	Introduction to the STH railway standards
Volume I.1	Railway track – layout geometry
Volume I.2	Railway – design of civil structures
Volume I.3	Railway track – drainage of track layout
Volume I.4	Railway track – gauge
Volume I.5	**Railway track – geotechnical investigations and design** The principles of acceptance of loads of earth structures, requirements concerning the subgrade and calculations of stability of the slopes of earth structures were specified.
Volume II.1	2 x 25 kV 50 Hz AC overhead catenary system and traction power supply
Volume II.2	3 kV DC overhead catenary system and traction power supply
Volume III.1	Engineering structures
Volume III.2	Tunnels
Volume IV	Non-OCL power engineering
Volume V.1	Non-public roads
Volume V.2	Public roads
Volume VI.1	Control command and signalling – basic equipment
Volume VI.2	Control command and signalling – European Train Control System (ETCS)
Volume VII.1	Fixed and wireless communication systems and data transmission
Volume VII.2	Telecommunication systems and telematics
Volume VII.3	Detection of rolling stock failure conditions (DSAT)
Volume VIII.1	Station and railway station buildings
Volume VIII.2	Technical buildings
Volume VIII.3	Structures
Volume VIII.4	Structural landscaping
Volume IX	Measures to minimise environmental impact
Volume X	Conflicts with external networks
Volume XI	Electromagnetic compatibility (EMC)
Volume XII	Railway line guard
Volume XIII	Technical support facilities
Volume XIV	Health and safety support systems for people and property
Volume XV	Survey control
Volume XVI	Railway rolling stock
Volume XVII	Automatic baggage check-in systems
Volume XVIII	Security, protection and cybersecurity integrity requirements

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Revisions of the document “Detailed technical conditions for the construction of railway infrastructure of the Solidarity Transport Hub; Volume I.5; RAILWAY TRACK – GEOTECHNICAL INVESTIGATIONS AND DESIGN”:

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Introduction

This volume I.5 of the Technical Standards – Design Guidelines is one of 30 volumes containing a description of detailed technical conditions for the construction of railway lines up to a speed of ≤ 350 km/h.

Technical scope

1

These guidelines apply to all categories of railway lines constructed by the Company. The guidelines should be applied for geotechnical calculations of earth structures: verification of the bearing capacity of the subsoil and the man-made and natural slope stability at all stages of designing earth structures.

1.1

Links to other volumes

The links between this volume of Standards with other volumes are presented in Table 1.

1.2

Table 1

Volume No	Volume title	Relation content
I.2	Railway track – Construction of civil structures	With regard to the structure and dimensions of the track substructure, protection of slopes and reinforcement of the subgrade of the earth structures
I.3	Railway track – Drainage of track layout	With regard to the requirements for slope and face drainage
IX	Measures to mitigate environmental impact	With regard to the impact of the excavations on the surroundings and neighbouring structures

Essential, basic and general requirements for the STH railway infrastructure

Table 2 defines the link between the detailed technical conditions and the essential, basic and general requirements for the STH infrastructure.

Table 2

2

sub-chapter of this volume defining detailed technical conditions	essential requirements (Directive on the interoperability of the rail system)						basic requirements	general requirements for the STH railway infrastructure			
	1.1. security	1.2. reliability and accessibility	1.3. health	1.4. environmental protection	1.5. technical compliance	1.6. accessibility	2.1. mechanical resistance and stability 2.2. safety in case of fire 2.3. hygiene, health and the environment 2.4. safety and accessibility in use 2.5. protection against noise 2.6. energy economy and heat retention 2.7. sustainable use of natural resources	3.1. focus on the needs of economy	3.2. orientation towards the needs of passengers	3.3. orientation towards the needs of carriers	3.4. compatibility with the railway infrastructure connected with the STH railway infrastructure
3.1	1.1.1. 1.1.3.	-	-	-	-	-	2.1.1. 2.7.1.	-	-	-	-
3.2	1.1.1. 1.1.3.	-	-	-	-	-	2.1.1. 2.7.1.	-	-	-	-
3.3	1.1.1. 1.1.3.	-	-	1.4.1-	-	-	2.1.1. 2.7.1.	-	-	-	-
3.4	1.1.1. 1.1.3.	-	-	1.4.1-	-	-	2.1.1. 2.7.1.	-	-	-	-

Cybersecurity

Technical solutions which collect, store, process, make available or transmit data ensuring the compliance with essential safety requirements (requirements from 1.1.1. to 1.1.11. specified in Volume A of the STH Railway Standards) and general requirements for the STH railway infrastructure concerning security (requirements 1.1.12. and 1.1.13 specified in Volume A of the STH Railway Standards) should be designed taking into account cybersecurity, i.e. "security of network and information systems", defined in the Directive concerning measures for a high common level of security of network and information systems across the Union, as follows:

> "security of network and information systems" means the ability of network and information systems to resist, at a given level of confidence, any action that compromises the availability, authenticity, integrity or confidentiality of stored or transmitted or processed data or the related services offered by, or accessible via, those network and information systems;
>
> *[as defined in Article 4 of Directive 2016/1148]*

Cybersecurity includes two types of threats resulting from unauthorised access to the systems/equipment/networks that collect, store, process, make available or transmit data:

1) physical security threats

It is necessary to secure systems/equipment/networks against direct access which could enable causing (intentionally or unintentionally) threats to functional safety.

2) IT security threats

It is necessary to secure systems/equipment/networks against logical access via IT systems/equipment/networks, which could enable causing (intentionally or unintentionally) threats to functional safety.

Cybersecurity defined this way applies both to information systems used for rail transport purposes and to operational systems used for rail transport purposes, but the STH railway standards do not include requirements for information systems, e.g. timetabling systems.

Physical security threats and IT security threats for operational systems for which requirements are defined in the STH railway standards should be addressed by railway operators as part of the risk assessment and by design engineers/manufacturers/contractors as part of threat control. Additionally, it is required for the applied protections to be documented and verified in accordance with the requirements included in Volume XVIII of the STH railway standards.

Cybersecurity within the scope of this volume of the STH railway standards

Currently, in the area covered by this volume of standards, there are no networks and information systems whose security could be endangered. However, it is possible that such networks and information systems or technical solutions that collect, store, process, make available or transmit data may arise. For example, a system of sensors may be used that, through wired or wireless networks, public or non-public networks or directly, will connect to, for instance, an infrastructure manager's system. Then, they should be protected against physical security and IT security threats in a manner compliant with the requirements of the Information Safety Management System (ISMS) implemented by the STH company.

At the same time, it should be kept in mind that the ISMS will be subject to changes because maintaining the required level of cybersecurity is not possible by meeting requirements of the standards once since cybersecurity is a process rather than a state. In order to minimise the number and size of cyber threats, the requirements (obligations) included in the Act of 5 July 2018 on the national cybersecurity system in Chapter 3 for operators of key services, in Chapter 5 for public entities should be continuously observed in operational processes and only digital service providers fulfilling the obligations described in Chapter 4 of that Act should be used.

3 Geotechnical design of earth structures

3.1 General requirements

In the scope covered by this standard, railway earth structures should be designed based on PN-EN 1990:2002, PN-EN 1991-2:2007 and PN-EN 1997-1:2008 standards. The requirements of the 1988 Regulation of the Minister of Transport and Maritime Economy**Błąd! Nie można odnaleźć źródła odwołania.** as amended and the 2012 Regulation of the Minister of Transport, Construction and Maritime Economy**Błąd! Nie można odnaleźć źródła odwołania.** should by applied.

The structure and dimensions of the structural elements should be assumed in accordance with the principles included in Volume I.2 of the Standards.

The basis for the geotechnical design are the ground investigations performed in accordance with the Guidelines **Błąd! Nie można odnaleźć źródła odwołania.** of 2021.

The provisions of the Standard apply to all design stages. In practice, they are used for preparation of the building permit design/technical design and detailed design, in exceptional cases for preparation of the Feasibility Study (FS).

<u>The FS, the Programme and Spatial Concept (PSC)</u> – at the FS stage, the Soil Survey Study and the Hydrogeological documentation are available for 3-4 options, at the stage of the PSC – the Soil Survey Report (SSR) for the selected Investor option.

Geotechnical design includes:

- guidance on the choice of route option and general technical solutions,
- identification of problematic sections with complex soil conditions, locations of possible adverse phenomena in the subsoil.

<u>Building Permit Design</u> – full investigation of the subsoil should be available: Geotechnical Report and Geotechnical Investigation Report, including investigations determined with the participation of the design engineer, Soil Survey Report, if necessary, and optionally Addenda to the Soil Survey Report and Hydrogeological documentation for construction drainage using boreholes (optional). Based on them, a geotechnical design report of earth structures is prepared.

<u>The geotechnical design report</u>, in accordance with PN-EN 1997 and the Regulation **Błąd! Nie można odnaleźć źródła odwołania.**, should include:

- the assumptions made, including partial safety factors taken into account in calculations, calculation methods and results (including an extract from the calculations) of the analysis of stability, serviceability and bearing capacity limits,
- a computational model (in simple cases, geotechnical profile) of the soil subgrade, design values of geotechnical parameters of soils and rocks,
- determination of necessary tests for quality assurance,
- calculations of bearing capacity and settlement as well as overall stability of the subgrade for representative cross-sections,
- geotechnical design calculations and drawings and adopted solutions
- limits of permissible behaviour of the structure, guidelines for control and monitoring as well as procedures for response to measurement results and recommendations for intervention actions.

In the scope of the earth structure, it should specify the structural and material solutions of the structure for representative types of geotechnical conditions based on calculations of the structure limit states. The design should include, where necessary, the following components:

- preliminary determination of the method of construction, preparatory works, milestones, necessity of special treatments, impact of works on the environment.
- verification of the ultimate limit state of components: the embankment subbase and its designed reinforcement (if necessary).

- checking the stability of the embankment slopes and the cutting, as well as the need for reinforcing actions or structures.
- verification of the serviceability limit state of components: settlement of the embankment (average, differential, progress in time), settlement or uplift and, possibly, the buoyancy of the structure in the cutting, if necessary – deformation of the cutting slopes.
- determination of methods of protection of slopes, in particular those exposed to water,
- determination of the scope of geotechnical monitoring.

The geotechnical detailed design should include:

- supplementary geotechnical investigation of doubtful places and resulting from the specificity of the construction method in the form of supplementary ground investigation documentation or test reports,
- detailed structural and material solutions for structure components,
- detailed subsoil reinforcement design (if planned),
- method of construction of geotechnical structures,
- detailed specifications for all elements of the structure and drainage,
- detailed work control program and monitoring plan during construction and operation of the structure, as well as procedures of response to the performed measurements and plan of intervention activities.

3.2 Classification of actions

Actions should be determined based on PN-EN 1990: 2002, PN-EN 1991-2:2007, and PN-EN 1997-1:2008 standards, and the 1988 Regulation of the Minister of Transport and Maritime Economy. Recommendations in prEN 1997-1:202x Appendix F can be used.

A railway earth structure is subject to the following impacts:

- static loads – track superstructure and other infrastructure elements in accordance with PN-EN 1991-2 point: 6.7.3. (1)P, 6.1(6), 6.2(2)
- geotechnical and environmental impacts;
- variable loads from rolling stock;

3.2.1 Permanent actions

Static loads of the track superstructure include the weight of the track grid and ballast (in the case of ballasted track) or the weight of the rails, fastenings, prefabricated elements and concrete slab (in the case of ballastless track) and other infrastructure elements.

The load to the track grid depends on the profile of the rails and the type of ties with fastenings. The load of the track grid can be assumed at 7 kN/m. The weight of ballastless track should be calculated individually, depending on the designed superstructure system.

The ballast load depends on its cross-sectional area and type. It is recommended to take its load into account by modelling it as other structural layers of the embankment. Ballast unit weight can be assumed to be 26 kN/m^3.

3.2.2 Geotechnical actions

The impacts should be assumed in accordance with PN-EN 1997-1, point 2.4.2. In particular, the following elements have to be taken into account:

- load from the track substructure (embankment fill) depending on its height and the type of soil and its parameters;
- subsoil unloading caused by excavation,

impact of groundwater (piezometric levels, their differences and changes in levels, flows in the ground).

The load from the track substructure (embankment fill) depends on its height and type of soil and the presence of groundwater (piezometric levels, their differences and changes in levels, flows in the ground).

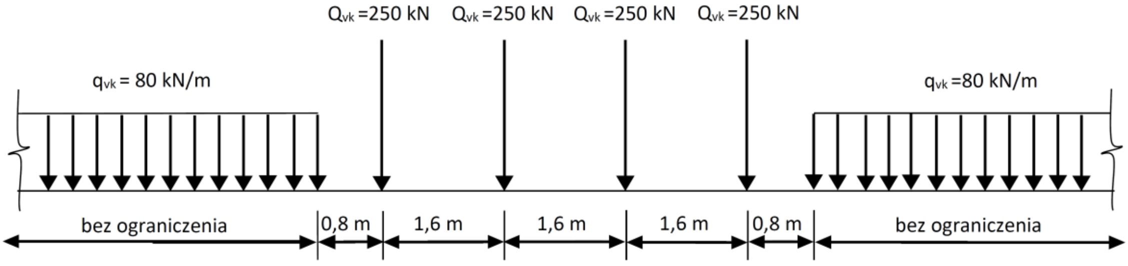
If the embankment acts (continuously or periodically) as a dam, it is subjected to lateral water pressure and the run-off pressure of water filtrating through the embankment.

In special situations, e.g. in the case of excavations below the piezometric level of groundwater, uplift by water may occur.

Impact of rolling stock

EN 1991-2 defines the equivalent load model LM71 for railway structures (Fig. 1).

3.2.3

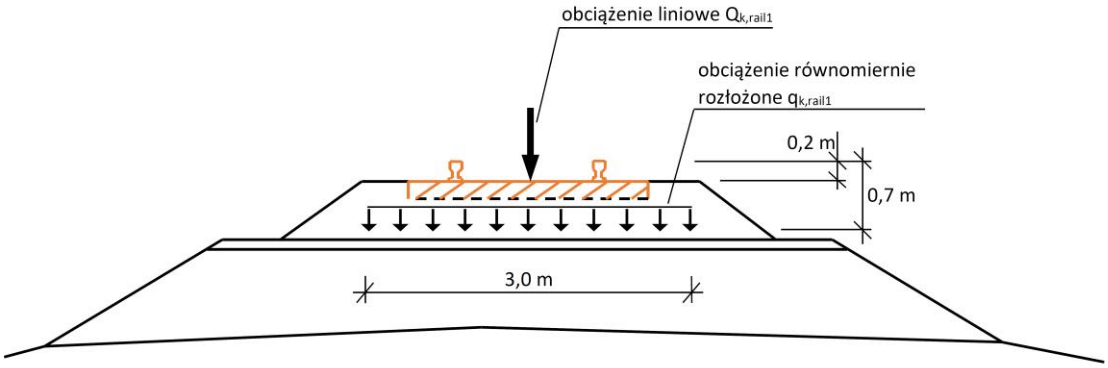


PL	EN
bez ograniczenia	without restriction

Figure 1. Load Model LM71 and characteristic values of vertical loads

(Fig. 6.1 acc. to [EN 1991-2])

In order to adapt the general model to the category of railway line, a classification factor should be used. The characteristic values given in figure A must be multiplied by the α coefficient. The load multiplied by α is called the "classified vertical load" or "characteristic load". The α coefficient value is given in EN1991-2 XX or specified in national regulations. α = 1.21 is assumed for the main and primary lines in accordance with the Regulation **Błąd! Nie można odnaleźć źródła odwołania.**.



PL	EN
obciążenie liniowe $Q_{k,rail1}$	linear load $Q_{k, rail1}$
obciążenie równomiernie rozłożone $q_{k,rail1}$	uniformly distributed load $q_{k, rail1}$

Figure 2. Equivalent load of rolling stock: linear loads on sleepers or evenly distributed at a depth of 0.70 m below the rail head (as per Appendix F to prEN 1997-1:202x **Błąd! Nie można odnaleźć źródła odwołania.**)

In addition to EN 1991-2 **Błąd! Nie można odnaleźć źródła odwołania.** in the load model LM71 of embankments and slopes, in accordance with Appendix F to prEN 1997-1:202x **Błąd! Nie można odnaleźć źródła odwołania.**, point loads may be replaced by characteristic uniformly distributed load $q_{k,rail1}$ = 52 kN/m ^{2} (at α = 1.0) with a width of 3.0 m and a length of 6.4 m, at a level of 0.7 m below the track level (rail head) (Fig. 2) or (plane strain conditions) by infinite linear load $Q_{k,rail1}$ = 100 kN/m.

The $q_{k,rail1}$ and $Q_{k,rail1}$ values should be multiplied by α = 1.21. Therefore, $q_{k,rail1}$ = 62.9 kN/m^2 or $Q_{k,rail1}$ = 121 kN/m.

A dynamic factor is not required for this evenly distributed load.

When verifying the limit states of earth structures according to PN-EN 1997-1, partial factors γ are adopted for impacts listed in Table 3 according to the National Appendix PN-EN 1997-1:2008 AP 2:2010P **Błąd! Nie można odnaleźć źródła odwołania.**.

Table 3. Partial factors γ for impacts

Type of impact	Ultimate limit states A1	Overall stability A2
Permanent unfavourable $\gamma_{G;dstb}$ Permanent favourable $\gamma_{G;STB}$	1.35 1.0	1.0 1.0
Variable unfavourable $\gamma_{Q;dstb}$	1.5	1.3

For verification of the UPL (uplift limit state), partial load factors (y_F) from Table A.15 of PN-EN 1997-1:2008 **Błąd! Nie można odnaleźć źródła odwołania.** should be used.

3.3 Subgrade requirements

3.3.1 General principles

The basis for the verification are the results of the geotechnical investigations determining the geological model of the subgrade and the geotechnical parameters determined based on the results of the investigations, directly or by means of correlations, theory or experience (derived values). The values of the parameters obtained from the investigations as well as other data should be interpreted in accordance with the considered limit state.

The design engineer defines characteristic values of geotechnical parameters as a cautious estimation of the value determining the occurrence of the limit state. On their basis, a geotechnical model of the subgrade and a computational model are created.

The design values of the geotechnical parameters (X_d) should be derived from the characteristic values using the following formula:

$$X_d = X_k/\gamma_M \qquad \text{(G1)}$$

or evaluated directly (as expert values).

In the formula (G1) for permanent and transient situations, partial factors γ_M specified in Annex A of the PN-EN 1997-1:2008 standard should be used **Błąd! Nie można odnaleźć źródła odwołania.**. The values of these coefficients according to the type of parameters and the limit state in question are presented in Table 4.

Table 4. Partial factors γ for geotechnical parameters and soil resistance GEO

Soil properties and resistance	Ultimate limit states	Overall stability

	M1	R2	M2	R3
Internal friction angle tanφ	1.0		1.25	
Effective cohesion c'	1.0		1.25	
Undrained shear strength c_u	1.0		1.4	
Uniaxial compression strength q_u	1.0		1.4	
Unit weight	1.0		1.0	
Shallow foundations – uplift – sliding		1.4 1.1		1.0
Retaining walls – uplift – sliding resistance – limit resistance		1.4 1.1 1.4		
Slopes – limit resistance				1.0

Preparation of the embankment/cutting subgrade

Preparation of the site for construction of an embankment includes:

3.3.2 removal of trees, shrubs, vegetation, elements of water drainage, fragments of structures and other obstacles,

1. removal or remediation of chemically contaminated materials and soils/rocks.

2. Grubbing-up and removal of topsoil is usually performed to a depth of at least 30 cm below the original level, at the width of the earth structure increased by 2 m on each side. Holes and cavities after grubbing-up should be filled with material used for embankments and compacted as embankment.

Earth materials from the subgrade of the embankment or excavation which are considered unsuitable:

1. organic soils (e.g. topsoil, peat),
2. man-made fill materials containing waste and impurities,
3. contaminated materials containing substances toxic to the environment and groundwater,
4. swelling soils, showing large volume changes with increased moisture content,
5. permeable soils sensitive to scour or suffosion (e.g. very fine sands of uniform grain distribution, dune sands, low-plasticity silts),
6. collapsible soils, soluble soils (e.g. gypsum), etc.

should either be remove or replaced with a material meeting the requirements, or reinforced, improved and protected against scour, or other geotechnical solutions should be used.

3.3.3 The material of the upper layers of the cutting should meet the same requirements as the material for the upper layer of the embankment. If the native soil does not meet these requirements, it should be replaced or reinforced to a depth of at least 1 m.

Verification of the ultimate limit state of the subgrade

1. When checking overall stability of earth structures according to PN-EN 1997-1, the main ultimate limit state verified is the GEO limit state which considers failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance.. In case of a structure reinforced with structural elements, such as sheet piling, anchors or piles, the strength of structural materials is important for ensuring resistance, and the STR state should also be checked. Additionally, in case of water buoyancy at the bottom of the excavation, the UPL state should be checked, and in case of pore water pressure of water flowing through the earth structure, also the HYD state should be checked.

Where relevant, it shall be verified that the following limit states are not exceeded::

failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO)' loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions (UPL);

hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD)

The bearing capacity GEO of the subgrade is checked similarly to the flexible foundation, applying the requirements of PN-EN 1997-1:2008 point 2.4.7 and 6.5.2. If organic or cohesive soils and groundwater are present in the subgrade, calculations should be carried out for both drained and undrained conditions. If the GEO bearing capacity condition turns out not to be met, the subgrade
2. should be reinforced using the solutions described in Volume I.2 of the Standards.

3. Verification of the subgrade stability against water pressure – UPL uplift is performed in accordance with PN-EN 1997-1:2008 point 2.4.7.4 and 10.2.

Verification of the vertical equilibrium (HYD) of the ground column in pore water pressure conditions should be performed using the requirements of PN-EN 1997-1:2008 point 2.4.7.5 and the formula (2.9b).

Construction of the embankment or the cutting should not cause reactivation of the existing landslides or creation of new ones in the vicinity of the embankment/cutting. The embankment and the cutting should be protected against scour if it is exposed to a direct action of flowing waters.

Verification of the serviceability limit state (SLS) of the subgrade

The design of the earth structure should ensure that the settlement and displacement that occurs after
3.3.4 its handover for use can be corrected using typical superstructure maintenance operations.

In accordance with PN-EN 1997-1:2008, serviceability limit states in the ground should be checked by means of the inequality:

$$E_d \leq C_d \qquad \text{(G2)}$$

where E_d – design value of the earth structure action on the subgrade;

C_d – design limit of the action effect.

The values of partial factors for serviceability limit states are generally recommended to be equal to 1.0.

The characteristic values of geotechnical parameters should consider and address any variation of soil/rock materials properties variability during operation period, for instance due to fluctuations of groundwater level, drying and wetting cycles, etc..

1.

Limit values of displacements

In the geotechnical design, the limit values of subgrade displacements should be determined in accordance with the guidelines specified below. Uneven displacements of the subgrade causing
2. deformations of the foundation structure should be limited so that no limit state occurs in the structure. Potential differences in settlement at the interface with engineering structures should also be taken into account in the calculations. The embankment structure should be made with allowances so as to reach the designed dimensions after structural settlement.

The basic requirement for the track substructure is to guarantee acceptable post-construction
3. settlement, i.e. trackbed settlement occurring from the moment the superstructure is installed until the end of its anticipated design life. Excessive total settlement or differential settlement during operation period may result in the necessity to adjust the track position by increasing the thickness of the ballast layer, which does not contribute to the stability of the track position. In the case of a ballastless track, it may prevent the adjustment of the vertical position of the track or cause damage to its structure.

Post-construction settlement S_R occurring after the track is handed over for use is the sum of the following settlement of the embankment and subgrade:

S_n – embankment self-weight settlement,

S_p – settlement of the subsoil due to embankment loading,

S_V – settlement from dynamic loads due to traffic load.

Allowable post-construction total settlement of the trackbed S_R should be assumed as follows:

a. 30 mm in the case of a ballasted track,
b. 15 mm in the case of a ballastless track (20 mm with possible track position correction).

Unless other requirements are specified, differential settlement should not exceed::

a. for total post-construction settlement occurring from the superstructure installation:
   i. 30 mm over a length of 30 m and
   ii. 10 mm over a length of 10 m,
b. during operation:
   i. 10 mm/year over a length of 200 m and
4. ii. 4 mm/year over a length of 30 m.

Rules for calculation of earth structure settlement

Determination of subgrade deformation/stiffness parameters

Derivation of soil/rock stiffness is crucial for the reliability and accuracy of structure settlement
3.3.5 assessment. For preliminary calculations, stiffness parameters may be derived on the basis of static cone penetration (CPT), flat dilatometer (DMT) etc. Other tests may be used provided their suitability has been demonstrated through comparable experience.. In cases where anticipated settlement exceeds the limit values, deformation moduli should be determined based on supplementary field tests and/or laboratory tests of soil samples from layers showing the highest deformations.

The following geotechnical parameters are required to calculate the settlement of the subgrade:

- in general – M oedometric moduli (primary and secondary)
- for highly compressible soils undergoing consolidation settlements, which cannot be eliminated prior to embankment construction: compressibility index C_C and secondary compressibility index $C\alpha$.

Soil stiffness derivation should consider and address relevant strain and stress level within soil/rock materials during construction and operation period. For the purposes of the forecast, it may be assumed that settlement of subgrade made of non-cohesive soils occur immediately after the construction of the earth structure and silty soils – in a short period of time, practically before the railway track is laid. On the other hand, moderately and highly cohesive soils and organic soils show delayed settlement, it is therefore necessary to make an estimate based on the calculations of their development over time and the period of disappearance (stabilisation) of primary and secondary (creep) consolidation settlements.

Determination of subgrade settlement

Settlement calculations should be consistent with soil mechanics principles. Analytical methods are recommended during preliminary design. Any software used for geotechnical design needs to be verified against results of analytical methods:

- a method of summation of deformations of subgrade layers – similar to the one specified in the PN-B-03020:1981 standard,
- a simplified elastic method, useful for subgrade with a relatively homogeneous deformability moduli.

In the case of complex ground model and/or earthwork geometry, numerical methods may be incorporated into design analyses in order to assess short and long term deformations.

Calculation of the subgrade settlement under the embankment by summing up the deformations of the subgrade layers.

The vertical component of the stress on the subgrade from the load of the rolling stock and the superstructure (in kPa) is calculated using the following formula

$$\sigma_{zd1} = \eta \cdot \sigma_{zd\max} \qquad \text{(G3)}$$

where η - the distribution coefficient of stresses determined from the nomogram presented in Fig. 3

for Z = H, whereas H means the height of the embankment, m.

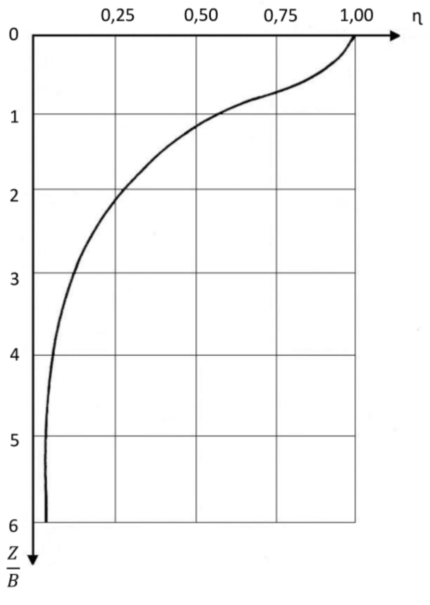


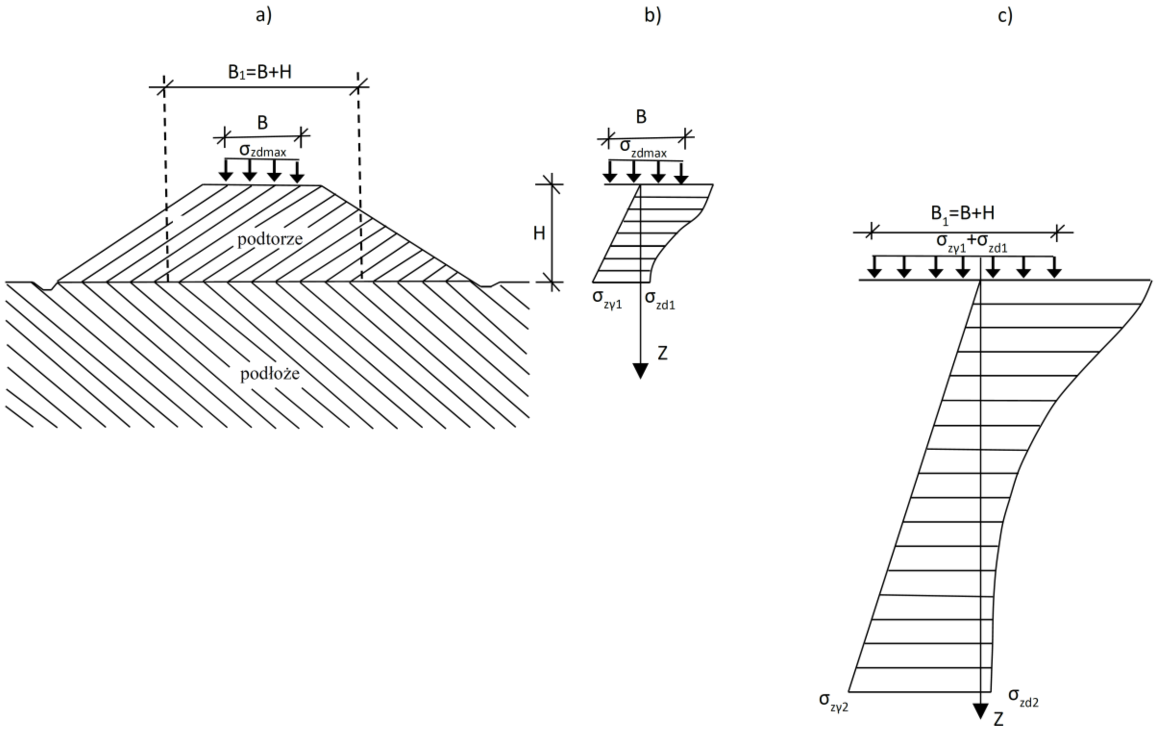
Figure 3. Nomogram for determination of coefficient η.

The stress on the subgrade from the embankment load (in kPa) is calculated using the following formula

$$\sigma_{z\gamma1} = \gamma_1 \times H \qquad \text{(G4)}$$

where γ_1 – the average unit weight of the embankment, kN/m^3.

The load on subgrade equal to the sum of the components of the vertical stresses $\sigma_{z\gamma1} + \sigma_{zD1}$ at the width of B1 = B + H (Fig. 4) and at the length of L = ∞ is assumed.



PL	EN
podtorze	track substructure
podłoże	subgrade

Figure 4. Stresses in the subgrade under the embankment: a) trackbed load, b) components of vertical stresses in the track substructure, c) components of vertical stresses in the subgrade.

The vertical component of the stress in subgrade σ_{zD2} from the load $\sigma_{z\gamma 1}$ + σ_{zD1} is then calculated using the nomogram given in Figure 3 and the vertical component of the stress from the self-weight of the subgrade σ_{z2} is calculated. The settlement of the subgrade (in cm) is calculated as the sum of settlement of individual geotechnical layers using the formula (G5):

$$S = \sum_{i=1}^{n} \frac{\sigma_{zd2i} \cdot h_i}{M_{oi}} \qquad \text{(G5)}$$

where:

σ_{zd2i} – vertical component of stress in the middle of geotechnical layer i, kPa,

h_i – thickness of geotechnical layer i, cm,

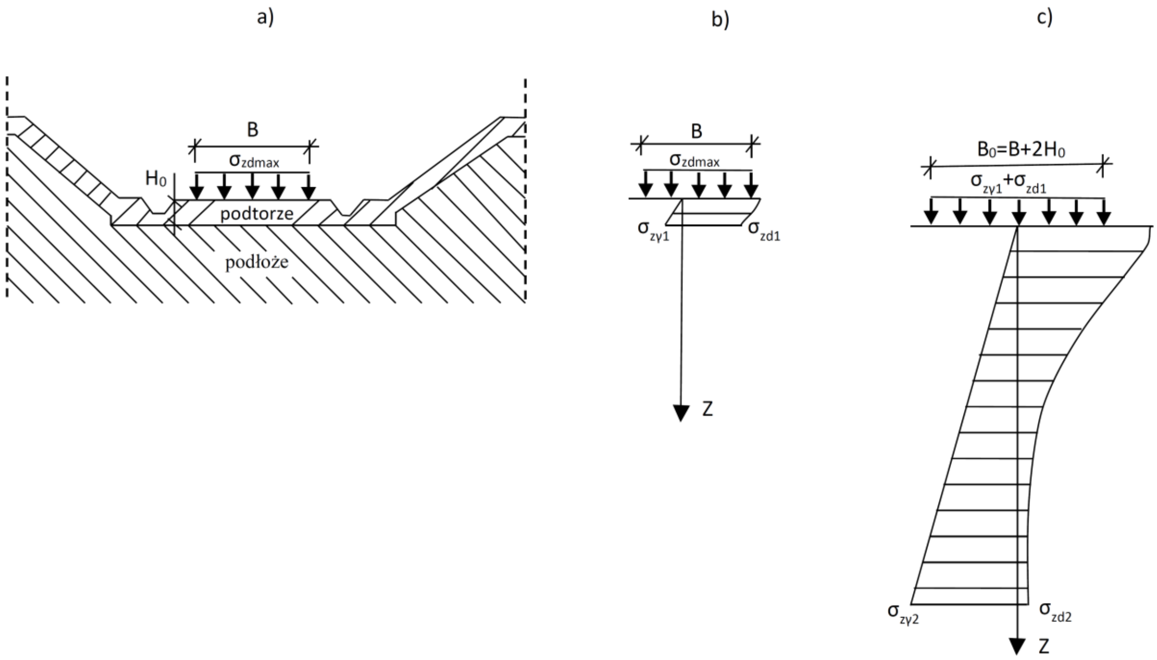
M_{oi} – oedometric primary compressibility modulus of the original geotechnical layer i, kPa.

Settlement is calculated for geotechnical layers to the depth at which the condition is met

$$\sigma_{zd2} < 0{,}2\ \sigma_{z\gamma 2}$$

If the structure of layers varies across the embankment cross-section, settlement should also be calculated on the edges of the embankment crown.

Calculation of subgrade settlement in the cutting. As the subgrade of the cutting is unloaded as a result of removal of the soil, settlement is small or track substructure heave may occur. The vertical components of stresses $\sigma_{z\gamma 1}$ andσ_{zd1} are calculated similarly to the subgrade under the embankment, assuming Z = H_0, where H_0 is the thickness of the protective layer. The load on subgrade equal to the sum of the components of the vertical stresses $\sigma_{z\gamma 1}$ + σ_{zD1} at the width of B_1 = B + 2H_0 (Fig. 5) and at the length of L = ∞ is assumed.



PL	EN
podtorze	track substructure
podłoże	subgrade

Figure 5. Subgrade stresses in the cutting: a) trackbed load, b) stresses in the track substructure, C) stresses in the subgrade.

The stress in subgrade σ_{zd2} and $\sigma_{z\gamma2}$ and the settlement of the subgrade are calculated similarly as in the case of the subgrade under the embankment.

3.3.6 Control tests and monitoring of earth structures

The Geotechnical Design and the Detailed Design should provide for control tests and monitoring of their condition and behaviour, according to the level of detail of the stage. It should specify, where appropriate, the scope of the tests, the objective of the monitoring, the values to be measured, the expected range of changes to the measured values and the alert, action and alarm levels. The detailed design of the monitoring defines the measurement methods and equipment and its arrangement, principles of documenting the measurement results, control and notification, requirements for operation of the measurements. The design and monitoring system should be compliant with PN-EN ISO 18674-1, PN-EN ISO 18674-2, PN-EN ISO 18674-3, PN-EN ISO 18674-4 and PN-EN ISO 18674-5.

Control tests during the execution of earth structures, in accordance with PN-EN 1997-1:2009, include, if necessary:

- verification of conditions in the subgrade: determination of the type and geotechnical properties of soils and rocks of the subgrade, in particular a description of soils and rocks exposed in excavations, using the methodology according to the guidelines of the Polish Geological Institute**Błąd! Nie można odnaleźć źródła odwołania.**;
- taking representative samples and determining the soil index properties;
- determination of the subgrade strength by measuring the value of the secondary deformation modulus E_2 using a plate load test under static pressure according to PN-S-02205:1998 Annex B **Błąd! Nie można odnaleźć źródła odwołania.**, using: inspection tests with light dynamic plate provided that the results are earlier calibrated with the static plate test, in cases of doubt by examining the indication of compaction I_S according to PN-S-02205:1998,
- in case of subgrade layers bound with cement or other binder – determination of thickness and compressive strength of the layer.
- measurements of groundwater levels, groundwater pressures and, if necessary, chemical composition of groundwater – the results should be compared with the values adopted in the design.

The designer should be notified immediately about any deviations from the type and properties of the ground adopted in the design. If it is necessary to deviate from the method of execution of a structure adopted in the design and specified in the geotechnical design (resulting e.g. from different soil conditions), this should be justified and agreed with the design engineer and/or the supervision personnel.

If hazardous materials are discovered during the works, the supervision personnel should be notified immediately. The necessary measures should be taken to safely remove and dispose of these materials.

Monitoring during the execution of earth structures includes, if necessary:

– measurements of settlement and horizontal displacements of the subgrade (e.g. using inclinometers) of embankments on weak and doubtful subgrade, observation of phenomena that may indicate a risk of loss of stability,

– observations of changes in groundwater levels,

– execution and correct operation of drainage systems.

During the operation of earth structures, the monitoring includes, if necessary:

– observation of the technical condition,

– measurements of displacements of the trackbed and man-made and natural slopes posing a threat to the safety and use of railway infrastructure,

– in mining areas, karst valleys, landslides and other areas with possible sudden movements: current analysis of the results of deformation measurements, planning of measurements and observations in critical zones and hazard signalling systems similar to point 3.5.2 (VOLUME XIV HEALTH AND SAFETY SUPPORT SYSTEMS FOR PERSONS AND PROPERTY).

Comprehensive data on the monitoring methods for road structures is included in the publication **Błąd! Nie można odnaleźć źródła odwołania.**.

3.4 Rules for slope stability calculation

Stability of man-made and natural slopes in railway earth structures should be verified according to the PN-EN 1997-1:2008 standard. Details of the calculation methodology are provided in the ITB Instructions (2011) **Błąd! Nie można odnaleźć źródła odwołania.**, in the PN-B-03010:1983 standard **Błąd! Nie można odnaleźć źródła odwołania.**, and in technical literature, e.g. Batog and Hawrysz (2012) **Błąd! Nie można odnaleźć źródła odwołania.**, Bogusz (2013) **Błąd! Nie można odnaleźć źródła odwołania.**, Bogusz and Godlewski (2019) **Błąd! Nie można odnaleźć źródła odwołania.**, Skrzyński (2017) **Błąd! Nie można odnaleźć źródła odwołania.**, Skrzyński (2020) **Błąd! Nie można odnaleźć źródła odwołania.**. Verified software should be used in the design only.

The stability check of man-made and natural slopes applies in principle to the ultimate limit state. If structures or utilities are located within the influence zone of the slope, their expected displacements (SLS) should also be assessed. The calculations should take into account the impact of the existing and planned structures.

Stability calculations are subject to a large margin of uncertainty. In cases of doubts or particular potential hazards resulting from loss of stability, a system for monitoring the movements of the slope and drainage operation should be provided.

3.4.1 Requirements for slopes of embankment and cuttings

The requirements for embankment slopes (track substructure) are specified in Volume I.2 of the Standards.

The stability of slopes with the height exceeding 6 m, and in any case in flood risk areas (possibility of temporary flooding of the slope), should be verified with calculations. If the stability conditions are not met, the slope inclination should be reduced, and if this is not possible, the slopes should be reinforced, e.g. with soil nails, rock anchors, shotcrete layer or rockfall protection netting, and in case of significant limitation of space – with retaining structures.

1. 2. 3.

Maximum embankments/cuttings slope gradient should consider the type of soil/rock and should not exceed:

soil and heavily cracked, weathered rock	1 : 2
slightly cracked rock	1 : 1
intact rock, uncracked	4 : 1.

In case of high embankments or deep cuttings of more than 10 m, it is recommended to use variable slope gradient or provide slope benches. High slopes should be divided with slope benches: every 10 m in soils and every 7 m of height in rocks. It is recommended to provide a bench width of 4 m, a cross slope (up to 5%) and a longitudinal slope of the surface to ensure water drainage.

Proper **drainage of slopes** should be provided, which is essential for their stability and durability. Slopes should be protected against erosion by water. For this purpose, an (upper) slope ditch should be provided to collect and discharge water flowing from the face above the excavation slope and a bottom slope ditch at the slope bottom, collecting and discharging water from the earth structure. The ditch should be reinforced and protected against scour. The slope ditch should be fenced. Details of solutions are specified in Volumes I.3 (drainage) and XII (minimising environmental impact) of the Standards and in the paper **Błąd! Nie można odnaleźć źródła odwołania.**.

If the slope crosses the aquifer and groundwater flows to the excavation slope, slope drains should be used to collect water and discharge it to the railway line drainage system. In case of slope saturation and abundant inflow of water, drainage may be needed to drain and reinforce the surface of the slope.

The surface of the slopes should be protected **with a vegetation cover** or other methods as necessary. Special protection of slopes should be provided in places where water may flow along the slopes. Requirements concerning slope protection are specified in Volume I.2 (construction of a civil structure).

3.4.2 Determination of strength parameters

The ground model, mechanical properties and geotechnical strength parameters shall be presented in Ground Investigation Report.

For slope stability analyses, a minimum scope of geotechnical parameters shall cover following parameters:

1. soil type,
2. unit weight,
3. non-cohesive soils: relative density I_D; effective angle of friction φ';
4. cohesive soils – long-term conditions effective: cohesion c' and friction angle φ';
   – short-term conditions undrained shear strength c_u,
5. rock faces: type and characteristics of rock, compressive strength, degree of weathering, nature and gradient of stratification or bedding, layout of joints,
6. special requirements: residual parameters of selected soil layers, strength of weak interbeddings; careful interpretation of results is recommended.

Special attention should be paid to the zone of contact of cohesive layers with non-cohesive fully saturated soils covering them. The surface of cohesive soil is moistened and constitutes a potential slip layer, therefore its strength parameters should be reduced. A similar situation occurs in layered rocks, e.g. flysch rocks, made of strong layers of sandstone and weaker mudstone or claystone. These layers are sensitive to water and, especially during unloading and stress relieving, their strength deteriorates.

Strength parameters should be reduced also in the case of the existing shear surfaces of landslides – residual values of parameters should be assumed there.

If FEM analyses are to be undertaken than depending on soil/rock constitutive model which is planned to be used appropriate geotechnical parameters will need to be derived accordingly.

Geotechnical parameters for numerical modelling will need to be determined based on suitable laboratory, in-situ tests results and comparable experience.

Geotechnical design shall consider groundwater levels and its fluctuations during construction period and long-term variations during design working life. Any adverse impact of groundwater conditions on soil and rock properties should be considered and appropriately addressed in the design, including existing slip surfaces, fissures, etc.

3.4.3 Slope stability criteria

Stability checks include:

1. slopes of embankments (track substructure) with subgrade,
2. slopes of cuttings,
3. natural slopes within the range of impact of the railway structure.

Slopes of new embankments (track substructure) made of controlled and properly compacted materials, with the required gradients and protected surfaces, are usually stable. Stability calculations are needed when an embankment is placed on a weak subgrade or when, for example, it loads down the existing slope, its construction requires the existing slope to be undercut, or if it can perform the role of a flood bank or similar, and water seepage through the embankment occurs.

Stability of cuttings, embankments and existing natural slopes should be verified by appropriate calculation method.

It should be noted that slope stability results are as certain as ground model and geotechnical parameters adopted in analyses only. Uncertainty in ground model, presence of joints, faults, fissures etc. is an inherent part of such analyses.

In accordance with PN-EN 1997-1:2008, the condition of the ultimate limit state (GEO and STR) is expressed by the inequality:

$$E_d \leq R_d \qquad \text{(G6)}$$

where: E_d – design value of the effect of actions destabilising the slope,

R_d – design value of soil resistance preventing loss of stability.

The F factor of safety is not directly checked in this case, and safety margins are introduced by comparing design values, i.e. increased actions and reduced resistance, by applying partial safety factors to actions γ_F and soil resistance γ_M. The resulting F factor in the calculation according to Eurocode 7 depends on the share of constant and variable actions: in the absence of variable actions, it amounts to 1.25 for effective parameters and 1.4 for total parameters (shear in undrained conditions). The overall factor of safety is increased by a partial factor to variable actions γ_F = 1.3 or 1.5 and a very cautious method of determining characteristic values of strength parameters according to Eurocode 7 (reduced rather than average values as in PN-B standards).

Computational model for the slope

3.4.4 In order to determine the computational model, the following should be determined:

1. geometry of the slope and adjacent area,
2. structures and utilities that load the slope and are located within the zone of influence,
3. layout and layering of soil,
4. parameters of geotechnical layers: characteristic and design values,
5. position and arrangement of the groundwater table, possible action of run-off pressure; in particular, the situation of sudden lowering of the water table, if possible, should be considered.

It is essential to define a representative geotechnical model presenting the layout of soil layers and their geotechnical parameters. Particular attention should be paid to the determination of the inclination of layers (generally horizontal or with inclination corresponding to the gradient of the slope or the opposite).

Details of the computational model used result from the selected stability verification method and the computer software used. Calculation methods are divided into conventional limit equilibrium methods (e.g. circular and non-circular slip surfaces, block or wedge method) and numerical methods – most often finite element methods (FEM).

Limit equilibrium methods are well known, relatively simple to use, and require a small number of parameters that can be obtained as a result of standard geotechnical investigations. The calculation is quick and the reliability is satisfactory. It is recommended to use them for preliminary analyses of numerous cross-sections and for routine verification of stability in sections of the route with similar conditions, using verified computer programs. Calculations should be made for conditions representative for the route section in question. However, these methods check the stability on the defined slip surfaces, which are not always the most unfavourable.

In case of weak interbeddings or privileged slip surfaces in the slope, calculations assuming circular slip surfaces are unreliable. In such a case, block methods should be used taking into account the slip of blocks or wedges on potential shear surfaces. However, in the case of heavily cracked rock masses, as well as soft rocks and cemented soils, failure on circular or similar surfaces can be assumed, assuming equivalent strength parameters of the mass.

Numerical FEM calculations are recommended for a detailed analysis of selected sections, for which the results of preliminary analysis are uncertain, as well as in special cases: complex soil conditions or slope shapes which cannot be reflected properly in typical analyses, as well as for verification of calculations using the conventional methods. The FEM methods are recommended in situations of significant water seepage to determine the distribution of water pressures in the ground, as well as for sizing of structural elements reinforcing the slope (e.g. soil nails, anchors, piles), taking into account safety margins. In this way, stability can also be calculated under spatial conditions in a 3D model.

The FEM calculations [17] provide extended analytical capabilities, but are more complicated and require a sound knowledge of this method. Preparation of input data and calculation model is complex, it is necessary to take into account more parameters, some of which are difficult to measure and are assumed based on experience. The results are strongly influenced by the distribution of the element grid, which should be more dense in the potential failure area, and the determination of the boundary conditions of the task. Calculation results are more difficult to interpret.

The FEM methodology of calculation is fundamentally different from that of the limit equilibrium methods. The direct result of the calculations is a map of soil movements. The limit state of the slope is achieved by reducing the soil strength parameters in subsequent steps. At the limit state, a clear failure surface is obtained. Two procedures are used:

1. the design (i.e. reduced) values of the parameters are assumed from the beginning of the analysis, which makes it possible to check directly the ultimate limit state of the overall stability; gradual reduction of shear strength (so-called "c-φ reduction") allows to determine the safety margin above the level resulting from the requirements of the standard;
2. the calculations are started assuming characteristic values of parameters and actions; for these data, calculations should be made at important construction stages and the serviceability limit state (allowable movements) is checked. Then, in order to check the ultimate limit state, the actions are increased and the strength parameters are reduced to the design values.

The factor of safety determined this way corresponds to the value of the factor by which the parameters were reduced in the last step preceding the loss of convergence of the computational process. The image of the potential slip surface can be obtained by comparing the movements obtained in the last two calculation steps. These movements (displacements) are not actual values, but they show the slip block area at the loss of stability.

FEM calculations require more parameters than in the limit equilibrium methods. It is recommended to assume characteristic values for parameters not related to the soil shear strength. Detailed guidelines are provided, among others, in the ITB Instructions of 2011 **Błąd! Nie można odnaleźć źródła odwołania.** and in publications, e.g. **Błąd! Nie można odnaleźć źródła odwołania. Błąd! Nie można odnaleźć źródła odwołania. Błąd! Nie można odnaleźć źródła odwołania. Błąd! Nie można odnaleźć źródła odwołania.**.

3.4.5 Rules for slope stability calculation

The scope of stability verification should include the design situations:

1. long-term (target) stability under fully drained conditions;
2. short-term stability under undrained conditions;
3. changes in stability over time, if any.

Long-term stability analysis under fully drained conditions

The basic condition is to check the long-term stability of the slope. Fully drained conditions mean that a change in groundwater pressure from the load applied reaches the state of equilibrium and there is no excess groundwater pressure. The fully drained state can be assumed in non-cohesive soils with a high filtration coefficient – sands and gravels. In poorly permeable soils, fully drained conditions occur after the completion of the consolidation process. Fully drained conditions are the most unfavourable situation in case of unloading of cohesive soils (e.g. as a result of excavation). In calculations for fully drained conditions, the effective parameters of soil strength (friction angle φ' and cohesion c') should be used, and the distribution of water pore pressure should correspond to the flow conditions in the determined filtration.

Short-term stability analysis in undrained conditions

Short-term stability shall be verified in low permeable mineral cohesive and organic soil when loaded.

This occurs when the load is applied within a time much shorter than the time needed for the pore pressure to return to the state of equilibrium in the consolidation process, e.g. during the construction of embankments on a weak subgrade or for variable loads from rolling stock traffic. The stability in undrained conditions is calculated for total stresses in the ground, assuming its undrained shear strength c_u.

Analysis of stability changes over time under subgrade consolidation conditions

If sufficient time is available, it is advisable to enable consolidation of weak subgrade under the weight of the embankment or additional surcharge embankment. As a result, water is removed from the pores of the soil which reduces its volume, increases its strength and generates a large proportion of soil deformations. This reduces delayed operational settlement of the embankment.

In such cases, it is advisable to perform an FEM analysis of porewater pressure changes in soil over time as a result of subgrade consolidation, which allows to assess settlement and changes in stability conditions over time. Such calculations are carried out for effective stresses and, if necessary, also for total values. The results so obtained provide the basis for a decision on the possibility of applying partial or complete consolidation, as well as on the need to execute vertical drains or perform other treatments to accelerate water drainage and subgrade settlement.

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4 Reference documents

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