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GUIDELINES FOR ROAD DESIGN, CONSTRUCTION, MAINTENANCE AND SUPERVISION

VOLUME I: DESIGNING

SECTION 1: ROAD DESIGNING

Part 7: ROAD STRUCTURAL ELEMENTS

Sarajevo/Banja Luka
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Chapter 1: EARTH WORKS

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1 EARTH WORKS

1.1 STABILITY OF SLOPES IN CUTS AND ON EMBANKMENTS

1.1.1 Subject of specification

The guidelines are valid for the structures of geotechnical categories 1 and 2. For geotechnical category 3 the guidelines present only minimum scope and the basic guidance in the design. With category 3 frequently other procedures and methods will be needed, and the cooperation of specialists will be required.

Providing stability for the slopes in cuts and on embankments assures safety against soil or rocks sliding down the slope.

Consequences of slope instability can be:

- loss of global ground stability and stability of nearby structures,
- excessive ground displacements due to shear strains, settlements, vibrations and ground elevations
- damages or reduced applicability of the nearby structures, roads and infrastructure due to ground movements.

Embankments are mainly built from quality materials built-in under constant control. These materials are generally not saturated with water, thus stability assurance of the embankment slope itself depends on the choice of the adequate slope gradient. Stability of the ground, loaded by embankment, is not the subject of this Section, but is described in Section 1.1.7.1.2

Slopes of cuts can be carried out in the soil (coherent or incoherent) or in the rock. Due to different nature of these two earth materials, the slope stability in each of these materials is treated by different methods.

At the slopes in cuts frequently also underground water appears, decreasing global stability and causing also surface or internal soil erosion.

The stability of slopes in cuts is due to natural diversity of the conditions that exist in the area where cuts are constructed, much more demanding, which is why this Section mainly deals with this subject.

When dealing with slope stability, comparable experiences need to be taken into account.

1.1.2 Symbols applied

c	cohesion
c'	cohesion expressed by effective stresses
c_u	undrained shear strength
h	height (of the embankment, excavation)
k	water permeability coefficient
N	normal force at the slip surface
T_c	resulting cohesion resistance along the slip surface
T_φ	resulting friction resistance along the slip surface
u	pressure of pore water
W	weight (of the soil)

Greek letters

β	slope gradient
γ	volume weight
γ_c, γ_φ	safety factors on cohesion and shear angle in drained conditions
γ_{cu}	safety factor in undrained conditions
θ	slip surface gradient
σ	normal total stress
σ'	normal effective stress

τ shear stress
φ' shear angle, expressed by effective stresses

Abbreviations

SMR Slope mass rating
RMR Rock mass rating
GSI Geological strength index
JRC Joint roughness coefficient
JCS Joint wall compressive strength

For the geotechnical calculations the following units and their multiples are recommended:

force	kN
mass	kg
moment	kNm
density	kg/m^3
volume weight	kN/m^3
stress, pressure, strength, rigidity	kPa
permeability coefficient	m/s
consolidation coefficient	m^2/s

Categories of structures according to Eurocode 7

Category 1: simple geotechnical structures
Category 2: the majority of structures
Category 3: very demanding geotechnical structures.

1.1.3 Impacts on slope stability

Each geotechnical project shall consider the following as the possible impacts on stability:

- soil, rock and water weight
- ground stresses
- earth pressures and groundwater pressure
- free water pressure, including wave pressures
- groundwater pressures
- stream forces
- self weight of structures and other loads originating from structures
- ground loads
- removal of load and excavation of soil
- traffic loads
- displacements due to mining, tunnel construction or construction of other underground areas
- swelling and shrinkage, caused by vegetation, climate or changes in water content
- displacements due to creep, sliding or settlement of earth masses
- displacements due to weathering, clay dispersion, decay, sinking and melting
- displacements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads
- impacts of temperature, including frost action
- loads caused by ice
- anchor prestressing forces
- negative friction
- distribution and performance of construction works
- new slopes and structures at or near the related location

- previous or still active ground displacements of different causes
- climatic changes, including change of temperature (frost and thaw), drought and heavy rainfall
- planting or removal of vegetation
- actions of people and animals
- changes of water content and pore water pressures
- possibility of drainage, filter or sealant failure

1.1.4 Ground data

The stability of natural or artificial slopes is mainly influenced by the following ground data:

- terrain morphology
- soil composition
- shear strength of individual ground layers (drained: c', φ' or undrained: c_u)
- volume weight of individual ground layers
- distribution of pore pressures in aquifer layers (see Section 1.1.9).

And in rocks also

- direction and gradient of inflow of all discontinuity systems, and
- shear strength along individual discontinuity systems.

In heterogeneous ground global stability of slopes depends essentially on the water presence and the resulting distribution of pore pressures in aquifer layers. For this reason tests should carefully record the phenomena of humid (wet) zones, even if only thin layers. Special attention shall be paid to aquifer layers between two non-permeable layers, regardless of their thickness.

In those cases when the impacts caused by constructing cuts or embankments necessitate checking the limit state of applicability, also the data on the rigidity of individual ground layers are important. When choosing computational values of rigidity, the magnitude order of strains and the manner of stress change (loading, unloading) shall be taken into account, which means that adequate material model of higher order shall be used.

The ground data shall be acquired with adequate testing of soil composition and properties.

Computational material characteristics shall present safe value, defined based on all the available test results.

1.1.5 Stability analyses of slopes

Global stability of slopes, together with the existing or planned structures in the influential area, shall be checked by carrying out stability analyses or failure probability analyses according to one of the verified methods, i.e.:

- in soils:
  - with analytical calculations for the assumed slip surfaces of simple shapes
  - with numerical calculations according to the lamella method for the assumed slip surfaces of circular, in sections flat or more complex forms (methods of Bishop, Janbu, Morgenstern and price, Spencer, Sarma…),
  - with numerical calculations according to the MKE or differential method,

Circular failure surface can be used for the analysis of slopes with relatively homogeneous and isotropic materials. Especially in those cases, when landslides along the contact of two different ground layers or along explicitly weak ground layer are expected, exclusive use of circular slip surfaces for the stability analysis is not acceptable.

When selecting the computational method, the following shall be taken into account:

- ground stratification,
- presence and impact angle of discontinuities,
- filtration and dispersion of pore water pressures,
- if it is a matter of short-term or long-term stability,
- creep due to high level of shear stresses,
- failure type (circular or arbitrary slip surface; overturning; flowing).

When analysing global ground stability, in the earth or rock, all the possible failure forms and types shall be taken into account.

When analysing stability, partial safety quotients shall be considered according to the method of limit states, i.e.:

- for effective shear angle γ_{φ} =1,25
- for effective cohesion γ_c =1,25
- for undrained shear strength γ_{cu} =1,40
- for uniaxial compressive strength γ_{qu} =1,40
- for self-weight of the ground γ_{γ} =1,00[1)]
- for permanent load on the ground surface γ_G =1,35
- for temporary load on the ground surface γ_Q =1,50
- for the analysis method γ_M =1,00 (or according to the user's estimation).

[1)] possible unreliability in determining the volume weight of the ground is taken into account in such way to repeat the analysis with the smallest and the largest volume weight.

When analysing slopes which used to be part of unstable slopes (fossil landslides), upper partial safety quotients are not necessarily appropriate. In such cases it should in the first place be shown that the construction of the cut or slope will not reduce safety compared to the initial state. Part of this analysis is also reverse stability analysis of the initial terrain state, which checks also if the input parameters are appropriate.

When selecting computational (characteristic) values of material properties for individual soil layers, deformation compatibility shall be taken into account. Especially in those cases soil layers of different rigidity or relatively rigid structures and more deformable soils contribute to safety, changeable deformations in individual soil layers frequently do not allow the activation of full shear resistance.

Especially the stability analysis in rocks will also require the application of the 3D stability analyses, depending to the rock structure (spatial position of discontinuities).

1.1.6 Providing slope stability with structural measures

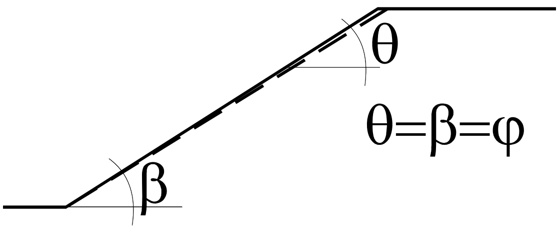
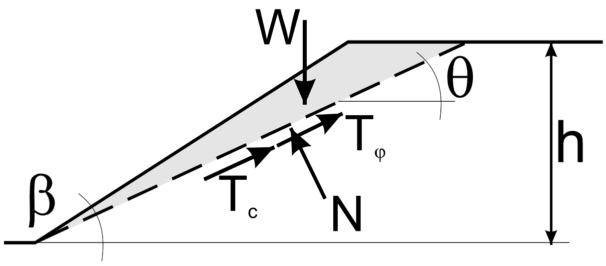
Adequate safety shall be assured to the potentially unstable slopes

- with the change of the slope geometry,
- by planting (mainly as protection against erosion),
- with drainage systems,
- with retaining structures, such as stone walls, crib walls, gabions, made of barb wires or geotextile,
- with earth or rock rod anchors,
- by reinforcing the soil,
- with concrete or reinforced concrete retaining or supporting structures with or without anchors,
- with the combination of all the above stated measures.

1.1.7 Stability of embankment slopes

Stable embankment gradient (β) of a homogeneous embankment, along which water is filtrated, can be for the cases of coherent and incoherent soils calculated according to the following equations:

Table 1: Quick control of slope stability of homogeneous soils without the presence of water

Incoherent soil	Coherent soil
	
$\tan\beta \le \tan\varphi'_d$	$h_{mej} = \frac{2c'_d}{\gamma} \frac{\sin\beta\cos\varphi'_d}{\sin^2\frac{\beta-\varphi'_d}{2}}$

where:

$\tan\varphi'_d = \tan\varphi' / \gamma_\varphi$

$c'_d = c' / \gamma_c$

Empirically, in most cases slopes with the following gradient *(height : length)* are appropriate:

- 1:3 for slopes from softer soils and side embankments,
- 1:2 for slopes from fine-grained soils and from aggregates of soft rocks (marl, flysch, permocarbonic rocks, etc.),
- 1:1,5 (2:3) for gravel embankments,
- 1:1 for embankments from stone material with consolidated stone cover.

Steeper embankment slopes (up to 90°) can be made by reinforcing the fill soil material or by constructing retaining structures. Contemporary software for stability analysis allows also the consideration of reinforcing geotextiles and structures in the stability analysis.

Slopes of heterogeneous embankments, made from different materials, and/or embankment slopes filtering water, shall be analysed using numerical methods of stability analyses.

Shear characteristics of fill material shall be defined by testing samples, compacted according to the Proctor method at optimal water content.

Embankments shall be made with filling over the design edge of the slope, as quality compacting up to the embankment edge is not possible. The execution with over-filling and later removal of excess material from slopes provides adequate (design) quality of the fill material also in the embankment slope.

Unfavourable effects of surface erosion can be prevented in the most efficient way with immediate protection of the newly formed slope by planting.

Unfavourable effects of water infiltration through the embankment can be prevented in the most efficient way with adequate layer of the drainage material at the contact between the embankment and the foundation ground. Minimum depth of such layer for long-term stability amounts to 1 m. Also, at the lowest point adequate filtering stability and water outflow shall be foreseen.

1.1.8 Stability of slopes for cuts in soils

For a quick evaluation of the stability of cuts in homogeneous soils without the presence of groundwater the equations from Table 1 can be used.

Critical for the stability of cut slopes is the final, drained state of the slope, which shall be dealt with effective parameters of shear strength.

The undrained state provides better safety, but only for a relatively short time, i.e. at temporary cuts. To take the advantage of temporarily higher safety at temporary cuts existing during the construction of the cut parts of structures in materials such as clay or silt, it is necessary to assure the protection of temporary slopes and their direct background against rainwater. This is especially vital for silts and clays with low plasticity and/or with sand silts and clays.

During the design and construction extra attention shall be paid to the phenomena of thinner layers of permeable soils (sands, gravels), between layers of coherent soils, through which water is filtered.

The inclination of cut embankments in soils is empirically the same as it is given in Section 1.1.7 for slope embankments. In any case, the stability shall be proved according to the provisions of this section and item 1.1.7. In rocks there can be stable as well as very steep embankments with the inclinations up to 5:1. In soft rocks (marl, slate, flysch) the embankments will normally have the inclination of 2:3 to 3:2. For steeper embankments it is very difficult to set up a vegetation protection. Empirically, in hard rocks (limestone, dolomite, magmatic rocks) the inclinations of 2:1 to 5:1 are being used. In rocks the stability shall be checked according to the provisions from item 1.1.9.

At longer cut slopes berms shall be foreseen at 8 to 12 m of the cut height. Their role is mainly:

- to decrease the erosion action of water,
- to decrease the general gradient of the cut slope,
- to allow access for the purpose of maintenance,
- to block rolling stones, snow slides.

For the drainage of water, berms shall be designed with longitudinal paved drainage trenches. Drainage pipes below the berm are not recommendable.

The construction of cut shall also be used as an opportunity to test the ground, mainly to compare the predicted and the actual soil composition, and, if necessary, also to take samples and make additional field measurements. When necessary, based on the new data, the analyses from the project shall be repeated.

1.1.9 Stability of cut slopes in rocks

Stability of slopes and cuts in rocks shall be checked against the possibility of translation and rotation of individual blocks of rock or larger stone masses, as well as regarding the possibility of falling rocks. Special attention shall be paid to pressures, caused by stale water in the cracks.

It shall be considered that slope or cut failure in solid rock with well defined crack systems may include:

- sliding rock blocks or pins,
- overturning of blocks or slabs,
- combination of overturning and sliding,

depending on the orientation of the slope and the direction of discontinuities.

Sliding of individual blocks or pins can generally be prevented by decreasing the slope gradient and by building-in anchors and internal drainages. At the cut slopes sliding can be prevented with adequate choice of the direction and orientation of the slope front, thus kinematically preventing displacements of rock blocks.

Overturning of blocks can generally be prevented by anchors and internal drainages.

When studying long-term stability of slopes and cuts, harmful effect of vegetation, environmental factors or pollution on shear resistance along discontinuities and on the strength of intact rock shall be considered.

In those cases, when falling rocks cannot be reliably prevented, they shall be left to fall and they shall be protected with nets, barriers or other appropriate measures. The project of measures to stop falling rock blocks and gravel along rocky slope shall be based on careful analysis of the possible trajectories of falling material.

There are several possibilities to analyse the stability of cuts in rocks:

- analyses of potentially unstable blocks and pins based on the direction and incoming discontinues are appropriate for stones with smaller number of discontinuity systems. The only possible slip surfaces are along the existing discontinuities. For this reason shear parameter, valid for cracks, shall be considered. These analyses are called "structural stability analyses". For simple cases they can be performed analytically, and more complex cases can be dealt with graphically. However, most frequently numerical methods must be used
- for strongly cracked and/or slaty rocks stability can be analysed using methods, applied for rocks. Potential slip surfaces can run partially along the existing (different) discontinuities, and partially in different direction. For this reason, for shear strength some average values of the strength for the total rock mass shall be taken into account (the Hoek-Brown criterion for cracked rocks)
- alternatively, the stability of slopes can be evaluated with the help of classification methods for rocks. Especially for the stability evaluation of rock slopes the classification called "Slope Mass Rating" (SMR) has been elaborated.

1.1.10 Technical monitoring

Slopes of cuts and embankments and their surrounding shall be monitored with adequate equipment, when

- calculations prove that the probability reaching limit states is small enough, or
- the presumptions, used in the computational analyses, are not based on reliable data.

Technical monitoring shall be planned in such way that it allows defining

- the groundwater table or the size of pore water pressures in the ground, by carrying out or checking the analysis with effective stresses,
- horizontal and vertical displacements in unstable ground, to allow the prediction of further deformations,
- depth and form of the slip surface in active landslide, to allow the definition of strength parameter of the ground for the project of repair works,
- velocity of displacements, to warn against the emerging danger. In such cases the system of remote digital reading of the measuring devices or the use of remote alarm system may be appropriate.

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Guideline 2: Embankments on low bearing foundation soil

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1.2 EMBANKMENTS ON LOW BEARING FOUNDATION SOIL

1.2.1 Subject of specification

Embankments on soft ground are those, where there is danger that during or after the construction such deformations would develop in the ground that could negatively influence the safety, durability and applicability of the embankments and of structures on it.

Under additional load excessive pore pressures develop in the soft ground, decreasing only slightly during the construction. During the construction low consolidation level is achieved, which is why the majority of the strains develop only after the construction. During the embankment construction the shear resistance in the ground increases very slowly.

Soft load-carrying and compressive ground are normally built of peat and organic soils, or of fine-grained cohesive soils, such as clays, silts and sands in viscous, soft consistency. To soft ground belong also non-uniformly settled mixed sediments and loose, non-controlled artificially filled materials. Soft ground cannot be described with uniform limit values on undrained shear strength and deformation modulus, since it is always necessary to analyse the interdependence between

- ground properties,
- embankment geometry,
- planned construction times, and
- sensitivity or vulnerability of structures in or on the embankment to deformations.

With regard to embankments built on soft and compressive ground, attention shall be paid to the following:

- check the allowable properties of the foundation ground and the risks of failure due to exceeded shear strength of soils and in the foundation ground during the construction
- calculations of settlements and time development of settlements during and after the construction

Critical for the embankment stability on soft ground is the construction phase immediately after the finished construction.

For the construction of embankments on soft ground different construction methods are used, adapted to the existing circumstances. The decision-making process related to the construction shall always consider past experience in comparable conditions.

1.2.2 Symbols applied

A	Skempton parameter
A_c	gravel pile cross-section
A_R	gravel share in the soil composite
A_∞	Area of the diagram of additional stresses
a	distance between the centre of circular failure surface and direction of force T_{cm}
B	*Skempton parameter*
b	width (of the embankment, foundation, wick drain)
b'	embankment width at the influential area of the slope
C_c	index of compression
C_c	index of recompression
c	cohesion
c'	cohesion in term of effective stresses
c_u	undrained shear strength

c_v coefficient of vertical consolidation
c_r coefficient of radial consolidation
E Young modulus
E_{oed} oedometer modulus of compressibility (also M_v)
E_a active earth pressure
E_p passive earth pressure
F_c, F_φ factors of safety on the shear strength of soils in terms of effective stress
F_U factors of safety on the shear strength of soils in terms of effective stress
f factor as result of calculation
H horizontal load or component of the total action acting parallel to the foundation base
h height (of the embankment, foundation)
h_w water level
K_0 coefficient of earth pressure at rest
k coefficient of water permeability
l length (of the embankment, foundation)
m *number*
N_c, N_q, N_q *Bearing capacity factors (Prandtl, Terzaghi, Vesić)*
n number; (e.g. number of piles, tests, slope gradient 1:n)
q ground resistance below the foundation slab, load
q_c ground load below the gravel pile
R influential radius of the gravel pile
r *radius*
r_c gravel pile radius
s settlement
s_0 immediate settlement
s_1 consolidated settlement
s_2 settlement due to viscous creep (secondary settlement)
s_c gravel pile settlement
s_s soil settlement around gravel pile
S_r saturation degree
T_{cm} mobilized reactive cohesion force
Tr time factor of radial consolidation
Tv time factor of vertical consolidation
u pore water pressure
U_R degree or radial consolidation
Uv degree of vertical consolidation
W weight (of the soil)
z vertical distance
x distance

Greek letters

2α inner angle of sector of circular curve
β slope inclination (positive with rising terrain)
γ weight density – unit weight
γ_N weight density of the fill material
γ' effective volume weight
γ_w weight density of groundwater
γ_s specific weight (volume weight without pores)

θ inclination of direction of load application

σ normal total stress

σ_p over consolidation pressure

σ' normal effective stress

ρ shrinkage (settlement)

η ratio (between vertical stresses in the pile and in the foundation ground)

ν Poisson coefficient

τ shear stress

φ' effective angle of internal friction

Abbreviations

CPT static penetration test

CPTU static piezocone penetration test

OCR over consolidation ratio

SPT standard penetration test

For the geotechnical calculations the following units and their multiples are recommended:

Force kN

Mass kg

Moment kNm

Density kg/m^3

Weight densit kN/m^3

stress, pressure, strengths, stiffness kPa

permeability coefficient m/s

consolidation coefficient m^2/s

Categories of structures according to Eurocode 7

1. category: small and relatively simple geotechnical structures

2. category: conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions (the majority of structures)

3. category: highly demanding geotechnical structures (very large, unusual, structures in highly seismic area..)

1.2.3 Methods of constructing an embankment on soft ground

1.2.3.1 General

Methods of constructing an embankment on soft ground include:

- methods, related to the embankment geometry and fill material properties (controlling the geometry of the embankments by flattening the slopes or providing wide berms at the toe and controlling the weight of the fill material)
- methods, related to the improvement of properties of the foundation soil (improving the properties of the foundation material by ground treatment methods or removal of the soft material by excavation plant or displacement by surcharging with suitable material)
- methods, related to time adapted and controlled construction procedures (controlling the rate of construction so that there is time for the foundations to consolidate and hence increase in strength sufficiently to remain stable. In these conditions it is desirable to monitor pore water pressures and lateral and vertical movements, which may take place) and
- different combinations of methods, mentioned above.

The adequacy of the selected construction method shall be proven by

- analysing embankment stability during and after the construction

- analysing settlements and time development of settlements, and
- analysing feasibility and price.

1.2.3.2 Construction methods, related to the embankment geometry and fill material properties

In order to increase (improve) stability of embankments on soft ground, the following adapted methods of embankment construction can be used:

- embankment construction with mild slope gradients (1: n), n = 2.5, 3, 4
- embankment construction with side slopes along the main embankment
- embankment construction from light or extremely light fill materials, such as: fly ashes, aggregates from expanded clay, slabs from extruded polystyrene, slabs of foamed cement concrete, etc.

At the construction of embankments with mild slope gradients the stability conditions are more favourable, but the embankment settlements are larger. The construction of lateral embankments has similar effects as the construction of embankments with very mild slope gradients.

When constructing the embankments for roads, the size of the foundation ground load depends mainly on the volume weight of the fill material. Volume weight of normal soil fill materials is between 18 and 24 kN/m^3. By using light or very light materials the loads on the foundation ground decreases, thus directly increasing safety and decreasing settlements.

1.2.3.3 Construction methods, related to the repair or improvement of soft ground

Methods for the repair or improvement of soft ground include different technological measures, which increase the permeability and/or improve strength and deformation properties of the foundation ground. Among these methods the following are most frequently used:

- building-in wick drains (gravel piles, wick drains) to accelerate consolidation in radial and vertical direction; the method is appropriate, when the influential depth of soft soil is large (5 to 30 m)
- building-in horizontal drainage ribs to accelerate vertical and horizontal consolidation and to improve shear resistance of the soft layer; the method is appropriate, when the influential depth of soft ground is relatively small
- substitution of soft ground with better materials; the method is appropriate when the influential depth of the soft ground is relatively small
- improving shear strength of soft ground by the methods of jet grouting (jet-grouting piles) or deep chemical stabilization (lime piles); the methods are appropriate, when the influential depth of soft ground is large (5 to 30 m)
- improving properties of soft grounds by the methods of additional compression (dynamic compression - heavy tamping, deep vibrating); the methods are appropriate in more permeable, non-homogeneous soils, such as silts and sands and non-controlled artificial fills
- improvement of the surface strength of the foundation ground by reinforcing it with geotextiles.

In special cases, when the described methods cannot ensure homogeneous improvement of the foundation ground properties in the foreseen time, the embankments can also be built on driven or drilled piles.

The stated methods are used independently or in combination with methods, described in sections 1.2.3.2 and 1.2.3.4.

1.2.3.4 Special methods, related to time-adapted construction methods

With the time-adapted construction methods the rate of embankment construction adapts to the achieved consolidation level in the foundation ground below the embankment. During the embankment construction the following shall be carried out:

- measurement of pore pressures in the foundation ground, or
- measurement of increases in shear strength, or
- measurement of displacements in the foundation ground or on the embankment.

The rate of constructing embankments on soft ground shall be adapted to the measurement results and analyses of the achieved consolidation levels. According to the principle of execution, most frequently the following forms of time-adapted construction methods are used:

- time-adapted growth of embankments with constant measuring and monitoring of the excessive pore pressures in the foundation ground
- construction with pre-loading; pre-loading means completing and consolidating the constructed embankment to the required consolidation level, before the construction of the pavement
- construction with pre-loading and overloading; in this case beside pre-loading also additional loading is carried out, which mobilises in the foundation ground adequately larger settlements than those that could develop only by preloading in the consolidation period; overloading is normally carried out on the foundation ground with viscous soils, where an important part of the settlements can develop due to creep or in cases when fast mobilisation of settlements is needed.

Obligatory element of the time-adapted construction is geotechnical monitoring.

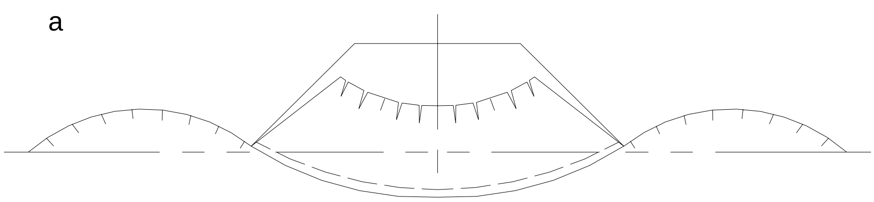
1.2.3.5 Construction of test embankments

When the efficiency of special methods is difficult to predict or capture within the calculation of deformations and stability, test embankments are constructed. Test embankments shall be built at the characteristic section of the line, with the same materials and by using the same methods as will be used at the main embankment. The geometry of the test embankment shall be such that the acquired information is appropriate for the evaluation of the construction methods within the regular embankment construction.

1.2.4 Basics of the embankment design at soft and compressible ground

1.2.4.1 General

The basic principle of embankment design on soft soil is to check the embankment safety against failure for different construction phases. Three possible forms of embankment failure (Figure 1) show the types of stability analyses to be performed. Compared to the embankments on stiff ground, the embankments on soft ground require beside stability calculus also checking of the settlements, their time development, and the embankment creep.



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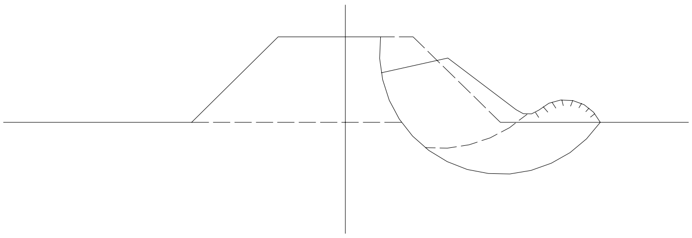
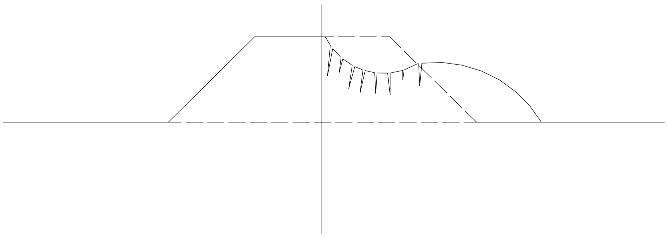




c

Figure 1: Typical examples of deformations on embankments on soft ground as a consequence of: a – exceeded bearing capacity, b – exceeded shear strength, c – pressed out soft ground just below the embankment

The background for the preliminary geotechnical calculations is provided by geological and geotechnical tests, and the control calculations shall be carried out based on the tests and monitoring results, acquired during and after the construction.

1.2.4.2 Geological and geotechnical tests - preliminary tests

The aim of the geological and geotechnical tests is to acquire the data on the characteristic geological geotechnical profiles of the ground in the transverse and longitudinal direction of the embankment. The depth of the tests shall be such to include all layers where settlements are expected to develop due to additional load of the embankment, and all layers relevant for the estimation of the embankment stability.

With no special additional demands, the calculus of settlements shall take into account the compressible layers of the ground to the depth, where the magnitude of additional vertical pressures due to embankment load is smaller than or equal to 20 % of the natural geological pressure.

The type and the scope of the tests, the distribution of testing probes, the depth and the number of samples depend on the conditions in the foundation ground and on the geotechnical category of the structure. Road embankments normally belong to category 2.

Tests shall be planned in such way to acquire for all the typical soil layers inside the foundation ground the following material data:

- Index tests: weight density, grain size distribution, plasticity, specific weight
- strength parameters
- deformability parameters
- permeability coefficients.

When designing embankments on soft ground, beside the classic tests by drilling and sampling, advantage shall be given to the following methods and in situ tests:

- CPT and CPTU test
- dilatometer test
- field vane test
- pressiometer test
- SPT and other penetration tests in non-cohesive soils.

The laboratory tests shall define the index properties of soil, such as grain size distribution and plasticity, weight density and specific weight of grains, and possible presence of harmful chemical and biochemical substance.

Special attention shall be paid to the deformability tests as well as drained and undrained shear strength. When performing the tests, the depth of the taken sample, natural pressures in the foundation ground before the construction and all additional pressures that may emerge in the foundation ground due to embankment construction, as well as changes of the groundwater level and other secondary impacts shall be taken into account.

1.2.4.3 Establishment of the characteristic soil profile and determine the engineering properties of the foundation ground

The characteristic soil profile must include

- The relevant data about the existing ground surface, including all typical morphological structures: dewatering trenches, subsurface drainages, wet zones
- The relevant data about the existing and the newly designed structures, which can be influenced by the embankment
- The relevant data from field investigations (bore – holes, penetration tests, etc)
- Groundwater table (location, fluctuation)
- Subsurface stratigraphy and soil profile

The characteristics soil profile must be interpreted in a proper scale (recommended scale is 1:100 or 1.200.

Engineering properties of the subsoil must be interpreted by:

- weight density(γ_s,γ, γ', γ_d)
- undrained shear strength (c_u)
- drained shear strength (c', φ')
- deformation parameters (E_{oed} or K, G, and E and ν) or consolidation parameters (C_c, C_r, c_v, σ_p)
- permeability k (σ)
- chemical and biological factors that may be detrimental to the reinforcement (huminic acids, sulphates)
- variation of properties with depth and areal extent.

1.2.4.4 Definition of the embankment dimensions and loading conditions

During the process of designing, the following steps are recommended:

Define the embankment dimensions:

- embankment height(h)
- embankment width at toe (b)
- embankment width at the influential area beneath the side slope(b')
- embankment length (l)
- embankment side slopes (h/b').

Define engineering properties of embankment fill materials:

- classification properties
- moisture – density relationships and the expected volume weight of the fill material
- shear strength and deformation properties

Define external loads:

- surcharges

- traffic loads
- dynamic loads (earthquake).

Define environmental considerations:

- frost action
- shrinkage and swelling
- drainage, erosion, floods

Define embankment construction rate

- project constraints
- anticipated or planned rate of construction.

1.2.4.5 Checking of the limit states

The following limit states should be checked at the design of embankments on soft ground:

- loss of overall site stability and loss of stability in different construction phases
- failure in the embankment slope or crest
- deformation in the embankment leading to loss of serviceability (e.g. excessive settlements or cracks on the road surface, dewatering systems),
- settlements and creep displacements leading to damages or loss of serviceability in nearby structures or utilities
- excessive deformations in transition zones (e.g. the access embankment of a bridge abutment)
- changes of environmental conditions (such as pollution of surface groundwater, lowering the ground water level)

1.2.4.6 Establishment of the minimum appropriate safety factors and settlement criteria for the embankment

The minimum appropriate safety factors are recommended by EN 1997. For the end of the construction (drained conditions) these are: Effective cohesion: $F_{c'}$ = 1,25 and effective shearing resistance $F_{\varphi'}$ = 1,25, for the construction time (undrained conditions) these are: undrained strength F_u = 1,40:.

Settlement criteria depend upon project requirements.

For the intermediate stages of constructions, the minimum safety factor depends on different factors (uncertainties of loads, analysis) and must be adjusted in such a way that the overall stability and the deformation are within the required values. The typical intermediate safety factor is $F_u > 1,15$.

1.2.5 Principles of geotechnical calculation of the embankments on soft ground

1.2.5.1 General

To calculate bearing capacity of soft soil, embankment stability, deformations and time development of deformations, different semi-empiric, analytical and numerical methods can be used, adapted for the application in different cases. The input data for the calculation are characteristic values, obtained from the data of field and laboratory tests.

1.2.5.2 Checking bearing capacity

When the thickness of soft soil is larger than the embankment width, for the approximate calculus of limit embankment load the following equation can be used (Prandtl), (Figure 2.1)

$$\gamma_N h = q_f < (2+\pi) c_u \qquad (1)$$

To calculate the influence of embankment slope gradient on the bearing capacity of the ground the following relation can be used (Figure 2.2)

$$\frac{1}{n} < \frac{1.95\, c_u}{\gamma_n h} \qquad (2)$$

Favourable influence of lateral embankment on the stability of the main embankment is shown in the relation (Figure 2.3)

$$q_f = 0.5\gamma\ B\ N_\gamma + c\ Nc + \gamma\ D\ N_q \qquad (3)$$

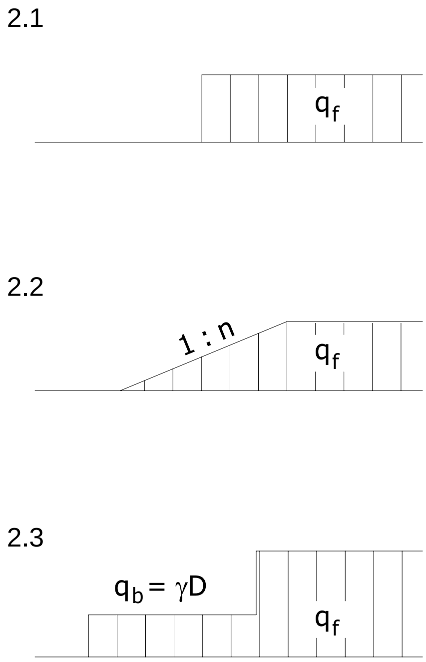


Figure 2: Schematic presentation of the embankment slope geometry on the bearing capacity of the ground below the embankment

The parameter of undrained shear strength (c_u) can be defined by in-situ tests or in laboratory. For the in-situ determination the following procedures are used: static cone penetration test (CPT), dilatometer test (DMT) and field vane probe. In laboratory the undrained shear strength can be determined with the following procedures: triaxial undrained unconsolidated test (UU), uniaxial compressive strength test using cylinder specimens ($c_u = q_u/2$), test with laboratory vane or laboratory cone.

1.2.5.3 Checking rotational shear stability

In the foundation ground the embankment construction causes the increase of excessive pore pressures and thus decrease of safety against failure. Critical is the condition immediately after the finished embankment construction, when excessive pore pressures in the ground are the largest. The stability calculation by taking into account undrained shear strengths ($\tau_u = c_u$, $\varphi_u = 0$) or the calculation by taking into account the drained shear strength and excessive pore pressures in the foundation ground (c', φ', Δu) agree to this state.

In the case of calculating stability by taking into account the undrained shear strength, pore pressures due to water level and additional loads do not need to be considered.

If the stability is calculated immediately after the embankment has been constructed, by taking into account the drained shear strength, normally the excessive pore pressure values (Δ u) are considered, which are the same as the values of additional vertical pressures due to slope weight ($\Delta u = \Delta \sigma_{zz}$).

The parameters of drained shear strength (c',φ') can be defined by in-situ tests or in laboratory. For the in-situ determination the following procedures are used: static cone penetration test in sands (CPT), dynamic penetration test in sands and gravels (SPT) and special methods for direct measurements of shear strength in bore holes or in excavations. In laboratory the drained shear strength can be determine by triaxial consolidated undrained test (CU), translatory and rotary direct shear test.

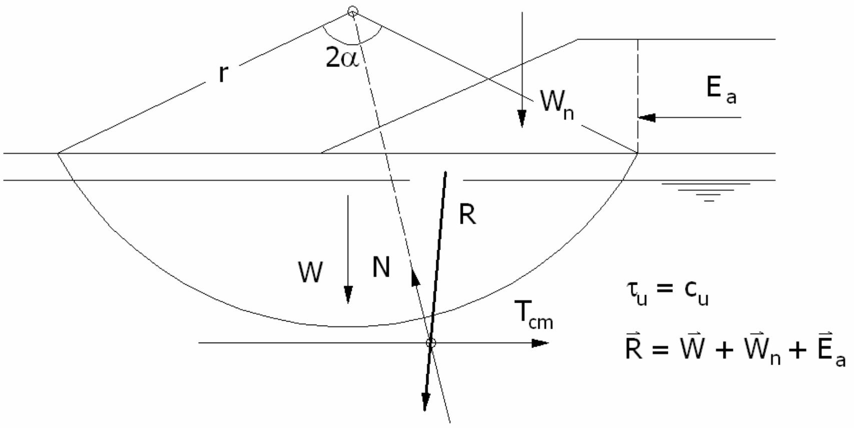


Figure 3: Graphic presentation of determining embankment stability in undrained conditions

If the undrained shear strength ($\tau_u = c_u$) along the circular slip surface is constant, the safety quotient F_u is calculated by taking into account that force T_{cm} lies in the direction parallel to the chord connecting the beginning and the end of the slip surface, is perpendicular to the centreline of the circular slip surface and is located away from the slip surface centre by the distance $a = r\,\alpha / \sin\alpha$.

The size of the mobilized reactive cohesion force T_{cm} can in this case be calculated from the moment equilibrium condition on central slip surfaces.

$$\sum M^0 = 0 \quad \Rightarrow \quad W\,x_W + E_a\,y_E = T_{cm}\,a \tag{4}$$

$$T_{cm} = \frac{W\,x_w + E_a\,y_E}{a} \quad , \quad c_m = \frac{W\,x_w + E_a\,y_E}{a\,l} \quad , \quad F_u = \frac{c_u}{c_m} \tag{5}$$

where:

r - is the radius of circular slip surface

α - is half value of the central angle of the slip surface

x_W - is the distance of the resulting embankment weight W_N to the slip surface centre

y_E - is the distance of force E_a to the centre of the slip surface

l - is the length of the chord connecting the beginning and the end of the circular slip surface

When calculating stability by taking into account the drained shear strength, due to water table beside pore pressures also the excessive (additional) pore pressures due to embankment load shall be considered. For the normally consolidated and saturated coherent soils pore pressures are increased due to the embankment load, and are the same as additional vertical pressure. To calculate excessive pore pressures (Δu) the following relation can be used:

$$\Delta u = B[(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \qquad (6)$$

Skempton's parameter *B* equals the saturation level S_r . Normally the saturation level of poor structural soil is 100%. However, Skempton's parameter *A* has the value 1 for normally consolidated soils and the value smaller than 1 if the soil is overconsolidated. For poor structural soil it is appropriate to assume Skempton's parameter *A* with in size of 1.

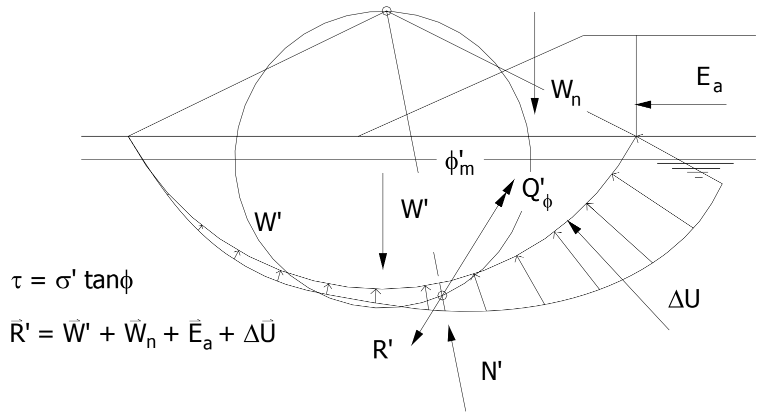


Figure 4: Graphic presentation of calculating embankment stability in the drained conditions

With progressing consolidation the embankment safety against failure increases up to the final value, reached when the excessive pore pressures fall to zero values. After the ground consolidations, the resultant of the excessive pore pressures Δu is zero.

In the case that the stability calculus in undrained conditions shows insufficient safety against failure, methods shall be prescribed with design and structural demands, which ensure adequate safety.

The slope stability calculations on soft soil can be carried out by using the failure probability analyses according to one of the recognised methods:

- analytical calculations for the presumed slip surfaces of simple shapes in homogeneous ground,
- numerical calculations according to the lamella methods for the presumed slip surfaces of circular, sectionally flat or more complex shape (the methods of Bishop, Janbu, Morgenstern, and Price, Spencer and others),
- numerical calculations according to MKE or differential method considering different constitutive models for the description of ground and slope behaviour (e.g.: elastoplastic models: Mohr-Coulomb, Drucker-Prager, Cam clay, Cap model and others ...).

1.2.5.4 Settlement evaluation

The calculus shall include initial and later settlements. The following three settlement components shall be taken into account:

- s_0: settlement due to the initial compression (distortion) mostly caused by preloading
- s_1: settlement due to the primary consolidation
- s_2: settlement due to the secondary consolidation.

Special attention shall be paid to organic soils, peat and other soft soils, where settlements due to viscous creep can go on for indefinite time. Viscous effects are determined from the data of oedometer tests or triaxial deformability test.

To calculate settlements, different tested and verified methods can be used. Two of these methods are:

- stress-strain method, and
- adapted elasticity method

1.2.5.5 Stress-strain method

With the stress-strain method first the distribution of vertical additional stresses in the foundation ground must be calculated. Here, the impacts of additional embankment loads as well as other loads shall be considered, e.g. impacts of lower water pressures in the ground due to the arrangement of trenches or pumping of water. The total settlement of a foundation on cohesive or non cohesive soil may be evaluated using the stress-strain calculation method as follows:

- computing the stress distribution in the ground due to the loading from the foundation (the influence of the decreasing of groundwater level due to dewatering must be considered),
- computing the strain in the ground from the stresses, using oedometric modulus values or other stress – strain relationships determined from laboratory test. The settlement of each particular layer (ρ_i) is calculated using the equation 7.

Strains in the ground – shrinkages of individual layers (ρ_i) are calculated in such way that the appertaining area of the diagram of additional vertical stresses in and individual foundation ground layer is divided by the modulus of compressibility which agrees to the changes in stress.

$$\rho = \frac{A_\infty}{E_{oed}} \qquad (7)$$

1.2.5.6 Adjusted elasticity method

The shrinkage of an individual layer is calculated as the difference in settlements of the top and bottom layer. The displacement magnitude depends on the embankment load, its shape, the depth of the layer and its deformability (elasticity module E and Poisson number ν). The total settlement of a foundation on cohesive or non cohesive soil may be evaluated using elasticity theory and an equation of the form

$$s = \frac{q.b}{E}.f \qquad (8)$$

where:

f –settlement coefficient, depends on the Poissons ratio, dimensions and shape of the foundation area, the thickness of the compressible formation, the distribution of the bearing pressure

The tolerable post construction settlement and embankment deformations depend on project requirements. When the calculated settlements and deformations exceed the required values, additional methods for ground improvement must be checked.

1.2.5.7 Calculation of the time rate of the consolidation

The time rate of the consolidation is calculated with the help of the consolidation parameters, obtained from the oedometer test:

$c_v = k \ M_v / \gamma_w$ (9)

$T_v = c_v \ t / h^2.$ (10)

$U_v = \rho / \rho_\infty$ (11)

For a quick estimation of the time development of the consolidation in the vertical direction, values shown in Tables 1 and 2 can be used.

Table 1: Achieved consolidation rate at different time factors

T_v	U_v (%)
0,004	7,14
0,008	10,09
0,012	12,36
0,020	15,96
0,028	18,88
0,036	21,40
0,048	24,72
0,060	27,64
0,072	30,28
0,083	32,51
0,100	35,68
0,125	39,89
0,150	43,70
0,175	47,18
0,200	50,41
0,250	56,22
0,300	61,32
0,350	65,82
0,400	69,79
0,500	76,40
0,600	81,56
0,700	85,59
0,800	88,74
0,900	91,20
1,000	93,13
1,500	98,00
2,000	99,42

Table 2: Consolidation time factors at the achieved consolidation rate

U_v (%)	***T_v***
0	0,000
5	0,002
10	0,008
15	0,018
20	0,031
25	0,049
30	0,071
35	0,096
40	0,126
45	0,159
50	0,197
55	0,239
60	0,286
65	0,342
70	0,403
75	0,477
80	0,567
85	0,684
90	0,848
95	1,129
100	≅ 2

When defining the time rate of the consolidation, advantage is given to the permeability coefficients, defined within field tests.

If the calculation of the time rate of consolidation shows that during the planned construction time, the required rate of settlement can not be expected, additional measures to accelerate consolidation shall be prescribed with structural requirements.

1.2.6 Basics of geotechnical calculation of structural measures for the embankment construction on soft ground

1.2.6.1 Calculation of the impacts of changed embankment geometry and properties of substitute fill material

The impacts of embankment geometry on the improved construction safety shall be calculated according to the methods, described in section 1.2.5.2. The impacts of decreased embankment slope gradients (n) compared to the initially designed conditions and the impacts of building side embankments shall be checked. At the same time, the impacts of the embankment slope geometry on the settlement magnitude shall be checked.

The calculation of the impacts of the substitute fill material shall be different for

- fill materials, used to replace soft foundation ground, to improve their strength and deformability, and
- fill materials, used as substitute of classical soil materials in the embankment.

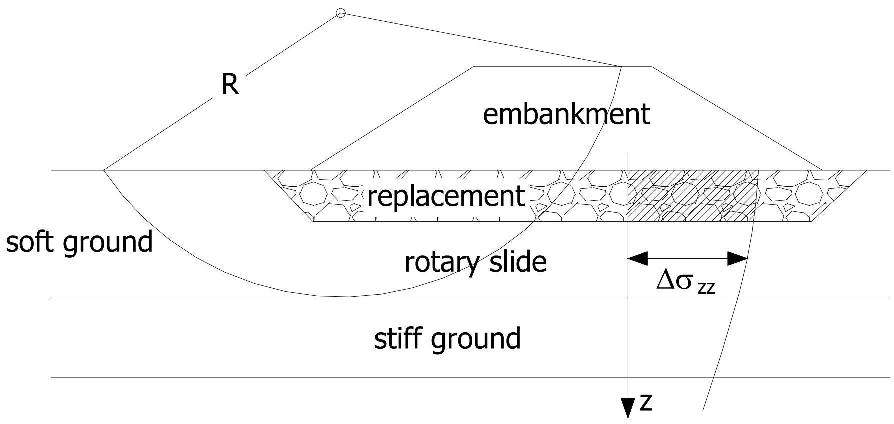


Figure 5: Example of embankment construction with partial replacement of soft ground below the embankment using stone material

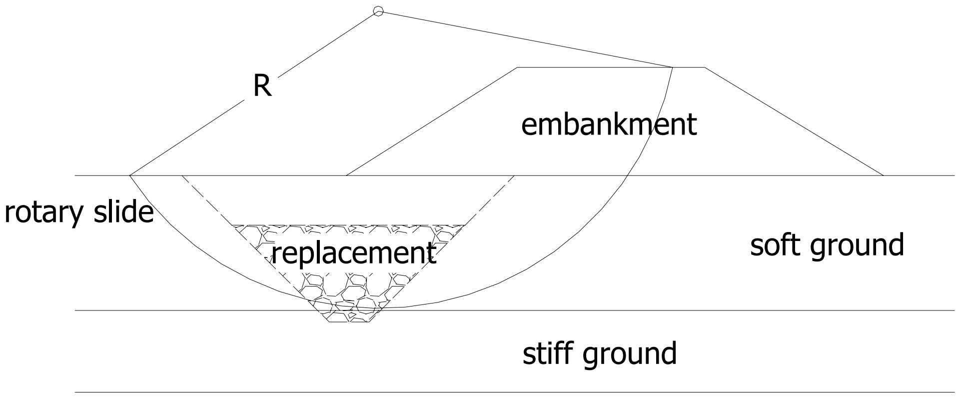


Figure 6: Example of embankment construction with partial replacement of soft ground in the area of the embankment base using stone material

When calculating the impacts of changed properties of the fill material in the filled body, most important is the correct definition of the volume weight of the fill material. Fill materials from conventional soil have weight density between 18 and 24 kN/m^3. Aggregates of expanded clay have for about 4 to 5 times lower weight density (3.5 to 7 kN/m^3), and slabs of expanded polystyrenes for up to 100 times lower weight density (0.25 to 0.35 kN/m^3) than classical soil fill materials.

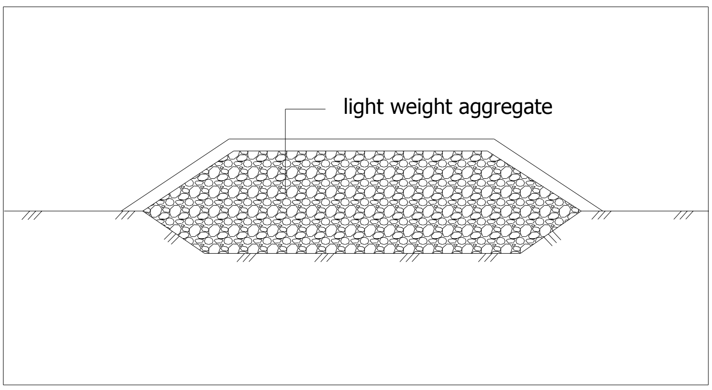


Figure 7: Example of embankment construction by using light aggregates from expanded clay (LECA – light expanded clay aggregate)

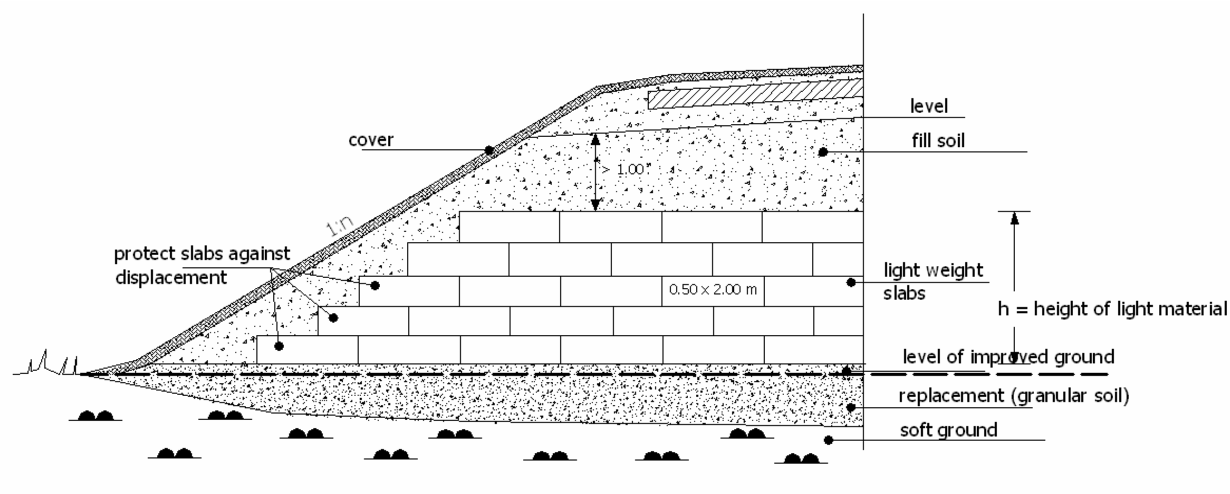


Figure 8: Example of constructing embankment by using slabs of extruded polystyrene.

When conditions require the embankments to be built completely of light slabs, adequate measures shall be taken to protect the slabs below the wearing surface (Figure 9).

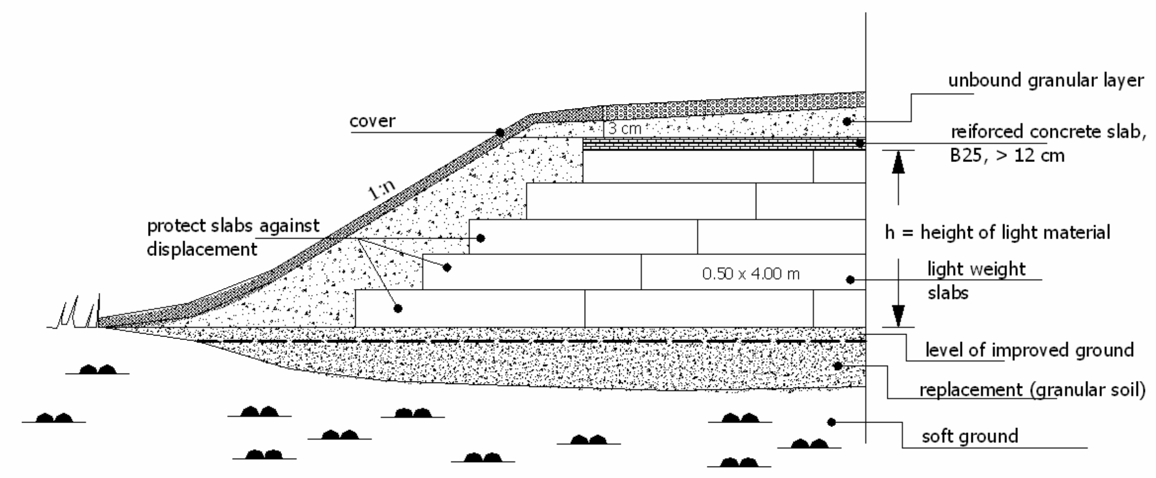


Figure 9: Example of embankment construction by using slabs of extruded polystyrene with cement-concrete protection above the slabs

By using light and very light materials, the loads on the foundation ground decrease, which has direct positive effect on safety and decreases settlements.

In the planning stage for the embankment construction from light and very light materials, the time increments of the material weight due to gradual saturation with groundwater and the structural measures for the protection of these materials against external impacts and animal actions shall be considered. Particularly, the differences in rigidity and deformation behaviour of different materials, built-in below, above and by the sides of the embankment made of light materials, shall be checked.

Also, when constructing embankments from light materials, combinations of the methods with preloading and/or overloading can be used (Figure 10).

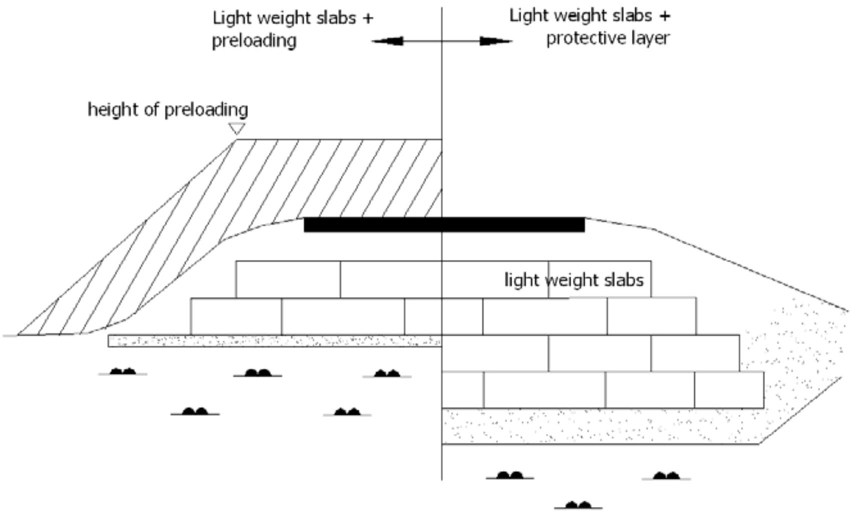


Figure 10: Example of combined embankment construction on soft ground with and without overloading

1.2.7 Calculations of the impact of gravel piles

1.2.7.1 General

By building-in gravel piles, tipple effect is achieved in the foundation ground:

- increased horizontal permeability of the soil layer and accelerated consolidation
- improved bearing capacity of the foundation ground
- smaller settlements below the design load.

The duration of the consolidation is one of the criteria for the design of gravel piles. With these criteria the distance between piles and their dimensions are defined, and the impacts of pile on the bearing capacity and reduced settlements is checked.

Gravel piles usually have the diameter of 40 to 100 cm and length up to 30 m, exceptionally more. The smallest distance between piles is normally 1.5 m.

The impact of gravel piles on the consolidation rate depends on the manner and the density of the pile distribution.

For the selected pile radius, permeability, oedometer module and the expected consolidation time, the number of piles has to be found

n = R/r_c,

to get the consolidation level of 95 %.

1.2.7.2 Total level of radial and vertical consolidation

$$U = 1 - (1 - U_V)\,(1 - U_R) \qquad (12)$$

$$T_V = \frac{c_V\, t}{h^2} = \frac{k\, E_{oed}\, t}{\gamma_w\, h^2} \qquad (13)$$

$$U_V = U_V(T_V) = 1 - \frac{8}{\pi^2} \sum_{1}^{\infty} \frac{1}{(2m-1)^2} \exp\left[-\frac{(2m-1)^2 \pi^2}{4} T_V\right] \qquad (14)$$

$$T_R = \frac{c_R\, t}{4\, R^2} = \frac{k_R\, E_{oed\,R}\, t}{4\, \gamma_w\, R^2} \qquad (15)$$

$$n = \frac{R}{r_0} \qquad (16)$$

$$\mu = \frac{n^2}{n^2 - 1}(\ln n - \frac{3}{4} + \frac{1}{n^2} - \frac{1}{4n^4}) \qquad (17)$$

$$U_R = U_R(T_R, n) = 1 - \exp\left[-\frac{8}{\mu}T_R\right] \qquad (18)$$

1.2.7.3 Distribution of gravel piles

Possible are different distributions of gravel piles: quadratic, triangular, hexagonal, etc.

Quadratic ground-floor distribution

$$R = \frac{1}{\sqrt{\pi}}\, a = 0.564\, a \qquad (19)$$

$$a = \frac{R}{0.564}$$

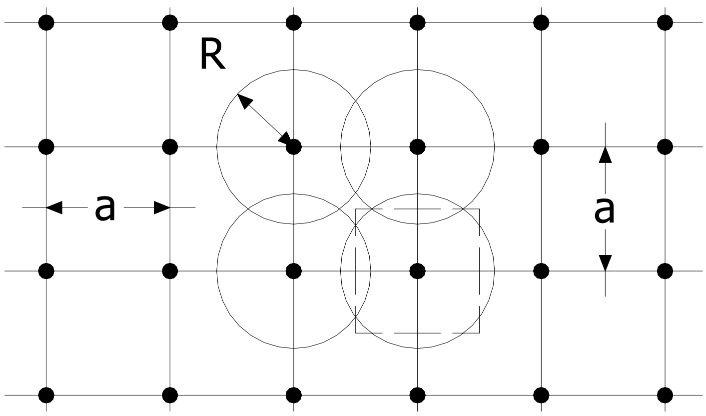


Figure 11: Schematic presentation of the quadratic distribution of gravel piles

Triangular ground-floor distribution

$$R = \sqrt{\frac{\sqrt{3}}{2\pi}}\, a = 0.525\, a$$

$$a = \frac{R}{0.525} \qquad (20)$$

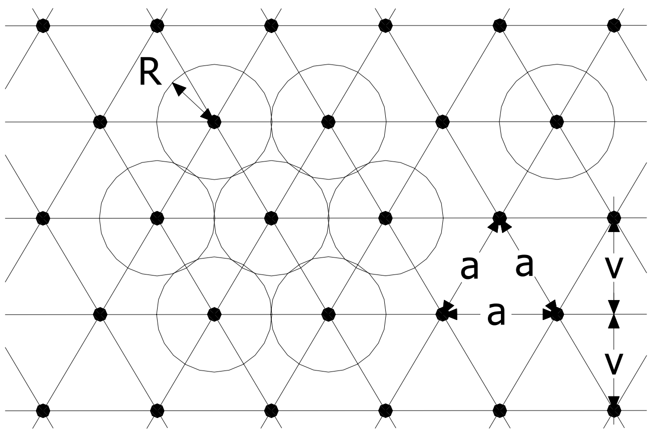


Figure 12: Schematic presentation of the triangular ground-plan distribution

1.2.7.4 Impact of gravel piles on settlements and on the foundation ground bearing capacity

Gravel piles influence the bearing capacity in two ways:

- due to intermediate layers of material with high shear resistance, located in the pile, considerably higher shear resistance of the foundation ground can be achieved
- due to the built-in gravel piles, inside the foundation ground the drained conditions created much faster
- the distance between gravel piles and their radius, the influence of which is included in the nondimensional quotient A_R, have strong impact on the quotient of the settlement reduction β.

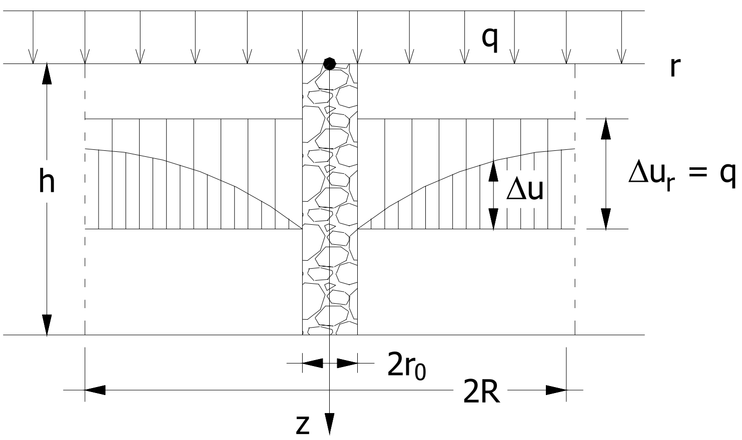


Figure 13: Schematic presentation of a gravel pile

Calculations:

Geometric quantities:

$2r_c$	diameter of a gravel pile
$2R$	influential diameter of the gravel column
a	axial distance between gravel columns
A_c	cross-section of the gravel pile
H	length of the gravel pile
A	cross-section of the cylinder in the influential area
A_s	cross-section of the basic soil
A_R	cross-section share of the gravel pile in the composite

$$A_R = \frac{A_c}{A} = \left(\frac{r_c}{R}\right)^2 \qquad (21)$$

$$A_c = \pi\, r_c^2 \qquad (22)$$

$$A = \pi\, R^2 \qquad (23)$$

$$R = \chi\, a \qquad (24)$$

χ = 0.525 ... triangular ground-plan grid of columns

χ = 0.565 ... quadratic ground-plan grid of columns

χ = 0.645 ... hexagonal ground-plan grid of columns

Settlement of the foundation ground without piles below the embankment is

$$s_o = \frac{qH}{E_{oed}} \quad (25)$$

Final settlement of the foundation ground with the built-in piles is

$$s = s_o\,\beta = \frac{q\,H}{E_{oedn}} \quad (26)$$

Pile settlement is

$$s_c = \frac{q_c H}{E_{oedc}} \quad (27)$$

Soil settlement between piles is

$$s_s = \frac{q_s H}{E_{oeds}} \quad (28)$$

$$qA = q_c A_c + q_s A_s \quad (29)$$

$$q_c = \frac{q\eta}{1 + A_R(\eta - 1)} \quad (30)$$

$$q_s = \frac{q}{1 + A_R(\eta - 1)} \quad (31)$$

$$f(\nu_s, A_R) = \frac{(1-\nu_s)\,(1-A_R)}{(1-2\nu_s + A_R)} \quad (32)$$

$$\sigma_{r\,c} = q_c\;k_{a\,c} = q_c\;\tan^2\left(45^o - \frac{\varphi'_c}{2}\right) \quad (33)$$

$$\eta = \frac{1 + 2\,f(\nu_s, A_R)}{2\,k_{a\,c}\;f(\nu_s, A_R)} \quad (34)$$

η = quotient of the load distribution ($\frac{q_c}{q_s}$), q is the embankment load magnitude

$$q_c = \frac{q\eta}{1 + A_R(\eta - 1)} \quad (35)$$

$$q_s = \frac{q}{1 + A_R(\eta - 1)} \quad (36)$$

$$\beta = \frac{s}{s_0} = \frac{1}{1 + A_R\;(\eta - 1)} \quad < \quad 1 \quad (37)$$

$$s = s_c = s_s \quad (38)$$

Substitute volume weight

$$\gamma_n = A_R\;\gamma_c + (1 - A_R)\;\gamma_s \quad (39)$$

Substitute strength parameters are

- in initial undrained conditions:

$$\tau_{un} = c_{un} + \sigma_{z_n} \tan\varphi_{un} \tag{40}$$

$$c_{u_n} = A_R \, (q_c - q\frac{\gamma_c}{\gamma_s}) \tan\varphi'_c + (1 - A_R)\, c_{u_s} \tag{41}$$

$$\sigma_{z_n} = \gamma_n \, z + q \tag{42}$$

$$\tan\varphi_{u_n} = A_R \, \frac{\gamma_c}{\gamma_n} \tan\varphi'_c \tag{43}$$

- in drained conditions:

$$\tau_n = c'_n + \sigma'_{z_n} \tan\varphi'_n \tag{44}$$

$$c'_n = A_R \, (q_c - q\frac{\gamma'_c}{\gamma'_n}) \tan\varphi'_c + (1 - A_R)\left[c'_s + (q_s - q\frac{\gamma'_s}{\gamma'_n}) \tan\varphi'_s\right] \tag{45}$$

$$\sigma'_{z_n} = \gamma'_n \, z + q \tag{46}$$

$$\tan\varphi'_n = A_R \, \frac{\gamma'_c}{\gamma'_n} \tan\varphi'_c + (1 - A_R)\, \frac{\gamma'_s}{\gamma'_n} \tan\varphi'_s \tag{47}$$

1.2.7.5 Calculation of the impacts of wick drains

The basic purpose of building-in wick drains is to accelerate consolidation due to increased soil permeability in the radial and partially vertical direction. To calculate the impact of wick drains on the consolidation acceleration, similar relations as when calculating gravel piles are considered. The impact of wick drains on the reduction of settlements and improvement of the foundation ground bearing capacity is in the conservative calculation normally neglected.

Substitute radius for the wick drain of width b and thickness t shall be calculated using the following equation

$$2\,\pi\, r_0 = 2\,(b + t) \quad \Rightarrow \quad r_0 = \frac{b + t}{\pi} \tag{48}$$

where:

b - wick drain width

t - wick drain thickness

1.2.7.6 Calculation of the impacts of deep consolidation on improved bearing capacity

The methods of deep consolidation (lime piles, jet grouting piles, consolidations with vibrations) are the methods, which increase the shear resistance of the soil by adding binder or additional energy into the ground. Using the methods of deep consolidation the following is achieved

- improved bearing capacity of the foundation ground,
- smaller settlements below the designed load
- improved safety against failure.

The magnitude of increase in shear strength is defined with the help of laboratory tests, in situ tests at the test fields, or from past experiences.

The calculation includes substitute values of corrected strength for the improved ground and procedures, described in section 1.2.7.5

1.2.8 Special demands for the design of embankments on soft ground

The design of embankments on soft ground shall take into account also the following:

- the decision about the removal of grass turf and layers of humus that often present »strong, load-carrying« crust above soft soils, depends on the geotechnical evaluation of each structure individually
- it is recommended that in soft ground a separating geotextile layer shall be built in before constructing the first gravel layer; to strengthen the foundation ground also reinforcing geotextile can be used; the design of the geotextile is carried out according to the procedures valid for the use of geotextiles
- the first embankment layer above soft foundation ground or above the geotextile shall always be made from well permeable stone - gravel material, functioning as a drainage layer and as a working plateau
- when designing the thickness of the wearing plateau and the stone material layers, the height of the expected settlements and the height of high (flood) waters shall be taken into account.

1.2.9 Geotechnical monitoring at the embankment on soft ground

The design of embankments on soft ground shall also include plans for geotechnical monitoring. The plans for geotechnical monitoring shall be divided into the following measures:

- monitoring the embankment during the construction, and
- monitoring of the embankment after the construction.

The following methods are appropriate to monitor embankments:

- settlement slabs for geodetic monitoring of settlements
- horizontal inclinometers for continuous measurements of settlements in the horizontal direction through the embankment
- vertical inclinometers for continuous measurements of settlements in the vertical direction in the embankment and in the foundation ground
- water pore pressure meters
- piesometres to monitor water levels
- meters measuring water outflow from drainages
- other methods (e.g. extensometers, strain gages to measure strains in the geotextile, etc.)

GUIDELINES FOR ROAD DESIGN, CONSTRUCTION, MAINTENANCE AND SUPERVISION

VOLUME I: DESIGNING

SECTION 1: ROAD DESIGNING

Part 7: ROAD STRUCTURAL ELEMENTS

Chapter 1: EARTH WORKS

Guideline 3: Geosynthetic materials

Sarajevo/Banja Luka
2005

1.3 GEOSYNTHETIC MATERIALS

1.3.1 Subject of specification

The guidelines are valid for the structures of geotechnical categories 1 and 2. For geotechnical category 3 the guidelines present only minimum scope and the basic guidance in the design. With category 3 frequently other procedures and methods will be needed, and the cooperation of specialists will be required.

Geosynthetics are planar products manufactured from a polymeric material used with soil, rock, earth, or other geotechnical-related material as an integral part of a civil engineering project, structure, or system.

Geosynthetics have six primary functions:

- separation
- filtration
- drainage
- reinforcement
- sealing
- protection

Well over 95 % of geosythetics for engineering purposes are made from synthetic polymers: polypropylene, polyester and polyethylene. These polymer materials are highly resistant to biological and chemical degradation. Less frequently used polymers include polyamides (nylon) and glass fibres. Natural fibres, such as cotton, jute etc., could also be used as geotextiles, especially for temporary applications, but they have not been researched or utilized in the same way as polymeric geosythetics.

The main families of geosynthetics are:

- geotextile: a permeable, planar, polymeric textile material, which may be nonwoven, knitted or woven, used in contact with soil, rock and/or other materials in geotechnical and civil engineering applications. The primary functions are: separation, filtration and protection. They can be used also for drainage and reinforcement. Symbol: GTX
- geogrid: a planar polymeric structure consisting of a regular open network of integrally connected, tensile elements, which may be linked by extrusion, bonding or interlacing, the openings of which are larger than the constituents. The primary function is reinforcement. Symbol: GGR
- geonet: a planar, polymeric regular structure consisting of a regular dense network, the constituent elements of which are linked by knots or extrusions and whose openings are much larger than the constituents. The primary functions are separation and drainage. Symbol: GNE
- geomembrane: a very low permeability material in the form of a factory made synthetic, polymeric or bituminous sheet, used in geotechnical or civil engineering applications with the purpose of reducing or preventing the flow of fluid and/or vapour through the structure. Symbol: GMB.
- geocomposite: a manufactured, assembled material, using among its components at least one synthetic product. The primary functions are: drainage and sealing. Symbol: GCO.
- Geosynthetic clay liner: a factory assembled structure of geosynthetic materials and low hydraulic conductivity clay materials (bentonites), in the form of a sheet. The primary function is sealing. Symbol: GCL.

1.3.2 Symbols applied

c cohesion

c'	cohesion in terms of effective stresses
c_u	undrained shear strength
c_a	soil adhesion – geosynthetic material
dh	spacing between reinforcing strips
l_{ai}	non-load-carrying length of the reinforcing strip
l_{ni}	load-carrying length of the reinforcing strip
h	height (of the embankment, excavation)
k	water permeability coefficient
k_a	coefficient of active earth pressure
p_a	active earth pressure
$\check{s}$	reinforcing strip width
E_i	earth pressure
N	normal force at the slip surface
T_c	resulting cohesion resistance along the slip surface
T_φ	resulting friction resistance along the slip surface
u	pressure of pore water
W	weight (of the soil)

Greek letters

β	slope gradient
γ	weight density
γ_c, γ_φ	safety factors on cohesion and shear angle in drained conditions
γ_{cu}	safety factor in undrained conditions
γ_{strip}	safety factor for geosynthetic material
θ	nagib drsine
ϑ	slip surface inclination
θ	slip surface gradient
σ	normal total stress
σ'	normal effective stress
τ	shear stress
φ'	shear angle, expressed by effective stresses

For the geotechnical calculations the following units and their multiples are recommended:

force	kN
mass	kg
moment	kNm
density	kg/m^3
weight density	kN/m^3
stress, pressure, strength, stiffnes	kPa
permeability coefficient	m/s
consolidation coefficient	m^2/s

Categories of structures according to Eurocode 7

Category 1: simple geotechnical structures

Category 2: the majority of structures

Category 3: very demanding geotechnical structures.

1.3.3 Evaluation of properties

Today, there are more than 600 different geosynthetic products available on the market. Because of the wide variety of products available, with their different polymers, filaments, weaving, manufacturing, they have a considerable range of physical and mechanical properties. The properties, listed in Table 1 cover the range of characteristics and test methods, required for use in road construction. The tests, listed in Table – 1 include index test. For specific application requirements, the specific performance tests are required.

Table 1: Geotextiles and geotextile related products used in the road construction – functions, function related characteristics and test methods to be used

		Function			
Characteristic	Test method	Filtration	Separation	Reinforcement	Drainage
Tensile strength [a]	EN ISO 10319	H	H	H	H
Elongation at maximum load	EN ISO 10319	A	A	H	A
Tensile strength of seams and joints	EN ISO 10321	S	S	S	S
Static puncture [a,b]	EN ISO 12236	S	H	H	--
Dynamic perforation resistance (cone drop test) [a]	EN 918	H	A	H	--
Friction characteristics	prEN ISO 12957	S	S	A	S
Tensile creep	EN ISO 13431	--	--	S	A
Damage during installation	ENV ISO 10722-1	A	A	A	A
Characteristic opening size	EN ISO 12956	H	A	--	--
Water permeability, normal to the plane	EN ISO 11058	H	A	A	--
Waterflow capacity in the plane	EN ISO 12958	--	--	--	H
Durability		H	H	H	H
Resistance to weathering	EN 12224	A	A	A	A
Resistance to chemical ageing	ENV ISO 12960 EN ISO 13438 ENV 12447	S	S	S	S
Resistance to microbiological degradation	EN 12225	S	S	S	S

Legend:

H – required for harmonization

A – relevant to all conditions of use

S – relevant to specific conditions of use

-- - indicates that the characteristic is not relevant for that function

[a] – it should be considered that this test may not be applicable for some types of products

[b] - if the mechanical properties (tensile strength and static puncture) are coded »H« in this table, the producer shall provide data for both. The use of only one is sufficient in the specification.

Characteristics required for use in the construction of tunnels, water reservoirs and underground water protection measures are not included in Table 1. The requirements must be specified in the design or in the special technical conditions for those structures.

1.3.4 Technical specifications for geosynthetics

The function of individual geosynthetic varies with its application. The geosynthetic must be designed with its function in the given application in mind.

The definitions of these functions must be defined with:

- general requirements
- specific requirements
- joints, seams and overlays
- installation procedures
- damages and repairs
- field inspections and conditions of acceptance and refuse.

1.3.5 Design with geosynthetics

Geosynthetics are relatively young material, with many possible design methods, which can differ among each other significantly. The ultimate decision for a particular application usually takes one of three directions:

- design by cost and availability. The funds available are divided by the area to be covered and a maximum available unit price than can be allocated for the geosynthetic calculated. The method is weak technically but is still practiced.
- design by specification. In this method several application categories are listed in association with various physical, mechanical or hydraulic properties. The designer decides for a particular application based on specified properties
- design by function. In this case the design consist of assessing the primary function that the geosynthetic will serve and then calculating the required numerical value of particular property, such as hydraulic conductivity, tensile strength, elongation for that function

Safety factor of geosynthetic is given as

$$F_{(G)} = \frac{allowable(test)\,property}{required\,(design)\,property}$$

allowable property: a numeric value based on a laboratory test that models the actual situation

required property: a numeric value obtained from design method that models the actual situation.

When calculating the required numerical value, it is important to consider the compatibility between soil and geosynthetic, especially for deformations. Geosynthetic can develop its peak tensile strength at quite large deformations, while soil, on the other hand, has already collapsed.

When designing with geosynthetics, it is important to consider that in many cases the conditions during installation are much more severe than the conditions during the life time of the structure. It is important to consider all relevant data, which can influence the geosynthetic behaviour and properties during the installation as well as during the life time:

- physical properties
- mechanical properties

- hydraulic properties
- properties, related to the durability and resistance against damages (damages during installation, creep, abrasion)
- properties, related to the degradation due to the ageing, chemical, biological, temperature and other environmental influences.

It is important to recognize that the geosynthetic tests are performed in the idealized laboratory conditions and therefore represent the maximum possible values. Thus, most laboratory test values can not be used directly. They must be modified by the in situ conditions. Recommended reduction factors for different applications are given in Table 2.

Table 2. Recommended reduction values for the use in strength related problems

Application	Range of reduction factors			
	Installation damage	Creep	Chemical degradation	Biological degradation
- separation	1.1 – 2.5	1.5 – 2.5	1.0 – 1.5	1.0 – 1.2
- unpaved roads	1.1 – 2.0	1.5 – 2.5	1.0 – 1.5	1.0 – 1.2
- walls	1.1 – 2.0	2.0 – 4.0	1.0 – 1.5	1.0 – 1.3
- embankments	1.1 – 2.0	2.0 – 3.5	1.0 – 1.5	1.0 – 1.3
- bearing capacity	1.1 – 2.0	2.0 – 4.0	1.0 – 1.5	1.0 – 1.3
- slope stabilization	1.1 – 1.5	2.0 – 3.0	1.0 – 1.5	1.0 – 1.3

1.3.6 Design for separation

1.3.6.1 Overview of Applications

The primary functions of geosythetics for separation are to prevent the mixing of two dissimilar materials. Separation can take place between different layers in road construction:

- between a fine grained subgrade and granular fill
- between a fine grained embankment and granular subbase
- between old and newly constructed base layer

Geosynthetics for separations prevent penetrations of stone aggregate into the subgrade (localized bearing failure as well as intrusions of fine grained soils up into the granular layers). Separation geosynthetics are important to maintain the design thickness, homogeneity, stability and load carrying capacity of the base and other aggregate layers.

The secondary functions of separation geosynthetics are filtration and drainage. The most common geosynthetic used for separation is geotextile.

1.3.6.2 Subgrade conditions in which geosynthetics are useful

The following subgrade conditions are considered to be the most appropriate for the use of geosynthetics in roadway construction

- poor soils: clayey gravel, clayey sand, silts, clays, organic soils and peat: GC, SC, ML, MH, CL, CH, OL, OH and Pt
- low undrained shear strength: $\tau_u = c_u < 90$ kPa; CBR < 3 % and Ms < 45 MPa
- high water table
- highly sensitive materials

Under these conditions geosynthetics function primarily as separators and filters to stabilize the subgrade, improving construction conditions and allowing long–term strength improvements in the subgrade.

The above mentioned conditions indicate the undrained shear strength $\tau_u = c_u < 90$ kPa as limit value for effective use of geosynthetics. From geotechnical point of view, soils with undrained strength higher than 90 kPa are considered to be stiff clays and generally quite good as foundation material. Nevertheless, some preconsolidated materials are very sensitive, prone to swelling and softening. In these cases, the use of geosynthetics as separators seems to be effective and reasonable.

1.3.6.3 Selection of geosynthetics for separation

1.3.6.3.1 General

Selection of geosynthetics for separation depends on:

- bearing capacity of subgrade
- type of fill material
- traffic load

The subgrade is natural geological soils or fills. Based on the bearing capacity, subgrades are divided into the following classes (Table 3):

Table 3: Subgrade classes as a function of bearing capacity

Bearing capacity	Class*	CBR (%)	E_{v2} (MN/m^2)
- very low	S_0	≤ 3	≤ 10
- low	S_1	3 - 5	10 - 20
- medium	S_2	5 - 10	20 - 60
- high	S_3	10 - 15	60 - 80

** When the bearing capacity conform to class S_3 or higher, geosynthetic as a separator is normally not necessary in inert soils. In the case of its use, the properties are the same as for class S_2.*

Fill materials are classified according to the grain size distributions and shapes into three classes

- class A: material with round or spherical grains, diameter > 150 mm:
- class B: material with sharp grains, diameter < 150 mm. Crushed stones, crushed gravel
- class C: other materials – different heterogeneous mixtures of natural soils, debris, alternative materials etc.

Traffic loads during the construction are divided into two classes:

- < 500 MN in
- > 500 MN.

1.3.6.3.2 Determining geosynthetic properties for separation

To determine the properties of separation geosynthetic, the following procedure is recommended

- Determine the type and properties of subgrade
- Determine the type of fill
- Determine:
- Traffic loading anticipated during the construction period
- Mechanical properties of the geosynthetic
- Hydraulic properties of the geosynthetic
- Specify the construction conditions

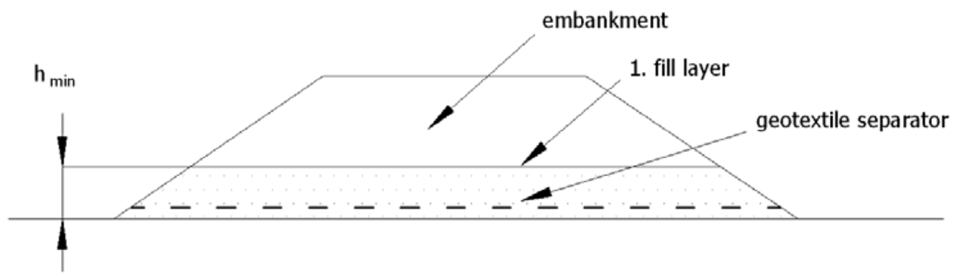


Figure 1: Geotextile separator beneath permeable granular layer (h_{min} is minimum required thickness for granular layer)

1.3.6.3.3 Determination of required aggregate thickness

The required aggregate thickness (h_{min}) depends on the bearing capacity of subgrade, i.e.:

- Subgrade class S_0 : h_{min} = 50 cm
- Subgrade class S_1 : h_{min} = 40 cm
- Subgrade class S_2 : h_{min} = 30 cm

1.3.6.3.4 Mechanical properties of the geosynthetic for separation

Minimum mechanical properties required for separation are given in Table 4 as a minimum required tensile strength (T_{min}) at minimum elongation $\varepsilon_{min} \geq 30$ %. In the case of geosynthetics with $\varepsilon_{min} \leq 30$ %, the requirements are given also as a minimum product $(T \times \varepsilon)_{min}$, expressed as a $(T \times \varepsilon)_{min} \geq T_{min} \times 30$ (kN/m.%).

Table 4. Requirements of mechanical properties. Tensile strength and elongations are determined according to EN ISO 10319

Subgrade	Minimum aggregate cover thickness	Mechanical properties	Traffic load during the construction time					
			< 500 MN			> 500 MN		
			Type of fill material					
			A	B	C	A	B	C
So	0,5 m	T_{min} (kN/m)	12	14	16	14	16	18
		$(T \times \varepsilon)_{min}$ (kN/m.%)	360	420	480	420	480	540
S1	0,4 m	T_{min} (kN/m)	10	12	14	12	14	16
		$(T \times \varepsilon)_{min}$ (kN/m.%)	300	360	420	360	420	480
S2	0,3 m	T_{min} (kN/m)	6	8	10	8	10	12
		$(T \times \varepsilon)_{min}$ (kN/m.%)	180	240	300	240	300	360

For the above given fill materials, the geosynthetic must comply also with the requirements, given as a cone drop test according to EN 918. Allowed diameters of the openings, determined by the test are:

- For fill material type A: O_d < 35 mm,
- For fill material type B: O_d < 30 mm,
- For fill material type C: O_d < 25 mm.

The static penetration test (CBR) according to EN ISO 12 236 can also be used as an alternative test method. Minimum required forces for penetration are:

- For fill material type A: $F_p > 1500$ N
- For fill material type B: $F_p > 2000$ N
- For fill material type C: $F_p > 2500$ N

1.3.6.3.5 Hydraulic properties for geosynthetics for separation

The secondary function of separation geosynthetics is filtration. Minimum hydraulic properties, required for separation geosynthetics are given in Table 5.

When the primary functions of geosythetic are separation and filtration, the geosynthetic must fulfil the requirements for mechanical properties given in Table 4, and the requirements for hydraulic properties, given in 1.3.6.3.5

Table 5: Requirements of hydraulic properties for separation geosynthetics

Subgrade material	Classification USCS	Characteristic openings O_{90} (mm) (accord. EN 12956)	Hydraulic conductivity k_G (m/s)* (accord. E – DIN 60500 – 4)
- sand	SW, SP	$0{,}05 < O_{90} < 0{,}5$	10^{-4}
- silt and silty soils	ML, GM, SM, GM-ML, SM-ML,GM-GC,SM-SC	$0{,}05 < O_{90} < 0{,}2$	10^{-5}
- clay and clayey soils	GC, SC,CL-ML,CL,GC-CH, SC-CH,CH	$0{,}05 < O_{90} < 0{,}5$	10^{-6}
- organic soils	OL, OH, Pt	$0{,}05 < O_{90} < 0{,}5$	10^{-4}

*k_G *means the minimum hydraulic conductivity, determined at the effective load of fill material. Common values are determined at 20 kN/m² and 200 kN/m². When the height of the fill is lower then 2 m, it is recommended to use the value, determined at 20 kN/m² and for higher fills values, determined at 200 kN/m².*

EN ISO 11058 defines the procedure for testing the hydraulic conductivity perpendicular to the surface (plane) of geosynthetic. The test result is given as a velocity index VI_{H50} (m/s). Due to the same measured units the velocity index is sometimes misunderstood as a hydraulic conductivity. It is recommended to require velocity index: $VI_{H50} > 3$ mm/s or $> 3 \times 10^{-3}$ m/s for separation geotextiles.

1.3.6.3.6 Installation and construction requirements

The geosynthetic should be laid smooth, on the prepared subgrade, from one edge to the other, using machines or by hand. The requirements, given in Table 4, are valuable for both types of installation.

Construction vehicles shall not be allowed directly on the geosynthetic. The aggregate (cover) layer should be placed such to keep at least the minimum specified thickness (Table 3) between the geosynthetic and equipment tyres at all times.

The width of geosynthetics is limited. Adjacent geosynthetic rolls should be overlapped, sewn or joined, as required in the plans. Overlaps should be in the directions as shown on the plans. Geotextiles are usually overlapped.

Overlap requirements depend on the bearing capacity and the smoothness of the subgrade. For subgrades (S_2, S_3) the minimum required overlapping is 30 cm. For lower class subgrade, the required overlapping is 50 cm. When the geosynthetic is installed under the ground water level, the minimum required overlapping is 100 cm.

1.3.7 Design for filtration

1.3.7.1 Overview of applications

The primary function of geosynthetics for filtration is prevention of washing out soil particles in to drainage system and prevention against internal erosion. The geosynthetic acts as a filter, when the main water flow is perpendicular to the geosynthetic plane (surface). The process of restraining soil particles subjected to hydraulic forces while allowing the passage of water into or across a geosynthetic is called filter stability. To ensure long life and efficiency, the geosynthetic filters should be properly designed, with respect to the properties of soils and geosynthetics.

Designing geosynthetics for filtration is essentially the same as designing graded granular filters. The basic information for the design is connected to the hydraulic conductivity and grain size distribution of protective soil. Geosynthetic must retain the soil, while allowing water to pass throughout the life of the structure. Water pressure is not allowed to increase behind the filter.

To be sure the geosynthetic will survive the construction process, specific geosynthetic strength and endurance properties are required for filtration. The most commonly used geosynthetic for filtration is geotextile.

1.3.7.2 Geotextile filter design

1.3.7.2.1 General

Geotextile filter design criteria are based on the following:

- Retention for laminar and turbulent flow
- permeability and permitivity
- clogging resistance
- endurance and strength

The level of design required depends on critical nature of the projects and the severity of the hydraulic and soil conditions. For conventional use in road construction projects, the designing procedure is described in 1.3.8.2.2. For critical projects, conservative design is recommended.

Selection of geotextile for filtration depends on:

- type and properties of protected soil
- water velocity or water pressure
- type of drainage layer
- installation conditions.

1.3.7.2.2 Determining geosynthetic properties for filtration

To determine the properties of geosynthetics for filtration, it is important to:

- Estimate the critical nature and site conditions. It is important to recognize, whether the initial washing is allowed or not
- Evaluate the properties of soil
- Evaluate the properties of drainage
- Estimate the conditions for maintenance

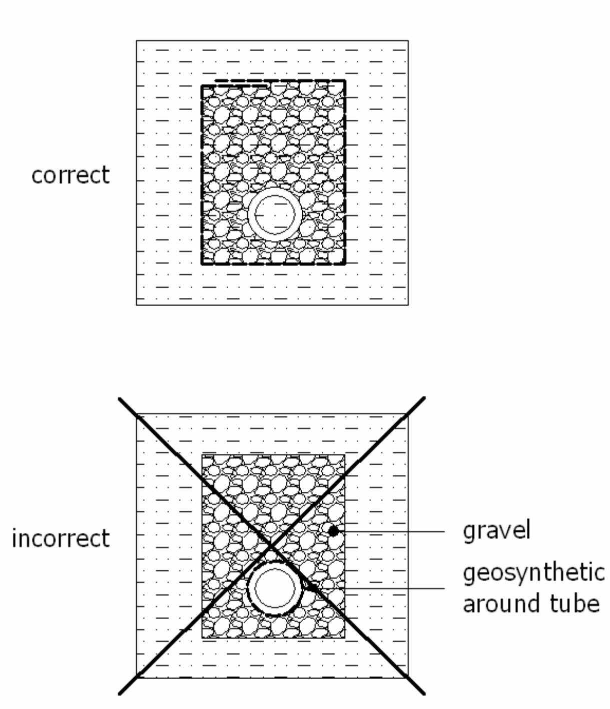


Figure 2. An example of correct and incorrect use of geosynthetic for filtration

1.3.7.2.3 Hydraulic properties

Table 6 gives the minimum required properties for ensuring filter stability in the case that initial washing of soil is allowed, and Table 7 in the case that initial washing is not allowed.

For non cohesive soils with d_{85} < 0.05 mm it is recommended to provide an additional level of protective measures to ensure filter stability.

For heterogeneous and layered soil, the design of characteristic openings depends on the properties of fine grained soil, while the design of hydraulic properties depends on coarse grained soils.

Table 6. Minimum hydraulic requirements for geosynthetics for the filtration (initial washing is allowed)

Hydraulic conductivity normal to the plane k_g (m/s)	Opening size O_{90} (mm)
k_g higher then 10 k_{soil}, recommended 100 k_{soil}	$O_{90} \le d_{85}$ $O_{90} \ge 0.05$ mm $O_{90} \ge 4x\ d_{15}$ *

**For silty gravel soil, prone to internal mass transportation and culmination*

Table 7. Minimum hydraulic requirements for geosynthetic for the filtration (initial washing is not allowed)

Grain size distribution	Hydraulic conductivity normal to the plane k_g (m/s)	Opening size O_{90} (mm)
$d_{50} \leq 0.06$ mm	k_g higher then 10 k_{soil}, recommended 100 k_{soil}	$O_{90} \leq d_{85}$ $O_{90} \geq 0.05$ mm
$d_{50} > 0.06$ mm	k_g higher than 10 k_{soil}, recommended 100 k_{soil}	$O_{90} \leq d_{85}$ or $O_{90} \leq 5\ d_{10} \times (Cu)^{1/2}$ $O_{90} \geq 0.05$ mm

$*k_G$ *means the minimum hydraulic conductivity, determined at the effective load of fill material. Common values are determined at 20 kN/m^2 and 200 kN/m^2. When the height of the fill is lower than 2 m, it is recommended to use the value, determined at 20 kN/m^2 and for higher fills values, determined at 200 kN/m^2.*

EN ISO 11058 determines the procedure for testing the hydraulic conductivity perpendicular to the surface (plane) of geosynthetic. The test result is given as a velocity index VI_{H50} (m/s). The velocity index is sometime misunderstood for hydraulic conductivity due to the same measured units. It is recommended to require velocity index: $VI_{H50} > 3$ mm/s or $> 3 \times 10^{-3}$ m/s for filter geotextiles.

1.3.7.2.4 Mechanical properties

Geosynthetic for filtration must have adequate mechanical properties to survive the time of installing and to keep the designed life time. Grain size and the shape of drainage aggregate are decisive for the design.

Minimum mechanical properties required for filtration are given in Table 8 as a minimum required tensile strength (T_{min}) at minimum elongation $\varepsilon_{min} \geq 30$ %. In the case of geosynthetics with $\varepsilon_{min} \leq 30$ %, the requirements are given also as a minimum product $(T \times \varepsilon)_{min}$, expressed as a $(T \times \varepsilon)_{min} \geq T_{min} \times 30$ (kN/m.%).

Table 8. Minimum requirements for mechanical properties for geosynthetic used as filters

Drainage material (class)	Minimum tensile strength (kN/m)	Minimum product $(T\times\varepsilon)_{min}$ (kN/m x %)	Diameter of the hole O_d (mm)
- round – (A)	6	180	40
- crushed– (B)	8	240	35

Drainage materials are classified based on grain size and shape of grain into two classes:

- class A: round materials
  - gravel d < 63 mm
  - gravel and boulder: d < 150 mm
- class B: crushed and sharp materials
  - crushed stones d < 16 mm
  - crushed stones and blocks d < 125 mm
  - crushed blocks d < 150 mm

1.3.7.2.5 Installation and construction properties

The surface on which the geotextile is to be placed should be excavated to design grade, to provide a smooth, graded surface. Care should be taken to place the geosynthetic in intimate contact with the soil so that no void spaces occur behind it. It is important that geosynthetic has proper elongation properties allowing proper installation.

Adjacent geosynthetic rolls should be overlapped at least 30 cm in both, transverse and longitudinal directions. When the geosynthetics for filtration are used under the stone blocks or in more aggressive environment, the mechanical properties must be defined, using more conservative method, and they should be specified in the project.

1.3.8 Design for drainage

1.3.8.1 Overview of Applications

The primary function of geosynthetics for drainage is to collect water from the soil and to draw it off from the influential area. With its action, the drainage geosynthetics prevent the rise of pore water pressure in the surrounding soil.

The geosynthetic acts as a drainage, when the main water flow is in–plane of the material. The most commonly used geosynthetics for drainage are geocomposites and geotextiles.

In road construction applications, the geosynthetics for drainage are used in the following structures:

- Edge drains in roads,
- Drains on unstable slopes,
- Drains behind the footings and retaining walls,
- Drains behind the underground structures, and
- Wick (strip) drains for consolidation on soft soils,

The drainage geosynthetics can be installed in homogenous soil, at the contact between two different soils or at the contact between the impermeable and very permeable materials.

Water running from the surrounding soils towards the drainage must be transported out with the smallest possible pressure loss. Therefore, the adequate in–plane flow capacity is required for drainage geosynthetics. The in-plane water flow capacity of a product, expressed at a hydraulic gradient of 1, is transmissivity θ (m^2/s).

It should be considered that the in–plane water flow capacity of geosynthetics is relatively small (around $2 \times 10^{-5} m^3/s/m'$ under the pressure of 12 kPa), compared to the conventional 0.15 – 0.30 m thick aggregate drainage layer. Thus, when designing the drainage geosynthetics, it is important to calculate the quantity of inflow water and the equivalency of water flow capacity of geosynthetics and conventional aggregate drainage.

The drainage geosynthetic must also act as a filter and must also fulfil the criteria for filter stability.

Strength properties for the underground drainage geosynthetics depend on the survivability level required to resist installation stresses.

Due to the temperature and ground water pressure changes in the drainage surroundings, in some geological structures geosynthetic can become chemically clogged by iron or carbonate precipitates or biologically clogged by algaes. Excessive chemical and biological clogging can influence drain performance. In such cases also this point of view must be considered, taking into account the life time of the structure and cleaning and maintenance possibilities.

1.3.8.2 Geosynthetic drainage design

1.3.8.2.1 General

For drainage geosynthetic design, there are three aspects of design:

- Adequate flow capacity - inflow and outflow capacity under design surcharge within the design life time
- proper soil retention and long term soil–to–geosynthetic flow equilibrium
- functioning within the designed structure (installation, general safety).

Selection of geotextile for drainage depends on:

- type and properties of surrounding soil
- water velocity or water pressure
- type of drainage layer and design life time
- mechanical properties regarding the type of function and installation conditions.

Traffic loads represent severe conditions for drainage geosynthetics. The failure of drainage geosynthetic in road construction can affect the failure of road. When designing the drainage geosynthetic, it is important to calculate all different stresses that can occur and influence its safe use.

1.3.8.2.2 Determining geosynthetic properties for drainage

To determine the properties of geosynthetic for drainage, it is important to define:

- critical nature of the structure and function of the drainage in the structure
- soil properties and the expected water flow
- installation conditions

1.3.8.2.3 Hydraulic properties

Minimum required hydraulic properties are given in Table 9. When the nature of the structure is estimated to be critical, which means that the effectiveness of the drainage influences the structure safety, the hydraulic properties must be defined based on the calculations during the structure design. For such cases, this technical specifications are not relevant.

The selection of drainage geosynthetics depends on the estimation of expected maximum water to be drained out. The runoff flow rate is evaluated by using Darcy's law :

$$Q = k_p \times i \times A = k_p \times i \times B \times d \qquad (m^3/s)$$

where:

k_p = in plane water conductivity of geosynthetic (m/s)

i = hydraulic gradient ($\Delta h/\Delta L$)

A = surface area of geosynthetic (m^2)

B = width (m)

d = thickness (m)

Table 9: Minimum hydraulic requirements for drainage

Grain size	Hydraulic conductivity k_g (m/s)	Pore openings O_{90} (mm)	Transmissivity θ (m^2/s)
$d_{50} \leq 0.06$ mm	$k_g > 10\ k_{soil}$	$O_{90} \leq d_{85}$ $O_{90} \geq 0.05$ mm	$\theta > (F\ .Q_{max})/(B.i)$ - F - factor of safety F = 5 (single-layer geotextiles) F = 2 (multiple-layer geotextile or geocomposite Q_{max} - max. water flow rate(m^3/s)
$d_{50} > 0.06$ mm	$k_g > 10\ k_{soil}$	$O_{90} \leq d_{85}$ or $O_{90} \leq 5\ d_{10} \times (C_u)^{1/2}$ $O_{90} \geq 0.05$ mm	

When deformable geosynthetics are used, the effect of overburden and creep on the effective thickness and time dependent flow rate capacity must be calculated.

1.3.8.2.4 Mechanical properties for drainage geosynthetics

Geosynthetic for drainage must have adequate mechanical properties to survive the time of installing and to keep the designed life time. Minimum requirements are given in table 10.

Table 10. Minimum mechanical requirements for drainage geosynthetics in longitudinal and transverse direction

Type of use	Required properties	Recommended value
- wall drainage (concrete wall/drainage/ soil	Tensile strength Elongation	min. 8 kN/m min. 10 %
- vertical trench drainage	Tensile strength Elongation	min. 8 kN/m min. 20 %
- horizontal drainage (blanket)	Tensile strength Elongation	Depending on secondary function, the value from Table 3 or 7 are recommended

1.3.8.2.5 Installation and construction properties

It is important to ensure free water flow through the drainage geosynthetics. Care should be taken at junctions and overlapping in the flow directions and at all connections of drainage geosynthetics to the shafts or trenches. All details must be defined during the design process and should be given in the project.

1.3.9 Reinforced embankments on soft foundations

1.3.9.1 Background

Embankments constructed on soft foundation soils have the tendency to spread laterally because of horizontal earth pressure acting within the embankment. These earth pressures cause horizontal shear stresses at the base of the embankment which must be resisted by the foundation soil. If the foundation soil does not have adequate shear resistance, failure can result.

Properly designed horizontal layers of geosynthetics can provide reinforcement which increases stability and prevents such failures. The reinforcement can also reduce horizontal and vertical displacement of the underlying soil.

The use of geosynthetic will not reduce the magnitude of consolidation and secondary settlements.

The use of geosynthetics in embankment construction on soft ground may contribute to:

- an increase in the design factor of safety
- an increase in the height of the embankment
- a reduction in embankment displacement during construction
- an improvement in embankment performance due to increased uniformity of post construction settlement.

The most commonly used geosynthetics for reinforcement are: geotextiles, geogrids and geocomposites.

This section assumes that all the common foundation treatment alternatives for the stabilization of embankments on soft foundation soil have been considered during the preliminary phase.

1.3.9.2 Overview of applications

Reinforced geosynthetics for embankments on soft foundation soils are commonly used in two different types of applications:

- For embankments, constructed on very weak, very soft and saturated soil with fairly homogeneous properties and stratigraphy
- for embankments, constructed over locally weak foundation soil, old filled creeks, cracks, carstic holes. In these cases, the geosynthetic acts as a bridge over the weak zones.

In the first case, the reinforcement is usually placed with its strong direction perpendicular to the centre line of the embankments. Additional reinforcement with its strong direction oriented parallel to the centreline may also be required at the ends of the embankment.

In the second case, the reinforcement may be required in different directions.

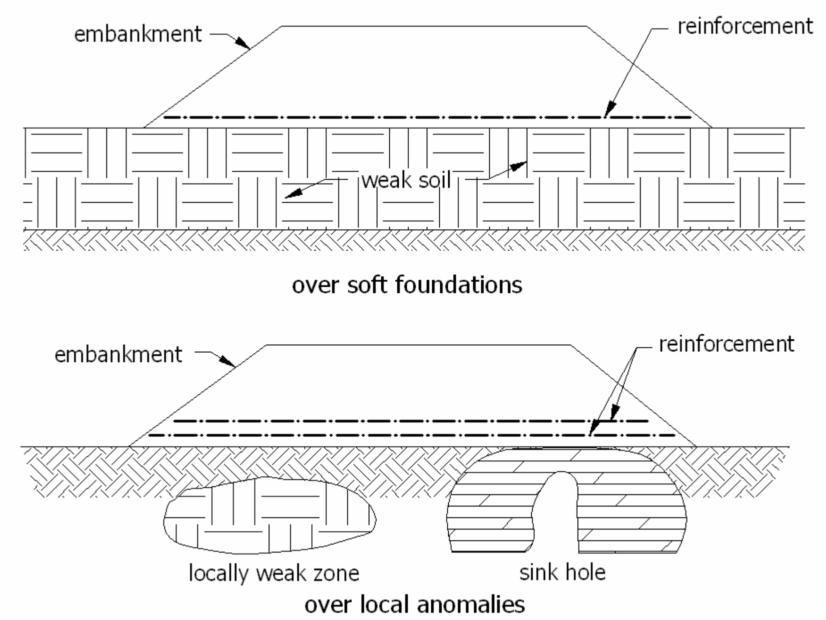


Figure 3: Reinforced embankment applications.Typical cases of using reinforcing geosynthetic below embankments on soft foundation soil

1.3.9.3 Design guidelines for reinforced embankments on soft soils

1.3.9.3.1 General

The basic design approach is to design against failure. The three possible modes of failure indicate the required types of stability analysis. Because the most critical condition for embankment stability is at the end of the construction, the geosynthetic has to function until the foundation soils reach sufficient strength. The design requires also the calculation of settlement and the effect of settlement on the creep behaviour of the geosynthetic.

The calculations required for stability and settlement utilize conventional geotechnical design procedures, modified only for the presence of the reinforcement geosynthetic.

The stability of an embankment over soft soil is usual determined by the total stress method, which means conservative approach. It is possible to calculate stability in terms of the effective stresses. The second approach requires an accurate estimation of the pore pressures to be made during the project design phase, in combination with high quality samples for K_o CU triaxial laboratory tests as well as the accurate field measurements of pore pressures during the construction to control the rate of consolidation.

1.3.9.3.2 Design considerations and steps

The following information should be acquired, established and analysed at the design:

- The definition of embankment dimensions and loading conditions
- The establishment of soil profile, engineering properties of soils and fill materials
- The establishment of appropriate factors of safety
- Checking of bearing capacity
- Checking of global and internal stability
- The establishment of geosynthetics properties

1.3.9.3.3 Definion of embankment dimensions and loading conditions

It is important to determine the following conditions:

- Embankment dimensions:
- Embankment height
- Embankment length
- Width of the crest
- Side slopes
- External loads:
- surcharges
- temporary (traffic) loads
- dynamic loads
- Environmental considerations
- Frost action
- Shrinkage and swelling
- Drainage, erosion ...
- Embankment construction rate :
- Project constraints
- Anticipated or planned rate of construction

1.3.9.3.4 Determination of geological and geotechnical conditions and the properties of fill material

It is important to determine the following geological and geotechnical conditions and properties:

- typical cross-section of the ground and engineering properties of soils in the foundation ground and in the embankment, consisting of:
- stratigraphy and ground cross-section
- underground water table
- undrained shear strength
- drained shear strength
- consolidation parameters
- chemical and biological actions
- variations of properties in the vertical and horizontal direction
- fill material type and classification
- compaction level and the ration water content-density
- strength properties of the condensed embankment
- chemical and biological properties that may influence on the goesynthetic

In this respect, the classical procedures for the selection and installation of the fill material shall be considered, except for the first layer above the reinforcement geosynthetic, which is always from well permeable granulated material. With such choice of material large friction resistance of the contact geosynthetic-embankment and the possibility of rapid draining and decrease of excessive pore pressures must be ensured for the first fill layer.

1.3.9.3.5 Determination of minimum factors of safety

When analysing stability, partial safety quotients shall be used according to the principle of limit state methods, i.e.:

- for effective shear angle $\gamma_{\varphi}=1{,}25$
- for effective cohesion $\gamma_{c}=1{,}25$
- for undrained shear strength $\gamma_{cu}=1{,}40$
- for uniaxial compressive strength $\gamma_{qu}=1{,}40$
- for weight density of the ground $\gamma_{\gamma}=1{,}00$[1)]
- for permanent load at the ground surface $\gamma_{G}=1{,}35$
- for temporary load at the ground surface $\gamma_{Q}=1{,}50$
- for limit bearing capacity of the reinforcement 1,15
- to shear at the reinforcement 1,4 – 1,5
- for the analysis method $\gamma_{M}=1{,}00$ (or according to the user's judgement)

[1)] Any possible unreliability at determining volume weight of the ground is considered by repeating the analysis for the smallest and the largest volume weight.

When planning reinforcement geosynthetics, according to experiences for a concrete situation other higher factors may be used.

1.3.9.3.6 Checking of bearing capacity

When the thickness of soft soil is larger than the embankment width, for the approximate calculation use classical bearing capacity theory:

$$\gamma_N\, h = q_f < (2+\pi)c_u < c_u\, N_c$$

where:

c_u – undrained shear strength

N_c – Bearing capacity factors

h - embankment height

This approach underestimates the bearing capacity of reinforced embankments. It assumes that geosynthetic does not increase the overall bearing capacity of the foundation soil. If the foundation soil can not support the weight of embankment, the embankment can not be built.

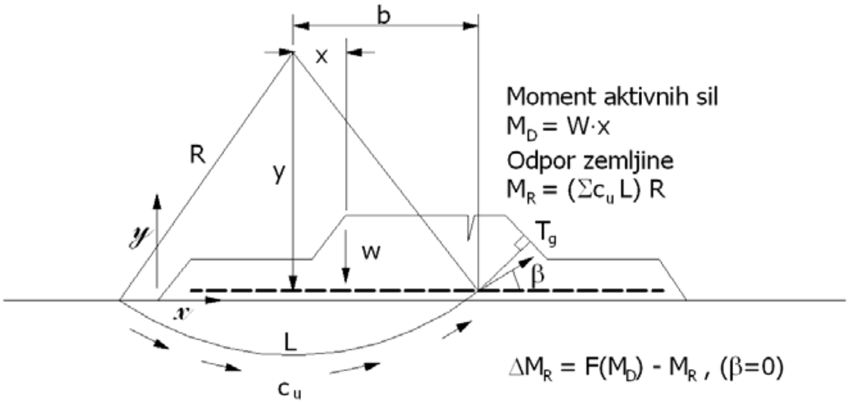
The bearing capacity of the foundation soil can be calculated using other conventional soil mechanics methods, which use limiting equilibrium type analyses for strip footings, assuming logarithmic spiral failure surfaces on an infinitely deep foundation. If the thickness of the underlying soft soil is relatively small, compared to the height of the embankment, these analyses are not appropriate. In this case, high lateral stresses in the confined soft stratum beneath the embankment could lead to a lateral squeeze – type failure. In this case, approaches discussed by (Holtz and Giroud, Rowe in Siderman etc.) are appropriate. It must be assumed that those methods are only approximate and have not yet been accepted by geotechnical engineers.

1.3.9.3.7 Checking of rotational shear stability

Rotational shear stability is appropriate only in homogeneous foundation soils. Rotational slip surface analysis on the non-reinforced embankment and foundation analysis must be done at the beginning to determine the critical failure surface and the factor of safety against local shear instability. If the calculated factor of safety is greater than the minimum required, reinforcement is not needed.

If the calculated factor of safety is lower than the minimum required, then calculate the required reinforcement strength of the geosynthetic material (T) to provide the adequate factor of safety (figure 4).

$$T = \frac{F(M_D) - M_R}{R\cos(\theta}$$



Moment aktivnih sil = moment of active forces

Odpor zemljine = soil resistance moment

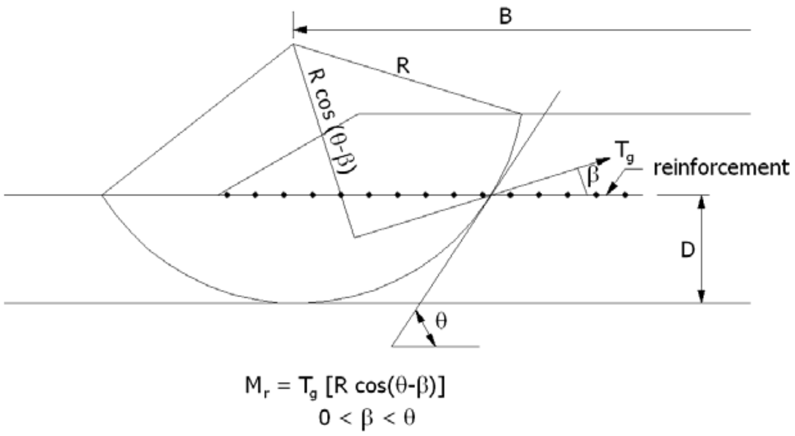


Figure 4: Reinforcement required to provide rotational stability. Above for the case of β = 0 (Christopher and Holtz, 1985); below for the case if the geosynthetic material does not improve soil strength (Bonaparte and Christopher,1987).

1.3.9.3.8 Checking of lateral spreading stability

Lateral spreading or sliding wedge stability analyses can be performed according to Figure 5. If the calculated factor of safety is greater than the minimum required, reinforcement is not needed for this failure possibility.

If the calculated factor of safety is inadequate, then determine the lateral spreading strength of reinforcement. At the and check also sliding above the reinforcement.

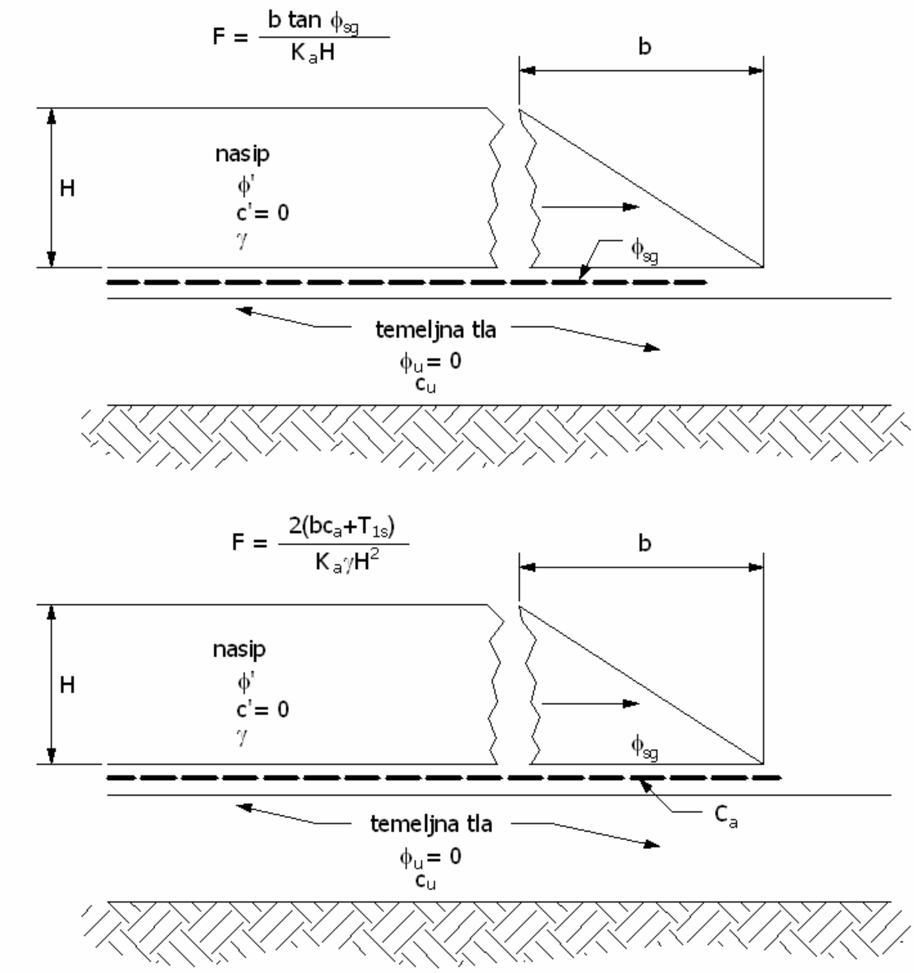


Figure 5: Reinforcement required to limit lateral embankment spreading

1.3.9.3.9 Establishment of the tolerable geosynthetic deformation requirements

Recommendations, based on the type of fill soil materials and for construction over peats are:

- Cohesionless soils : ε = 5 – 10 %
- Cohesive soils: ε = 2 %
- Peats: ε = 2 – 10 %

1.3.9.3.10 Determination of properties of geosynthetic

The requirements for geosynthetics depend on the results of calculations.

It is recommended to test the friction between the soil and geosynthetics for specific project. For preliminary design and in absence of specific geosynthetic data, friction can be estimated as: $\varphi_{sg} = 2/3\varphi$.

Stiffness of geosynthetics influences the installation condition and damages that may occur during construction. It is important to define the geosynthetics stiffness, considering the specific site conditions and past experiences. Construction on soft underground requires geosynthetics with high stiffness. It is important to consider all other factors (fill soils, installing technology etc.) which can influence the behaviour and endurance of geosynthetics.

1.3.9.3.11 Other considerations

It is important to define:

- Settlements and time of consolidation, using conventional geotechnical methods
- installation procedures
- methods of geotechnical monitoring and additional safety measures if necessary according to the results of field observation

1.3.10 Geosynthetics for reinforced slopes

1.3.10.1 Background and applications

Reinforced soil is a composite material, which combines the properties of two very different materials. Soil, which is relatively cheap and accessible, has quite good compression and shearing properties and very poor tensile properties. Geosynthetic is, compared to the soil, relatively expensive material with high tensile strength. Combining these two materials allows to combine good properties of both for engineering purposes.

Geosynthetics are primarily used as a slope reinforcement for the construction of slopes to angles steeper than those constructed with the fill material alone. Applications which highlight this kind of use are given in Figure 6. Reinforcement can be also used for:

- Construction of new embankments ,
- Widening of existing embankments
- Construction of alternatives to retaining walls
- Repair of failed slopes
- Noise barrier embankment
-

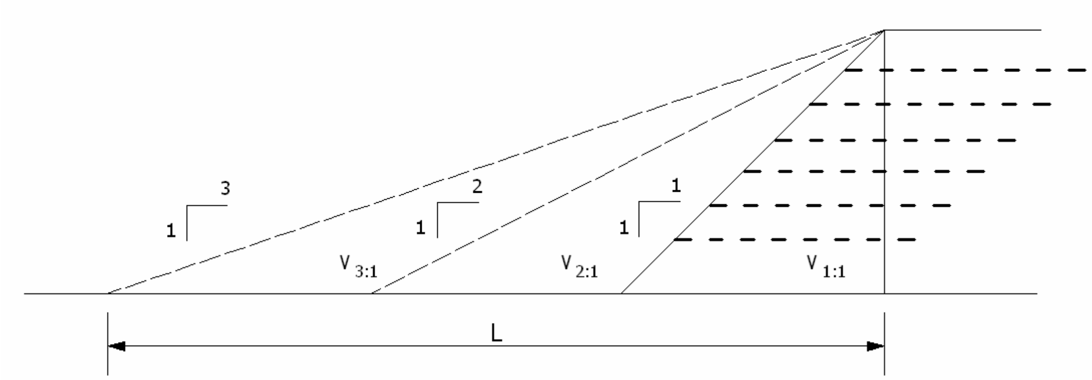


Figure 6. Reinforced soil slope – cost and space effective solution

Geosynthetics, used for slope reinforcement, can significantly influence the design by (Figure 7):

- reducing the volume of required fill,
- allowing the use of less quality fill,
- creating usable landscape at the crest or at the toe of the slope
- offering a cost effective alternative to retaining walls

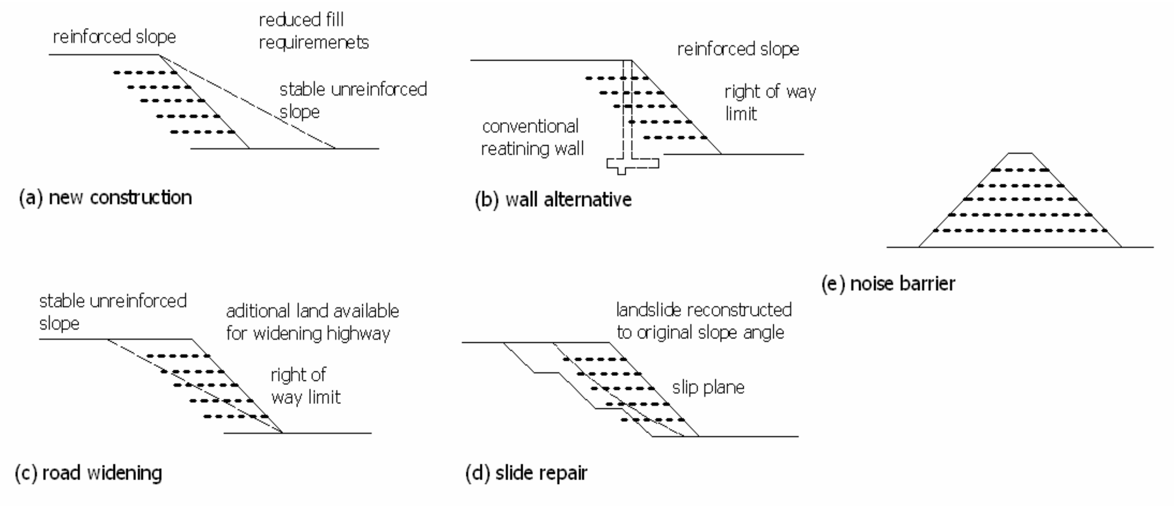


Figure 7. Applications of reinforced slopes

The design of reinforcement for safe, steep slopes requires rigorous analysis. The design of reinforcement fir these application is critical because reinforcement failures result in slope failure.

Reinforcement with geosynthetics is appropriate also for some other applications:

- to provide lateral resistance of the edges during compaction of a compacted fill slope
- to improve erosion and surface stability of the slope
- to accelerate the consolidation etc.

In the above mentioned cases special analyses are not required.

Other applications of reinforced soil slopes include

- increased height of the dams, when the bearing capacity of the foundation soil is not problematic
- embankment construction with wet, fine grained soils
- construction of permanent levees and temporary flood control structures
- steepening abutments and decreasing bridge spans
- temporary road widening for detours
- noise barriers

1.3.10.2 Design for reinforced slopes

The overall design requirements for reinforced slopes are similar to those for non-reinforced slopes: the factor of safety must be adequate for both the short term and long term conditions and for all possible modes of failure using one of the conventional methods.

Permanently reinforced slope (planned life cycle is longer than one to three years) can be considered uncritical when the presented factor of safety for the same geometry of non-reinforced slope is larger than F = 1.1; the reinforcement using geosynthetic can be used to increase this factor.

The reinforced soil shall be treated as critical in the following cases:

- if there is mobilised tension in the reinforcement for the designed life of the structure,
- if the failure of the geosynthetic results in the slope failure,
- if the failure of the slope presents threat to people and properties.

When analysing stability of reinforced slope, the following possible forms of stability failure shall be considered:

- internal: failure plane passes through the reinforcing elements
- external: failure surface passes behind and below the reinforced mass and
- compound: failure surface passes behind and through the reinforced part of the slope.

The largest angle of the slope built of non-homogeneous material without cohesion and without reinforcement is $\beta = \varphi$, where φ is the shear angle of the soil in the slope. When the slope is to be built from the same material in larger angle, additional resistance to maintain equilibrium in the soil shall be assured. The simplest way is to introduce additional horizontal resistance by laying geosynthetic in horizontal layers. In this way the shear resistance of the soil is improved. Additional force, needed to maintain equilibrium, is (Jewell,1991)

$T = 0.5\ K\ \gamma\ H^2$,

where :

H – slope height

K – coefficient of earth pressure, depending on the slope angle β*, strength parameters of the soil and the coefficient of pore pressure*

γ *- volume weight of the soil*

1.3.10.3 Planning soil reinforcements

When planning slope reinforcements, first the following is to be determined:

- slope geometry,
- external loads,
- geotechnical properties of the foundation ground and soil in the ground,
- height and pressures of underground water,
- global stability of non-reinforced slope,
- engineering properties of fill soil in the area and behind the area of reinforcement, and
- design parameters for reinforcement: selection of geosynthetic, its strength, rigidity and interaction properties with the soil.

When these data are acquired, the following shall be checked:

- number,
- vertical distribution and
- the necessary length of the reinforcement belts.

This can be done by using

- commercial software to check stability,
- analytical methods developed by different authors,
- finite element methods or
- diagrams for design, based on the determination of the earth pressure coefficient K and reinforcement length L based on the slope angle, shear angle of the soil and size of the pore pressure.

The usual geotechnical approach to check stability of reinforced slopes is to use the limit state method for the presumed potential rotational stability.

Factor of safety can be expressed as

$F = (M_R + \Sigma T_i\, y_i)/M_d = (\tau_f \,.\, L_{sp} \,.\, R + \Sigma T_i\, y)/(W.x + q.d)$

where:

M_R - moment provided by soil resistance

Md - moment of active forces

T_i - available reinforcement strength of the geosynthetic

y1 - distance

n - number of reinforcing layers

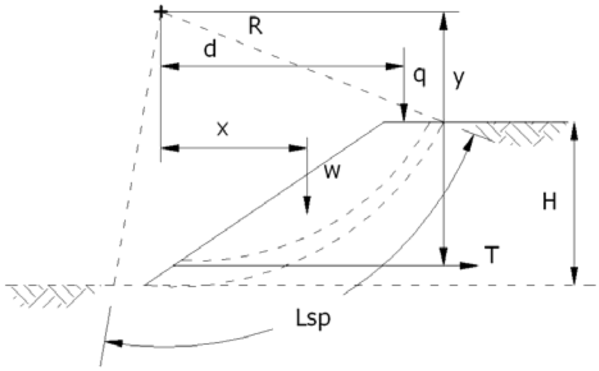


Figure 8: Rotational shear approach to determine required strength of reinforcement

The factor of safety for different cases of rotational stability shall be calculated with adequate software, which allows defining critical failure surfaces and the necessary reinforcement geometry.

Alternatively, internal stability of the reinforced slope, assured by geosynthetics, can be defined with the help of dimensioning diagrams developed by different authors (Jewell, Murray, Ruegger..).

The dimensioning diagrams of reinforced slopes with geogrids were defined by Jewell (Figures 9, 10 and11). By using these diagrams the following can be determined:

- value of the minimum necessary force of reinforcing belts K_{req},
- the necessary minimum value L/H to provide for general stability (L = belt length, H = slope height),
- the necessary minimum value L/H to provide for safety against slide,
- designed value of strength of the geosynthetic, which is

  $T_{design.} = T_{allow}./F$

  $T_{allow}. = T_{max}/F_{geosynthetic}$

  $F_{geosyntheti} = (F_{creep} \text{ x } F_{installation} \text{ x } F_{joints} \text{ x } F_{biol}.\text{x } F_{chem})$
- height of the fill layer of the soil or the number of reinforcing layers n,
- vertical spacing $S_v = T_{proj}/(K_{req} \,.\, \gamma \,.\, H)$, where H is the height of the reinforced slope, and
- areas of equivalent heights.

The use of diagrams in Figures 9 to 11 is valid for the following assumptions:

- the slope is uniform, inclined by 30 to 90°, the surface behind the slope is horizontal
- the slope is on flat foundation soil with adequate bearing capacity
- the fill material is homogeneous
- the strength properties of the fill material are given by parameters of drained state
- the pore pressures are expressed in the form of coefficient $R_u = u/(z \,.\, \gamma)$

- the loading at the crest is uniform
- the geosynthetic is laid continuously and in horizontal layers

The use of dimensioning diagrams is not allowed in the following cases:

- the slope is sunk
- at the crest of the slope point or line load is active
- dynamic loads
- strength properties of the fill material are given by parameters of undrained state
- discrete reinforcements with belts or rods

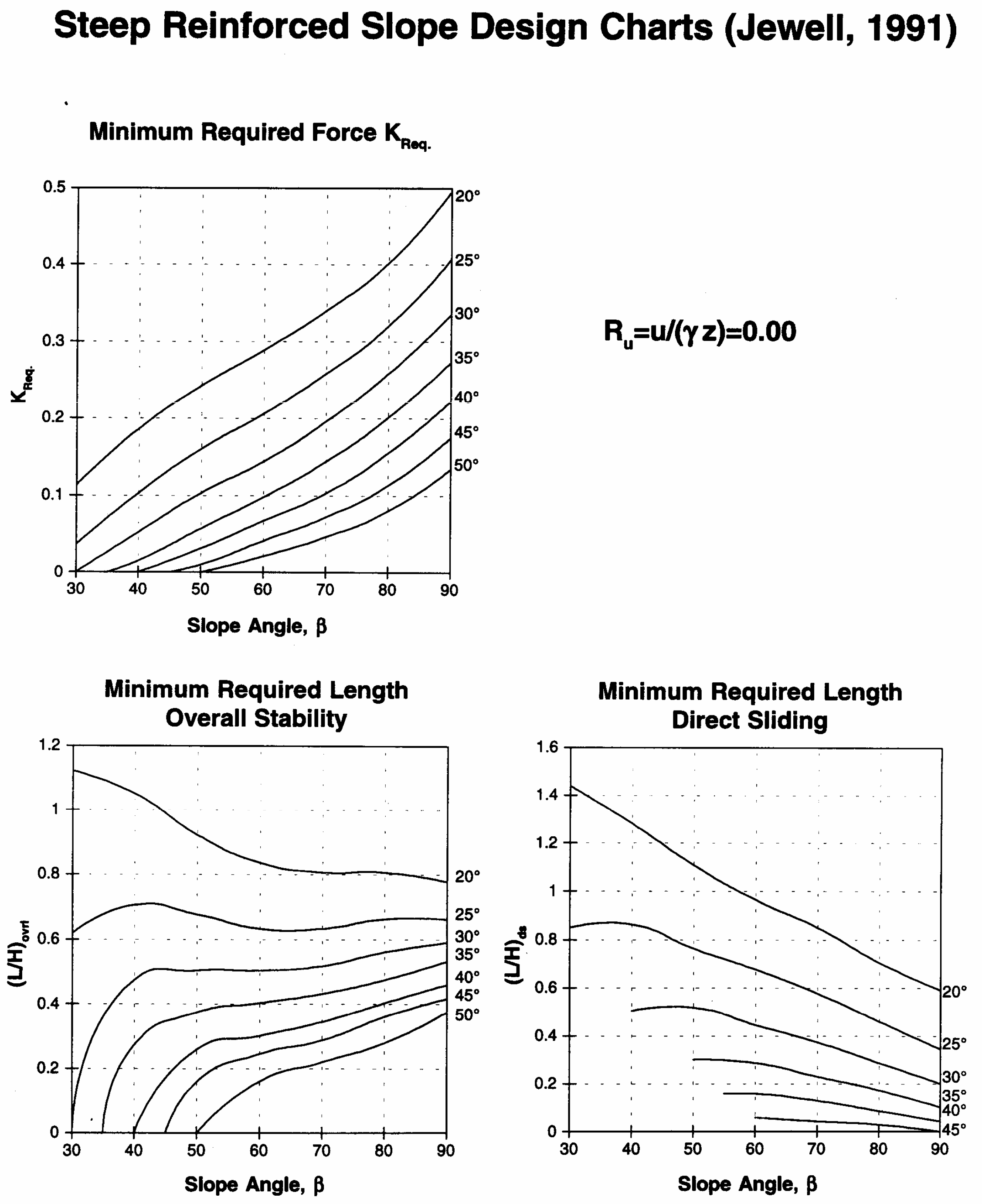


Figure 9: Diagrams to design reinforced slopes, R_u = 0

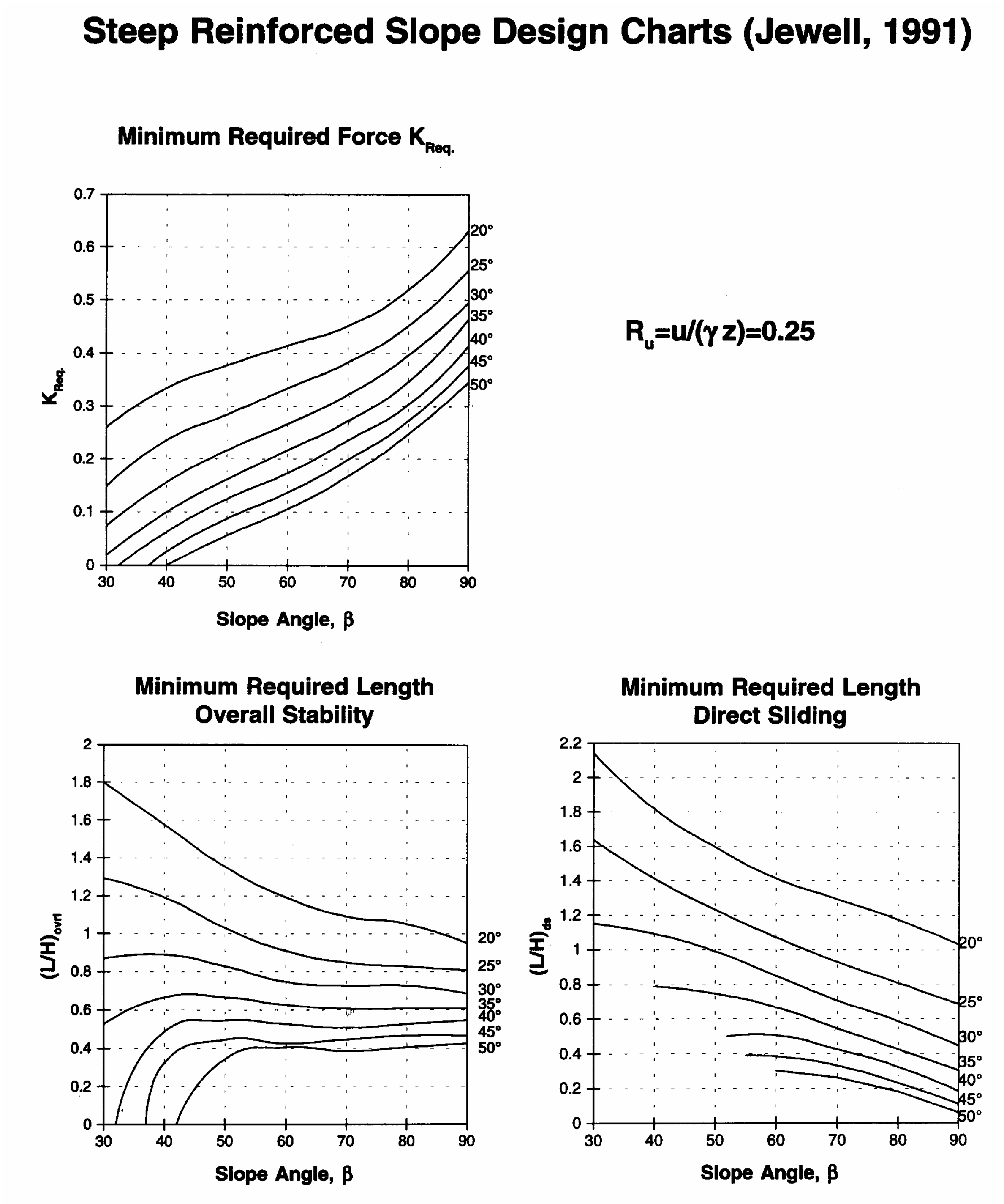


Figure 10: Diagrams to design reinforced slopes, Ru = 0.25

Steep Reinforced Slope Design Charts (Jewell, 1991)

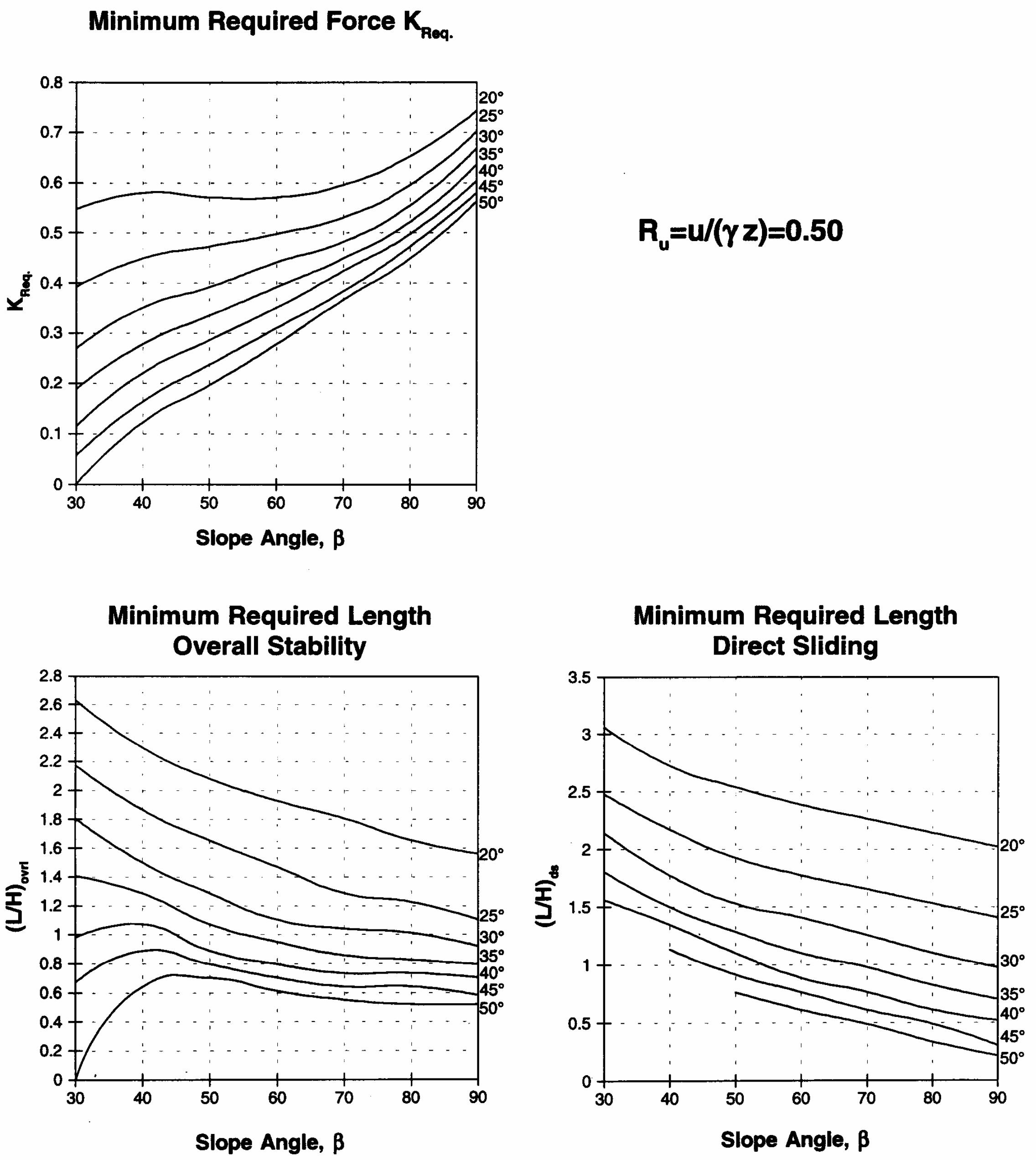


Figure 11: Diagram to design reinforced slopes, Ru = 0.50

1.3.10.4 Installation procedures

Reinforcement layers are easily incorporated between the compacted lifts of fill. Therefore, construction of reinforced slopes is similar to normal embankment construction up to slope angles 1:1. The design should define following requirements:

- For site preparation (clearing, grubbing, levelling and compaction of subgrade)
- for geosynthetic placing and overlapping

- for fill lift thickness and degree of compaction. It is recommended the compaction degree is ≥ 95 % at optimum water content variation up to 2 m.-% of optimum.
- for compaction control

For slopes with angle > 45^0 or 1H:1V, a facing treatment should be applied to prevent erosion during and after construction. In this case, the reinforcement is simply extended to the face as shown in Figure 12. Every lift or every other lift should be wrapped, with the thickness of the lift no greater than 40 cm.

For very steep slopes, form work may be required to support the face during construction, especially if lift thickness of 0. 5 – 0.6 m is used. When geogrids are used, a fine mesh screen or geotextile may be required at the face to retain backfill materials.

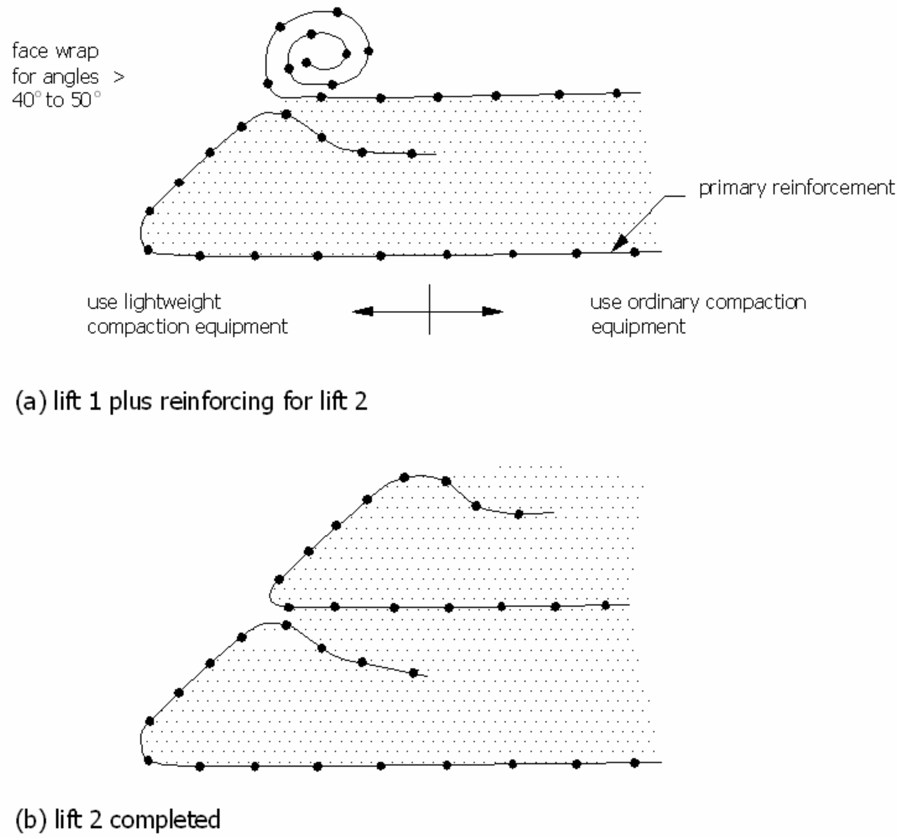


Figure 12. Construction of reinforced slopes - face wrappings for angles > 40 – 50^0

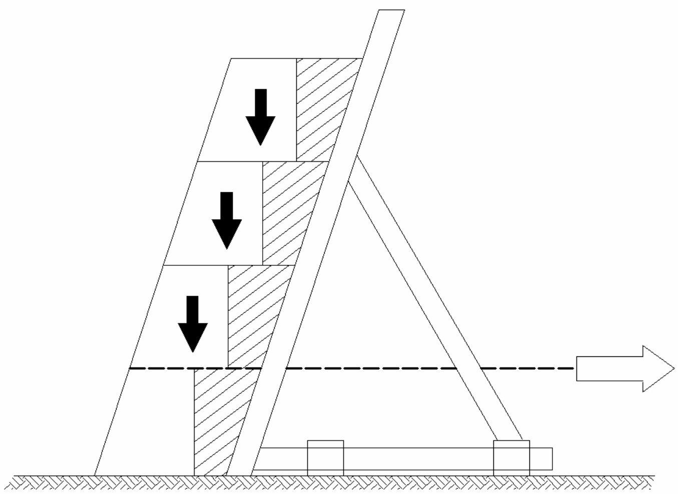


Figure 13. A sketch of possible form work, required to support the face during construction on very steep slopes

1.3.10.5 Geotechnical monitoring

The design of reinforced slopes shall also include plans of geotechnical monitoring. The following types of observation are recommended:

- geodetic observation of settlements and displacements on the crest
- relative displacements measured by extensiometers
- vertical inclinometer measurements
- observations of level and/or groundwater pressures.

1.3.11 Reinforced soil retaining walls and abutments

1.3.11.1 Area and purpose of use

Reinforced soil retaining walls should be considered as an alternative to conventional retaining walls. When compared with conventional retaining walls system, there are often some advantages to using walls with reinforced backfills. These systems are more flexible and less sensitive to seismic actions. They are especially appropriate when the supporting structure is to protect the slope, for example at the slopes connecting to structures or when constructing slopes in very limited area (Figure 14).

Reinforced soil retaining walls can be considered as a special, very steep to vertical type of reinforced slopes with additional face protection wall, which can be done using different elements or without them (Figure 15)

- face wrapping without face elements (a),
- face wrapping with elements (b),
- using geogrids and concrete panels (c)
- using strips, with tieback anchors in face panels (d)
- other forms of reinforcements by using geosynthetic or steel strips with different front shaped products.

When designing reinforced soil retaining walls, (see the section on embankment stability in cuts and at slopes, item 1.1) the following should be calculated:

- global (external) stability
- internal stability, length of the reinforced, tensile strength, vertical spacing, joints and overlapping.

The most commonly used method is classical Rankine earth pressure theory combined with tensile resistant tie backs, in which the reinforcement extends beyond an assumed Rankine failure plane.

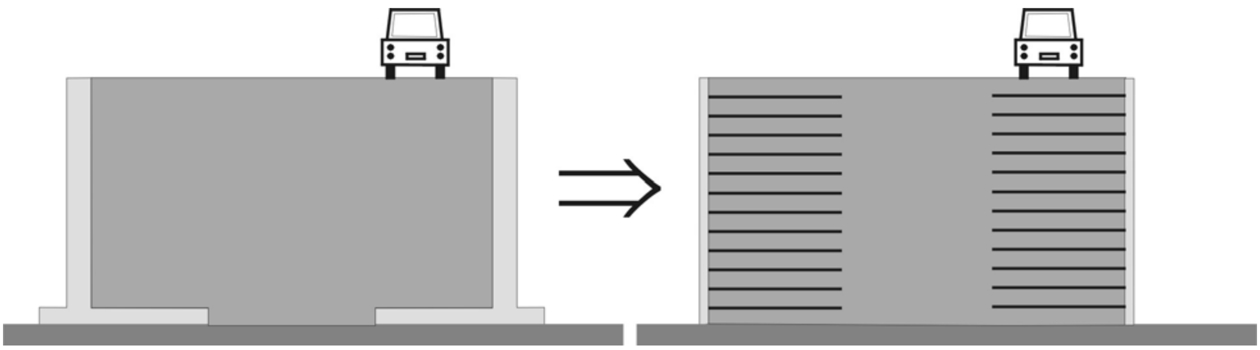


Figure 14. Example of supporting structure from reinforced soil instead of classical concrete wall.

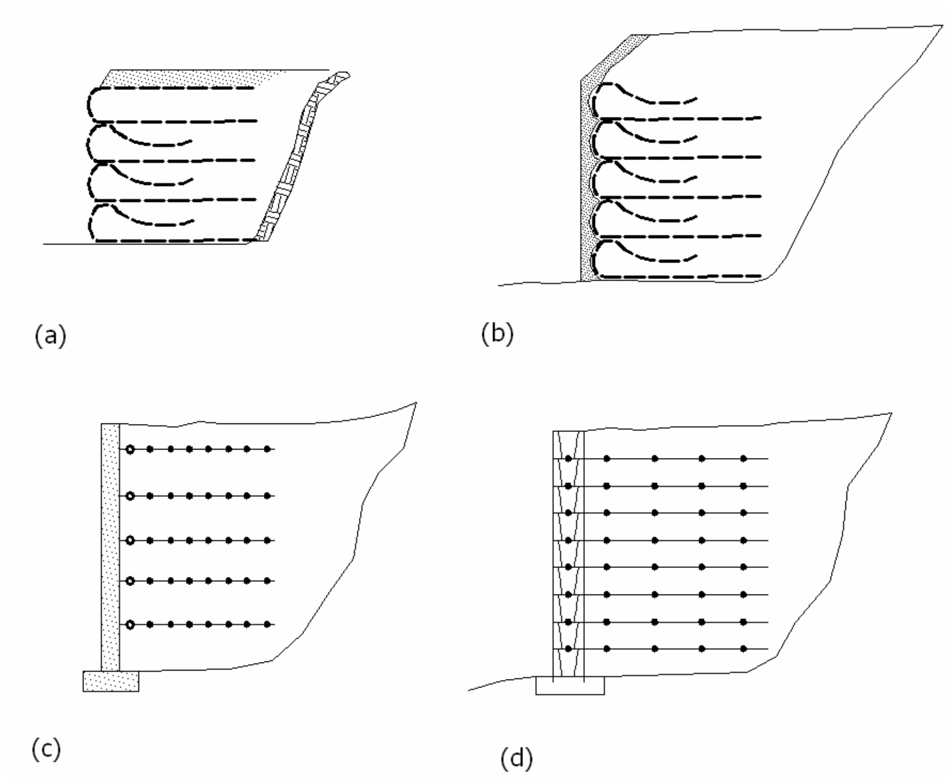


Figure 15. Schematic overview of reinforced retaining walls using geosynthetics

1.3.11.2 General design principles

When planning a supporting structure from reinforced soil, besides checking the global stability also the necessary length and the necessary number or spacing between reinforcement strips shall be determined computationally. The reinforcing strips can have the shape of a real narrow strip (steel, polyester), or the reinforcement is provided by geosynthetic material, spread continuously at the layer surface (geogrid or geotextile). The strips (geogrids, geotextiles) shall be long enough to transfer by friction and adhesion the earth pressure to the background, for the potential failure plaine, which corresponds to the total calculated active earth pressure (Figure 16).

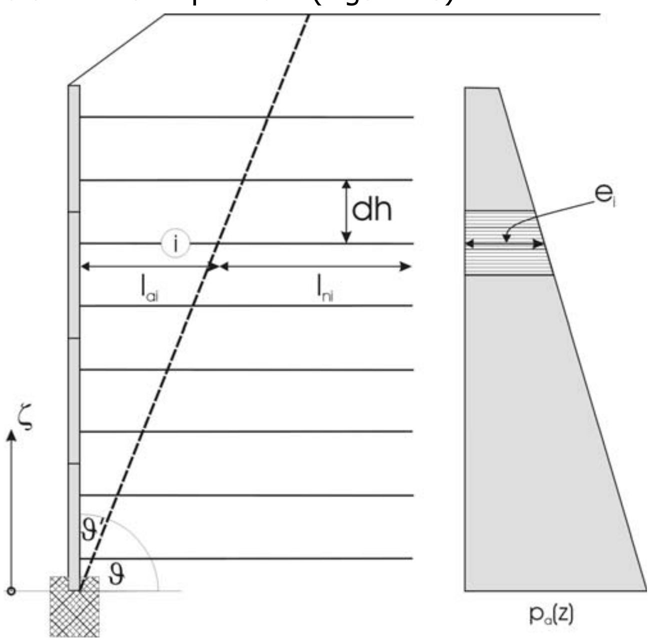


Figure 16. Schematic presentation of the calculus of earth pressures

The diagram of active earth pressures (per 1 running meter of the supporting structure) is divided into shares that load individual type of strips (geogrid, geotextile). Thus, the adequate resultant, taken over by individual strip type (geogrid, geotextile), equals the product of the earth pressure at the strip depth (geogrid, geotextile) and vertical distance between strip types (geogrid, geotextiles).

$k_a = \tan^2(\pi/4 - \varphi'/2)$

$\vartheta = \pi/4 + \varphi'/2$

$\vartheta' = \pi/4 - \varphi'/2$

$e_i = p_a\ (z_i) = \sigma_v\ k_a - 2\ c'\ \sqrt{k_a}$

$E_i = dh\ e_i$

Number of strips at level "i"

Force in individual strip (geogrid, geotextile) shall not exceed its design load-carrying capacity. For this reason, at each level adequate »strong« geogrid or geotextile, corresponding to force E_i, or adequate number of »strong« enough strips shall be placed.

The design force in the geogrid or in a group of strips is calculated from limit force of the geogrid or geotextile, or from the limit force of the strip by taking into account adequate safety quatient. The design values of the safety quotients are presented in Table 2. The actual values shall be selected according to the critical situation of the application.

For the geogrid:

$P_d = P_{limjt} / \gamma_{geogrid}$

For the strip:

$P_d = P_{limit} / \gamma_{strip}$

In case of using strips also the number of strips at level "i" shall be defined.

$n_i = E_i / P_d$

The number of strips at individual level at one slab shall be ≥ 2, to prevent rotation of slabs around one strip.

Strip and/or geogrid length at level "i"

Strip (geogrid, geotextile) length is the sum of two partial lengths of each strip (geogrid, geotextile):

- non-load-carrying length (l_{ai}), which bridges the distance from the screening plates to the active failure plane and
- load-carrying length (l_{ni}), which transfers by friction the strip (geogrid, geotextile) force to the background soil for the Rankin failure plane.

The non-load-carrying length is calculated from the geometry of active failure plane.

$l_{ai} = \zeta_i \tan \vartheta'$

The load-carrying length is defined from the equilibrium condition for individual strip (geogrid, geotextile) levels in the horizontal direction: force in the strip (geogrid, geotextile) E_i shall be equal to the friction along the load-carrying part of the strips with the width of *š*, or to the friction along the load-carrying part of the geogrid with the width of 1 m at the treated level:

Geogrids or geotextiles:

$E_i = \tau_i\ A_i = \tau_i\ 1\ l_{ni}$

$\tau_i = \sigma_{vi} \tan \varphi' + c'$

$$l_{ni} = \frac{E_i}{(\sigma_{vi} tg\varphi' + c')}$$

Strips:

$E_i = \tau_i\ A_i = \tau_i\ l_{ni}\ 2\ n_i\ š$

$\tau_i = \sigma_{vi} \tan \varphi' + c'$

$$l_{ni} = \frac{E_i}{n_i \, 2š(\sigma_{vi} tg\varphi' + c')}$$

In the last equation n_i is the actual number of strips at individual level.

The total strip (geogrid, geotextile) length is the sum of both partial lengths:

$l_i = l_{ai} + l_{ni}$

There are different analytical and numerical procedures available to calculate structures made of reinforced soil. The manufacturers of teotextiles normally possess prepared computational procedures and diagrams for the strip design, adapted to the inclination of the supporting structure front side made of reinforced soil (different values of active earth pressures) and to individual specific product (strip, geogrid), which are available free of charge on the Internet.

When performing geotechnical monitoring of supporting structures made of reinforced soil, the same principles as for any other geotechnical supporting structures are applicable.

GUIDELINES FOR ROAD DESIGN, CONSTRUCTION, MAINTENANCE AND SUPERVISION

VOLUME I: DESIGNING

SECTION 1: ROAD DESIGNING

Part 7: ROAD STRUCTURAL ELEMENTS

Chapter 2: PAVEMENT STRUCTURE

Sarajevo/Banja Luka
2005

2 PAVEMENT STRUCTURE

2.1 TRAFFIC LOAD

2.1.1 Subject of specification

The present specification provides a method for evaluation the design traffic loading, on the basis of which pavement layer dimensions are assessed. Pavements with asphalt and cement concrete surfacing for new road construction as well as for both repair and strengthening of existing roads are in question.

2.1.2 Reference documents

The specification is based on the following reference documents:

AASHTO Interim Guide for Design of Pavement Structures, AASHTO, Washington, D.C., 1974

JUS U.C4.010: 1981, Design and construction of roads, Assessment of total equivalent traffic loading for asphalt pavement design.

Road Note 29: 1970, A guide to the structural design of pavements for new roads, Road Research Laboratory, London

RStO 86: 1989, Richtlinien für die Standardisierung des Oberbaues von Verkehrsflächen (Guidelines for standardization of pavements of traffic surfaces)

SNV 640 320: 1971, Dimensionierung – Aequi-valente Verkehrslast (Design – equivalent traffic loading)

SNV 640 324: 1971, Dimensionierung – Straßenoberbau (Design – road pavement)

The specification includes provisions of other publications, either by dated or undated references. For dated references, subsequent supplements or modifications shall be considered, if they are included by a supplement or revision. For undated references the latest edition of the reference publication is valid.

2.1.3 Explanation of terms

The technical terms used in thisl specification shall be understood as indicated below:

Traffic analysis (Verkehrs-analyse, analiza saobraćaja) means recording, description, and evaluation of the existing traffic condition.

Pavement dynamic loading (dynamische Belastung der Fahrbahnbefestigung, dinamičko opterećenje kolovozne konstrukcije) is an additional loading resulting from the pavement surface condition and/or motor traffic, or the ratio of the actual traffic loading, acting on the pavement structure, to the static loading.

Pavement life time (Lebensdauer der Fahrbahn-befestigung, vrijeme trajanja kolovozne konstrukcije) is the design time of an adequate serviceability of a pavement surface in view of traffic safety, comfort, and economy.

Tandem / triple axle (Tan-dem/ Dreiachsig, Dupla / trostruka osovina) are two or three consecutive vehicle axles at spacing up to 1.8 m.

Equivalent traffic load (äquivalente Verkehrslast, ekvivalentno prometno opterećenje) is a loading expressed in terms of an equivalent number of vehicles of a nominal axle load of 82 kN as a rule.

Single axle (Einzelachse, jednostruka osovina) is an individual axle of a vehicle.

Equivalency faktor (Äquivalenzfaktor, faktor ekvavilentnosti) is an equivalent effect on fatigue in relation to the nominal axle loading.

Wheel load (Radlast, opterećenje točka) is the normal weight acting via wheels on the pavement structure.

Surfacing (Decke, zastor) is the pavement upper layer, generally built of a wearing course and bound pavement base-bearing course with a suitable binder.

Design traffic loading (massgebende Verkehrsbelas-tung, mjerodavno saobraćajno opterećenje) is a characteristic value of traffic loading acting on the pavement structure of one traffic lane in the design life; it is determined on the basis of the average annual daily traffic (number of vehicles) and of its growth, as well as of the additional factors: traffic lane number and width, maximum longitudinal fall of the carriageway, and eventual dynamical effects; it is a sum of passages of nominal axle load of 82 kN.

Traffic forecast (Verkehrs-prognose, prognoza saobraćaja) is an assessment of traffic condition in the future (in an appointed period).

Nominal axle load - (NOO) (nominelle Achslast, nazivno (nominalno) osovinsko opterećenje) is a standard/nominal single axle load of 81.6 (82) kN, transferred by double wheels of 4 x 20.4 kN to the pavement surface; it is defined as a base to compare effects of different axle loads.

New construction (Neubau, novogradja) is the first construciton of a road.

Strengthening (Verstärkung, ojačanje) means placing one or more additional material layers to the existing pavement to improve its bearing capacity and/or preserve its serviceability at suitable level.

Axle load (Achslast, osovinsko opterećenje) is a force, which is transferred onto the carriageway over wheels located on a single axle of a vehicle.

Repair (Instandsetzung, popravak) is a common term for measures to replace deficient or damaged items/members/places on the structure; such measures are periodically repeated.

Average annual daily traffic (ADT) (durch-schnittlicher täglicher Verkehr (DTV), prosječni godišnji dnevni saobraćaj) is the average daily number of motor vehicles having passed the selected road cross section in a specified year, assessed on the basis of traffic counting.

Traffic loading class (Verkehrsbelastungsklasse, razred saobraćajnog opterećenja) is a classification with regard to the traffic loading.

One-way carriageway (Richtungsfahrbahn, smjerni kolovoz) is such a carriageway, on which vehicles may move ahead in a specified direction only.

Traffic count (Verkehrszählung, brojanje saobraćaja) is a method of establishing vehicle type and number, or axle loads, passing a selected road cross-section in a specified time.

Vehicle weighing (Fahrzeug-wiegung, vaganje vozila) means measuring of vehicle mass or weight.

Weigh-in-motion/WIM, (Wiegen des rollenden Verkehrs, vaganje vozila u toku vožnje) is a measurement of axle loads acting on the pavement structure during driving.

Carriageway, (Fahrbahn, kolovoz) is a uniformly and continuously consolidated part of the pavement, suitable to vehicle traffic.

Traffic lane, (Fahrstreifen, saobraćajna traka) is a part of the carriageway, of a suitable width, allowing movement of one vehicle type in a single direction.

2.1.4 Traffic analysis

To assess traffic loading on a carriageway, the following shall be carried out:

- to specify the average annual daily traffic, and
- to specify the weight of individual axles of vehicles, or
- to estimate the vehicle utilization rate

2.1.4.1 Average annual daily traffic

Information on the average annual daily traffic (AADT) on existing motorways, expressways, and other national roads, established by the results of traffic count in selected characteristic roadway cross-sections, is assembled in suitable publications (e.g. "The Traffic" published by the Road Directorate).

The average annual daily traffic for construction of new roads shall be assessed by traffic forecasts.

For less loaded roads the average annual daily traffic can be estimated only.

The data on average annual daily traffic shall, as a rule, comprise the following classification of representative motor vehicles:

- motorcars and estate cars
- buses
- lorries:
- light – of carrying capacity up to 3 t
- medium – of carrying capacity 3 - 7 t
- heavy – of carrying capacity above 7 t
- heavy with trailers and trailer trains

2.1.4.2 Assessment of vehicle weight

Actual motor vehicle weight and individual axle load can only be assessed by a suitable weighing method. Weighing can be:

- static weighing by means of fixed or mobile weighing machines, or
- dynamical weighing for vehicles in motion (at normal speed, or at a speed up to 10 km/h on special platforms).

For weighing motor vehicles or individual axle loads such procedures are only adequate, where all the vehicles, or at least a representative pattern of these vehicles are weighed.

On the basis if weighing, motor vehicle axle loads shall be classified in appropriate classed, generally of a range of 5, 10, or 20 kN.

Results of weighing motor vehicles or axle loads shall be evaluated in terms of histograms for single, tandem, and triple axles, as to be directly applicable for assessment of traffic loading on existing roads, and for forecasting traffic loading on new roads.

When motor vehicle weighing results are not representative, they shall e corrected by suitable factors taking account of e.g. seasonal and/or daily influence.

2.1.4.3 4.3 Estimation of vehicle utilization rate

When the motor vehicle weight is not established by weighing, axle loads of representative vehicles shall be assessed by an estimation of their utilization rate.

Characteristic loading by selected represen-tative motor vehicles including the estimated utilization rate are indicated in annexes 1/1 to 1/6.

2.1.5 Equivalent traffic loading

The fatigue of pavement materials depends on the following:

- motor vehicle characteristics:
- axle loads,
- arrangement of axles on a vehicle,
- arrangement of wheels on a vehicle axis, and
- number of loading by motor vehicles, i.e. vehicle passages through the carriageway cross section.

2.1.5.1 Equivalent axle load

Motor vehicle axle loads shall be converted into equivalent traffic loading.

For a quantitative evaluation of the effect of different axle loads of motor vehicles on pavement material fatigue, the modified equation of the AASHO Road Test shall be introduced:

$$FE_{naz} = 10^{-8} \times f_o \times (f_k \times L_{stat})^4$$

$E_{naz} = E_{nom}$

where:

FE_{nom}- factor of equivalent effect of the actual motor vehicle axle load on fatigue related to the effect of the nominal axle load (NAL) of 82 kN.

f_a - factor of axle arrangement on a motor vehicle:

- for a single axle $f_{a11} = 2.212$
- for a tandem axle $f_{a2} = 0.1975$
- for a triple axle $f_{a3} = 0.048$
- for an individual axle of a tandem $f_{a12} = 1.583$

f_w - factor of wheel arrangement on a vehicle axis:

- for a single usual wheel and for weighed axles (temporarily) $f_{w1} = 1.0$
- for a double usual wheel (pair) $f_{w2} = 0.9$
- for a single wide wheel $f_{w3} = 0.97$

On the abovementioned bases equivalency factors of axle loads for actual axle loads of weighed vehicles (table 1), and of selected representative values (table 2) shall be evaluated.

2.1.5.2 Equivalent vehicle loading

Equivalency factor FE_v of a representative motor vehicle shall be assessed by the following equation:

$$FE_v = \sum FE_{naz} \quad \text{naz = nom}$$

Average values of equivalency factor for representative vehicles are indicated in table 3.

In cases where a prevailing type of motor vehicles on a certain road is known, and the vehicles are not weighed, a suitable equiva-lency factor FE_v for such vehicles shall be assessed by the mentioned equations.

Where the composition of heavy lorries is not known, informative values indicated in table 4 may be assumed as average values of equivalency factors FE_v for such vehicles.

2.1.5.3 5.3 Equivalent daily traffic loading

Assessment of equivalent daily traffic loading is defined by the method of how the traffic loading on carriageways is assessed.

Table 1: Equivalency factors of axle loads of weighed vehicles in relation to nominal axle load of 82 kN

Axle load (kN)	Equivalency factor single axle	tandem axle	triple axle
4	0.000006		
6	0.000029		
8	0.000091		
10	0.00022		
15	0.00112		
20	0.00354	0.00063	
25	0.00864	0.00154	

Axle load (kN)	Equivalency factor single axle	tandem axle	triple axle
30	0.01792	0.00320	
35	0.03319	0.00593	
40	0.05663	0.01011	
45	0.09071	0.01620	
50	0.13825	0.02469	0.00900
55	0.20241	0.03614	0.01318
60	0.28668	0.05119	0.01866
65	0.39486	0.07051	0.02570
70	0.53110	0.09484	0.03457
75	0.69989	0.12498	0.04556
80	0.90604	0.16179	0.05898
85	1.15468	0.20619	0.07517
90	1.45129	0.25916	0.09448
95	1.80169	0.32173	0.11729
100	2.21200	0.96500	0.14400
105	2.68870	0.48012	0.17503
110	3.23859	0.57832	0.21083
115	3.86880	0.69086	0.25186
120	4.58680	0.81907	0.29860
125	5.40039	0.96436	0.35156
130	6.31769	1.12816	0.41128
140	8.49762	1.51743	0.55319
150	11.1982	1.99969	0.72900
160	14.4965	2.58867	0.94372
170	18.4748	3.29908	1.20270
180	23.2207	4.14655	1.51165
190	28.8270	5.14768	1.87662
200	35.3920	6.32000	2.30400
210		7.68200	2.80053
220		9.25311	3.37329
230		11.0537	4.02971
240		13.1052	4.77757
250		15.4297	5.62500
260		18.0506	6.58045
270		20.9919	7.65275
280		24.2789	8.85105
290		27.9376	10.1848
300		31.9950	11.6640
310			13.2987
320			15.0995
330			17.0773
340			19.2432
350			21.6090
360			24.1865
380			30.0260
400			36.8640

Table 2: Equivalency factors of axle loads of selected representative motor vehicles in relation to nominal axle load of 82 kN

Axle load kN	Equivalency factor				
	single axle		individual axle in tandem		
	singe wheel	double wheel	singe wheel	double wheel	single wide wheel
4	0.000006	0.000004	0.000004	0.000003	0.000004
6	0.000029	0.000019	0.000021	0.000013	0.000018
8	0.000091	0.000059	0.000065	0.000043	0.000057
10	0.000221	0.000145	0.000158	0.000104	0.000140
15	0.00112	0.00074	0.00080	0.00053	0.00071
20	0.00354	0.00232	0.00252	0.00166	0.00224
25	0.00864	0.00567	0.00618	0.00406	0.00547
30	0.01792	0.01176	0.01282	0.00841	0.01135
35	0.03319	0.02178	0.02375	0.01559	0.02103
40	0.05663	0.03715	0.04052	0.02659	0.03588
45	0.09071	0.05951	0.06491	0.04259	0.05747
50	0.13825	0.09071	0.09894	0.06491	0.08759
55	0.20241	0.13280	0.14485	0.09504	0.12824
60	0.28668	0.18809	0.20516	0.13460	0.18162
65	0.39486	0.25906	0.28258	0.18540	0.25016
70	0.53110	0.34846	0.38001	0.24937	0.33648
75	0.69989	0.45920	0.50087	0.32862	0.44342
80	0.90603	0.59445	0.64840	0.42541	0.57402
85	1.1547	0.75758	0.82634	0.54216	0.73155
90	1.4513	0.95219	1.0386	0.68143	0.91947
95	1.8017	1.1821	1.2894	0.84595	1.1415
100	2.2120	1.4513	1.5830	1.0386	1.4014
105	2.6887	1.7641	1.9241	1.2624	1.7034
110	3.2386	2.1248	2.3177	1.5206	2.0518
115	3.8688	2.5383	2.7687	1.8165	2.4511
120	4.5868	3.0094	3.2825	2.1537	2.9060
125	5.4004	3.5432	3.8647	2.5357	3.4214
130	6.3177	4.1450	4.5212	2.9664	4.0026
140	8.4976	5.5753	6.0813	3.9899	5.3837
150	11.198	7.3472	8.0139	5.2579	7.0947

Table 3: Average values of equivalency factors for representative vehicles

Representative vehicle	Average equivalency factor
- motor car	0.00006
- bus	1.20
- lorry:	
- light	0.01
- medium	0.20
- heavy	1.10
- heavy with trailer	2.00

Table 4: Average informative values of equivalency factors for lorries

Average number of heavy lorries per day	Average equivalency factor
< 200	0.9
> 200 - 1,000	1.3
> 1000	1.8

2.1.5.3.1 Assessment on the basis of actual axle loads

When actual, i.e. weighed axle loads are known, the total daily equivalent traffic loading in the carriageway cross-section (T_d) can be assessed as a sum of all the measured axle loads:

$T_d = \sum FE_{naz}$ naz = nom

When the sum ΣFE_{nom} is not known (i.e. it is not directly provided by the weighing system), it shall be calculated from the contributions of sums of axle loads for an individual arrangement of axles $FE_{o,i}$ from the axle load histograms:

$$FE_{o,i} = \sum_{j=1}^{R} N_{i,j} x FE_{naz,j}$$

where:

i	–	single, tandem, or triple axle
R	–	number of classes in axle load histograms
N_j	–	number of axles of the j^{th} class
$FE_{nom,j}$	–	factor of equivalency effect of a mean value of axle load of the j^{th} class

The contributions of individual axle arrangements on motor vehicles shall be assessed by the following equations:

- for single axes:

$$FE_{o1} = 10^{-8} \times 2{,}212 \times \sum_{j=1}^{R} N_{j1} \times L_j^4$$

- for tandem axles:

$$FE_{o2} = 10^{-8} \times 0{,}1975 \times \sum_{j=1}^{R} N_{j2} \times L_j^4$$

- for triple axles:

$$FE_{o3} = 10^{-8} \times 0{,}048 \times \sum_{j=1}^{R} N_{j3} \times L_j^4$$

2.1.5.3.2 Assessment on the basis of average values of equivalency factors

The total daily equivalent traffic loading in a carriageway cross-section (T_d) can be assessed on the basis of the design average daily number of motor vehicles in the first year of use of the road by the following equation:

$T_d = \sum FE_v \ x \ n_v$

where:

FE_v - equivalency factor of a representative motor vehicle

n_v - number of motor vehicles of a certain type (representative) per day at the beginning of use of a road

2.1.6 Additional factors influencing a traffic loading

Additional factors influencing a traffic loading are represented by the road characteristics:

- number of traffic lanes
- traffic lane width
- longitudinal fall of carriageway vertical alignment.

2.1.6.1 Number of traffic lanes

The effect of distributing the traffic loading on traffic lanes on a carriageway shall be considered by the cross-section factors f_{pp}, indicated in table 5.

When the traffic loading is established by weighing on a traffic lane, the value of the cross-section factor shall be taken as f_{pp} = 1.0 for the particular traffic lane.

Table 5: Factors of distribution of traffic loading on traffic lanes

Number of traffic lanes	Factor of distribution of traffic loading on traffic lanes					
1	1.00					
2		0.50			0.50	
3		0.50		0.05		0.45
4	0.45		0.05	0.05		0.45
5	0.45		0.05	0.02	0.08	0.40
6	0.40	0.08	0.02	0.02	0.08	0.40

2.1.6.2 Traffic lane width

The effect of the carriageway traffic lane width on the traffic loading shall be considered by the factors f_{lw} indicated in table 6. .

Table 6: Factors of the effect of traffic lane width on traffic loading

Traffic lane width (m)	Factor of traffic lane width
< 2.50	2.00
2.50 – 2.75	1.80
2.76 – 3.25	1.40
3.25 – 3.75	1.10
> 3.75	1.00

2.1.6.3 Longitudinal fall of carriageway vertical alignment

The effect of the (greatest) longitudinal fall of the carriageway vertical alignment on the traffic loading shall be considered by the factors f_{nn} indicated in table 7. indeks nn = ?

Table 7: Factors of the effect of longitudinal falls of carriageway vertical alignment on traffic loading

Longitudinal fall of vertical alignment (%)	Factor of longitudinal fall of vertical alignment
< 2	1.00
above 2 up to 4	1.02
above 4 up to 5	1.05
above 5 up to 6	1.09
above up to 7	1.14
above 7 up to 8	1.20
above 8 up to 9	1.27
above 9 up to 10	1.35
> 10	1.45

2.1.6.4 6.4 Dynamic effects

Motor vehicle sway being a consequence of certain carriageway unevenness provoke additional dynamic loading, which can be considered by the factor f_{dv} (indeks?) amounting to:

- for good driving conditions f_{dv} = 1.03

- for medium driving conditions $f_{dv} = 1.08$

The values of factors f_{dv} depend particularly on the quality of the works carried out.

2.1.7 Design traffic loading

The design traffic loading in defined by the following:

- design equivalent daily traffic loading T_d,
- additional effects being a consequence of road characteristics, and
- duration of traffic, and annual traffic growth.

The evaluation of the equivalent daily traffic loading T_d is defined in clause 5, whilst of the additional effects due to road characteristics in clause 6.

2.1.7.1 Duration and increase of traffic loading

The planned duration and increase of the traffic loading due to traffic growth in this period shall be considered by the factor f_{tp} indicated in table 8.

Table 8: Factors of traffic loading increase in dependence on the design annual traffic growth and planned duration

Planned duration (years)	Annual traffic growth rate (%)									
	1	2	3	4	5	6	7	8	9	10
	Factor of traffic loading increase f_{tp}									
5	5	5	5	6	6	6	6	6	7	7
10	11	11	12	12	13	14	15	16	17	17
15	16	18	19	21	23	25	27	29	32	35
20	22	25	28	31	35	39	44	49	56	63

2.1.7.2 Assessment of design traffic loading

The design traffic loading T_n shall be assessed by means of the following equation:

$$T_n = 365 \cdot T_d \cdot f_{pp} \cdot f_{šp} \cdot f_{nn} \cdot f_{dv} \cdot f_{tp}$$

where:

T_n - *design traffic loading in a period of n years*
T_d - *equivalent daily traffic loading*
f_{pp} - *factor of average carriageway cross-section*
$f_{šp}$ - *factor of traffic lane width*
f_{nn} - *factor of longitudinal fall of vertical alignment*
f_{dv} - *factor of additional dynamical effects*
f_{tp} - *factor of traffic loading increase due to traffic growth in the period considered*

2.1.8 Classification of traffic loading

In dependence on the number of passages of the nominal axle load per day and in a 20-years period respectively, the pavement traffic loading relevant to determine the layer thicknesses, are classified into 6 traffic loading groups.

The traffic loading classificaton is indicated in table 9.

Table 9: Classification of traffic loading into traffic loading groups

Traffic loading group	Number of passages of nominal axle loading of 82 kN	
	per day	in 20 years

- exceptionally heavy	above 3,000	above 2×10^7
- very heavy	above 800 up to 3,000	above 6×10^6 up to 2×10^7
- heavy	above 300 up to 800	above 2×10^6 up to 6×10^6
- medium	above 80 up to 300	above 6×10^5 up to 2×10^6
- light	above 30 up to 80	above 2×10^5 up to 6×10^5
- very light	above 30	up to 2×10^5

2.1.9 Annex 1/1

2.1.9.1 Calculation of equivalency factor of a representative motor car

Vehicle carrying capacity: 4 kN
Vehicle loading: 4 persons with luggage

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		5.5	0.00002	5.5	0.00002
semi-loaded		6.0	0.00003	6.0	0.00003
fully loaded		7.5	0.00007	7.5	0.00007
	Portion %				
empty	60	0.000012		0.000012	
semi-loaded	30	0.000009		0.000009	
fully loaded	10	0.000007		0.000007	
Total:		0.000028		0.000028	
			FE_V =	0.000056	

$FE_V \cong$ 0.00006

2.1.9.2 Calculation of equivalency factor of a representative bus

Vehicle carrying capacity: 40 kN
Vehicle loading: 50 passengers

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		50	0.13825	70	0.34846
semi-loaded		55	0.20241	85	0.75758
fully loaded		60	0.28668	100	1.45129
	Portion %				
empty	5	0.006913		0.017423	
semi-loaded	60	0.121446		0.454548	
fully loaded	35	0.100338		0.507952	
Total:		0.228697		0.979923	
			FE_V =	1.208620	

$FE_V \cong$ 1.20

2.1.10 Annex 1/2

2.1.10.1 Calculation of equivalency factor of a representative light lorry

Vehicle carrying capacity: up to 30 kN
Vehicle loading: 25 kN

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		15.0	0.00112	10	0,00015
semi-loaded		17.5	0.00207	20	0,00232
fully loaded		20.0	0.00354	30	0,01176
	Portion %				
empty	25	0.000280		0,000036	
semi-loaded	25	0.000518		0,000580	
fully loaded	50	0.001770		0,005880	
Total:		0.002568		0,006496	
			FE_v =	0,009064	

$FE_V \cong$ 0.01

2.1.11 Annex 1/3

2.1.11.1 calculation of equivalency factor of a representative medium lorry

Vehicle carrying capacity: 30 - 50 kN
Vehicle loading: 40 kN

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		20	0.00354	20	0,00232
semi-loaded		25	0.00864	35	0,02178
fully loaded		30	0.01792	50	0,09071
	Portion %				
empty	25	0.000885		0.000580	
semi-loaded	25	0.002160		0.005445	
fully loaded	50	0.008960		0.045355	
Total:		0.011905		0.051380	
			FE_{v1} =	**0.063285**	

Vehicle carrying capacity: 50 - 70 kN
Vehicle loading: 60 kN

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		30	0.01792	30	0,01176
semi-loaded		35	0.03319	55	0,13280
fully loaded		40	0.05663	80	0,59445
	Portion %				
empty	25	0.004480		0,002940	
semi-loaded	25	0.008298		0,033200	
fully loaded	50	0.028315		0,297225	
Total:		0.031093		0,333365	
			FE_{v2} =	**0,364458**	

FE_{v1} : 0.063285 x 50 % = 0.031642
FE_{v2} : 0.364458 x 50 % = 0.182229
0.213871

$FE_v \cong$ 0.20

2.1.12 Annex 1/4

2.1.12.1 Calculation of equivalency factor of a representative heavy lorry

Vehicle carrying capacity: > 70 kN
Vehicle loading: 100 kN

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		40	0.05663	40	0.03715
semi-loaded		55	0.20241	75	0.45920
fully loaded		70	0.53110	110	2.12480
	Portion %				
empty	25	0.014158		0.009288	
semi-loaded	25	0.050602		0.114800	
fully loaded	50	0.265550		1.062400	
Total:		0.330310		1.186488	
			FE_{v1} =	1.516798	

Vehicle carrying capacity: > 70 kN
Vehicle loading: 140 kN (tandem axle)

Vehicle loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		40	0.05663	2 x 20	2 x 0.00166
semi-loaded		55	0.20241	2 x 45	2 x 0.04259
fully loaded		70	0.53110	2 x 75	2 x 0.32862
	Portion %				
empty	25	0.014158		0.000830	
semi-loaded	25	0.050602		0.021295	
fully loaded	50	0.265550		0.328620	
Total:		0.330310		0.350745	
			FE_{v2} =	0.681055	

FE_{v1} : 1.516798 x 50 % = 0.758399
FE_{v2} : 0.681055 x 50 % = 0.340528
1.098927

$FE_v \cong$ 1.10

2.1.13 Annex 1/5

2.1.13.1 Calculation of equivalency factor of a representative heavy lorry with trailer

Vehicle carrying capacity: 50 – 70 kN Trailer carrying capacity: 60 kN
Vehicle loading: 60 kN Trailer loading: 60 kN

Trailer loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		10	0.00022	10	0.00022
semi-loaded		25	0.00864	25	0.00864
fully loaded		40	0.05663	40	0.05663
	Portion %				
empty	25	0.000055		0.000055	
semi-loaded	25	0.002160		0.002160	
fully loaded	50	0.028315		0.028315	
Total:		0.030530		0.030530	
			FE_{p1} =	0.06106	

FE_v :	0.212923
FE_{p1} :	0.061060
	0.273983

Vehicle carrying capacity: > 70 kN Trailer carrying capacity: 120 kN
Vehicle loading: 100 kN Trailer loading: 120 kN

Trailer loading		Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2
empty		30	0.01176	30	0.01176
semi-loaded		60	0.18809	60	0.18809
fully loaded		90	0.95219	90	0.95219
	Portion %				
empty	25	0.002940		0.002940	
semi-loaded	25	0.047023		0.047023	
fully loaded	50	0.476095		0.476095	
Total:		0.526058		0.526058	
			FE_{p2} =	1.052116	

FE_{v1} :	1.516798
FE_{p2} :	1.052116
	2.568914

Vehicle carrying capacity: > 70 kN Trailer carrying capacity: 160 kN
Vehicle loading: 140 kN Trailer loading: 160 kN

Trailer loading	Front axle L_1 (kN)	Front axle FE_1	Rear axle L_2 (kN)	Rear axle FE_2

empty		40	0.03715	40 + 20	0.02659+0.00252
semi-loaded		70	0.34846	70 + 35	0.24937+0.02375
fully loaded		100	1.45130	100 + 50	1.03860+0.09894
	Portion %				
empty	25	0.009288		0.006648 + 0.000630	
semi-loaded	25	0.087115		0.062342 + 0.005938	
fully loaded	50	0.725650		0.519300 + 0.049470	
Total:		0.822053		0.644328	
			FE_{p3} =	1.466381	

FE_{v2} : 0.681055

FE_{p3} : 1.466381

2.147436

FE_{v} + Fe_{p1} : 0.273983 x 15 % = 0.041097

FE_{v1} + Fe_{p2} : 2.568914 x 40 % = 1.027566

FE_{v2} + Fe_{p3} : 2.147463 x 45 % = 0.966358

2.035021

FE_{v+p} ≅ 2.00

2.2 CLIMATE AND HYDROLOGICAL CONDITIONS

2.2.1 Subject of technical specification

The present specification provides bases for definition of both climatic and hydrological conditions in road construction, which directly affect the determination of both type and dimensions of road pavements.

The specification provides explanation of the most frequently used terms related to freezing and thawing of road body materials, frost characteristics, and occurrence of damages due to both freezing and thawing.

On the basis of the present specification, the required measures are indicated to protect materials from damage due to freezing at the design, construction, and maintenance stage, as well as to ensure economy of those measures.

The contents of this technical specification cannot be so interpreted and implemented as to prevent or condition a suitable application of construction products approved for the use in compliance with the provisions of the Law of construction products.

2.2.2 Reference documents

The present technical specification is based on the following reference documents:

SN 640 317a: 1988 Dimensionierung, Unterbau und Untergrund (*Design, Substructure and Subgrade*)

SN 670 140a: 1988 Frost (*Frost*)

SN 670 005: 1970 Klassifikation der Lockergesteine, Feldmethode nach USCS (*Classification of Soils, In-situ Method by USCS*)

SN 670 008: 1970 Klassifikation der Lockergesteine, Laboratoriumsmethode nach USCS (*Classification of Soils, Laboratory Method by USCS*)

Zusätzliche technische Vertragsbedingung-en und Richtlinien für Erdarbeiten im Strasenbau - ZTVE 94, DIN 18196, Bodenklassifikation für bautechnische Zwecke

(***Additional Technical Contractual Condi-tions and Guidelines for Earth Works in Road Construction – ZTVE 94, DIN 18196,*** *Classification of Soils for Construction Purposes*)

The specification includes dated provisions of other publications. Subsequent supplements or modifications shall be considered, if they are included by a supplement or revision.

2.2.3 Explanation of terms

The technical terms used in this Technical specification shall be understood as indicated below.

Frost depth, (Frosttiefe, dubina smrzavanja,) is a maximum depth to which the isotherm 0°C reaches at long standing frost.

Hydrological conditions, (hydrologische Verhältnisse, hidrološki uslovi) are conditions defining water conditions in the ground next to road.

Frost index, (Frostindex, indeks mraza) is the sum of mean (negative) daily temperatures from the beginning to the end of the frost season; it denotes frost duration and intensity at certain location.

Climatic conditions, (klimatische Verhältnisse, klimatski uslovi) are conditions, defined by the air temperature in certain time period at certain location or in certain area, where a road is situated.

Ice lens, (Eislinse, ledeno soćivo) is typical form of pore water, occurring at freezing due to increased water content in the material.

Micro climate, Mikroklima, mikroklima) is an entirety of equal conditions, such as tempera-ture, sun radiation, precipitations, snow con-ditions, and wind, characteristic for a limited area.

Frost insensible material, (frostunempfindliches Material, na smrzavanje neosetljiv material) is material, in which the freezing pore water does not cause any significant increase of the bearing capacity, neither the thawing causes any essential reduction of the bearing capacity of such material.

Thaw, (Auftauen, otapanje) is a whole of physical phenomena occurring in materials, when the temperature rises above 0°C after frost period.

Frost damage, (Frostbeschädigung, oštećenje zbog smrzavanja) is damage to the structure, which is either a direct or indirect consequence of frost action in connection with water; it can lead to structural failure.

Frost heave, (Frosthebung, dizanje zbog smrzavanja) is a local lifting of the carriageway due to ice lens formation in an unsuitable material built-in to the frost depth.

Freeze, (Frieren, smrzavanje) is a complex of physical phenomena arising in materials when the temperature is below 0°C.

2.2.4 Definition of conditions

The extent of changes of road body materials during freezing and thawing depends in particular on the characteristic events in these processes. The freezing and thawing process is dependent especially on the following:

- material characteristics, and
- local climatic and hydrological conditions.

The influence of material characteristics is described in detail in chapter 5. The influence of local climatic and hydrological conditions of freezing and thawing is the basic term to define pavement dimensions.

2.2.4.1 Climatic conditions

2.2.4.1.1 General

When assessing the hazard of changes of executed pavement characteristics or road body materials due to freezing and thawing, the most unfavourable foreseeable local conditions shall be considered, i.e.:

- long lasting frost and a slow penetration of the 0°C isotherm into the material, and
- fast thawing.

It can be estimated that there is no hazard of harmful changes due to freezing and thawing in the following cases:

- during a short period of freezing (also at severe frost), when the 0°C isotherm penetrates only into frost resistant material layers, and
- during a short period of thawing (south wind).

The depth of frost penetration into the road body depends of the material thermal properties, particularly on their compaction and moisture content.

As the moisture content in the subbase and substructure materials is generally low, the frost can propagate in such material relatively quickly. In fine-grained materials containing greater water quantities the frost propagates slower. This means that when the pavement thickness is increased to provide protection from changing the properties of materials, which are non-resistant to freezing, partly provokes a deeper frost penetration, thus causing freezing.

2.2.4.1.2 Definition of climatic conditions

The climatic conditions in a certain environment can be defined either by

- frost depth (frost penetration) h_m, or
- frost index I_m.

Both values shall be assessed by suitable measurement methods.

2.2.4.1.3 Frost depth h_m

To measure frost depth the following methods can be adopted:

- by means of a sound with measuring vials,
- by means of a sound with an frost depth indicator, and
- with an electric sound.

The medium to measure temperature in the freezing point area or beyond it is inserted into a suitable pipe, and the latter is placed into a vertical borehole in the road body.

Basically, frost depth measurements are intended for the monitoring of temperatures in road pavements.

On the basis of established maximum frost penetration depths in typical regions in Slovenia, a map of informative frost penetration depths (Annex 1) has been prepared, which enables a simple and quick assessment of the design frost penetration depth in a certain environment.

2.2.4.1.4 Frost index I_m

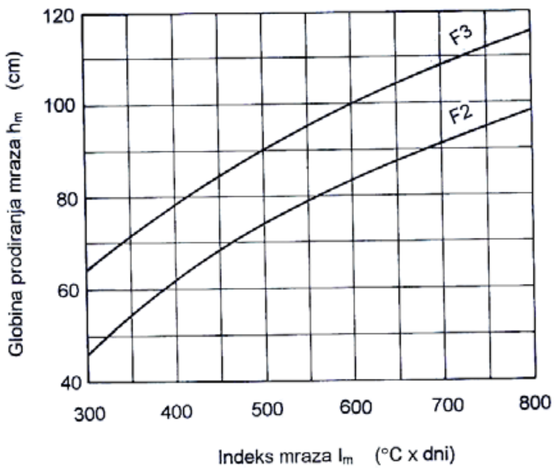
The frost index is defined by the absolute difference between the highest and the lowest point of the integrated curve of mean daily (negative) air temperatures (°C x days). It represents both intensity and influence of low temperatures in a certain milieu.

The assessment of the mean daily air temperature is based on a method used by the hydro-meteorological service (average of temperature measurements at 7 a.m., 2 p.m., and 9 p.m. at a height of 1.2 m above the ground). The integrated curve of the mean daily temperatures increases when the temperatures are positive, and begins to decrease when the temperature drops below 0°C. In case that a transient warning has occurred during the freezing period, which, however, has not caused any thawing of the material as a whole, the total difference between the highest and the lowest point of the integrated curve shall be considered as the frost index design value.

For the road pavement design and for determination of measures to protect the pavement from adverse freezing and thawing effects, the design frost index shall be assessed for the design service life. As a rule, this is the mean value of the frost index in the three coldest winters in the selected series of years.

On the basis of the data provided by the Slovenian hydro-meteorological institute on temperatures measured in characteristic regions in Slovenia (for the years from 1951 to 1970), a map of informative values of frost index I_m has been worked out (Annex 2).

The effect of frost index I_m on assessment of the required protection of materials F2 and F3 prone to freezing is determined indirectly by the depth of frost penetration h_m (diagram 1).



indeks mraza I_m =frost index I_m (°C x days)
globina prodiranja mraza = frost penetration depth

Diagram 1: Dependence of the frost penetration in materials F2 and F3 on the frost index

2.2.4.2 Hydrological conditions

Hydrological conditions in certain environment are essential to estimate the sensitivity of the pavement and its constituent materials respectively, to freezing, and to determine measures for damage prevention.

Hydrological conditions are defined by the following:

- ground water level,
- frost depth, and
- material sensitivity to freezing.

On the basis of these factors hydrological conditions can be classified into

- favourable, and
- unfavourable.

Hydrological conditions are favourable, where:

- a road fill is at least 1.5 m high,
- the ground water level is permanently below the frost depth h_m,
- a shallow cut is well drained, and
- water inflow into the road body from the side (from water threads) or from the surface is prevented above the ground water level.

Hydrological conditions are unfavourable, where

- a road fill is lower than 1.5 m,
- the ground water level is in the frost depth area h_m,
- a shallow cut is insufficiently drained,
- a cut is deep, and
- capillary water lifting, or inflow of water from the side or from the surface is enabled.

In fine-grained materials generally being more sensitive to freezing, capillary water lifting can be significant. Therefore, in most cases unfavourable hydrological conditions shall be considered, notwithstanding that the ground water level is located several metres below the subgrade formation.

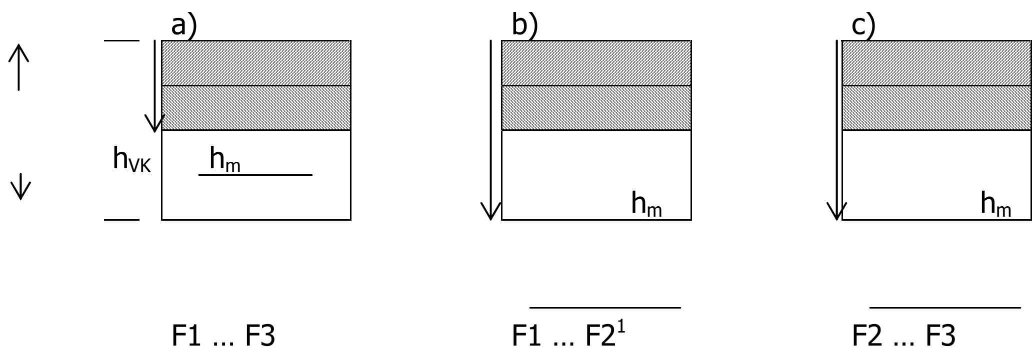
2.2.4.3 Factors influencing design

The risk of occurrence of damages to roads due to freezing and thawing depends on the following factors:

- frost penetration depth h_m
- hydrological conditions,
- sensitivity of materials to freezing defined in terms of classes in item,
- thickness of pavement, which is insensitive to freezing.

Measures to prevent adverse effects of freezing and thawing are only required and relevant for materials classified into classes F2 and F3, when the frost depth reaches up to these materials, and unfavourable hydrological conditions have to be considered (Fig. 1c).

Under all other conditions (Fig. 1a and Fig. 1b), pavement fatigue resistance is relevant to the design.



Legend:

h_{VK} - pavement thickness

h_m - design frost penetration depth

F1 … F3 - material sensitivity to freezing

[1] - materials classified into F1, if the condition indicated in diagram 2 is fulfilled

Fig. 1: Types of measures to prevent adverse freezing effects

To protect a road pavement from harmful freezing effects or from damage, it shall be executed of resistant materials at certain depth. Experiences gained on roads of heavy traffic loading where no damage due to freezing and thawing has been noticed, have shown that the

minimum required pavement thickness h_{min} (i.e. the thickness of resistant materials) is not equal to the measured maximum frost penetration depth h_m, but that, as a rule, a smaller total depth h_{min} of the layer of frost resistant materials is sufficient. On this basis, Table 1 has been worked out.

Table 1: Minimum required pavement thicknesses h_{min}

Resistance of material below pavement to freezing and thawing effects	Hydrological conditions	Pavement thickness h_{min}
resistant	favourable	$\geq 0.6\ h_m$

	unfavourable	≥ 0.7 h_m
non-resistant	favourable unfavourable	≥ 0.7 h_m ≥ 0.8 h_m

2.2.5 Freezing and thawing

2.2.5.1 Basic characteristics of freezing and thawing

2.2.5.1.1 Freezing of water

At approximately 0° C, the water is converted from a liquid state into a crystal structure, which specific volume is increased by approximately 10 %, and its density decreased by approximately 9 %. During the freezing process 335 kJ/kg of heat energy is released.

When there are no crystallization nuclei in the water, of when the water contains chemical solutions, or when the water is under pressure, the freezing point can be shifted towards a lower temperature.

As pure water does not contain any crystallization nuclei, significant undercooling of water can occur without any ice formation.

2.2.5.1.2 Freezing of moist material

As the pore water in material is freezing, ice crystals are growing thus changing the natural water equilibrium in dependence on both granulometric composition and mineralogical properties of the material. Due to reduction of content of non-frozen water in the material, certain negative pressure (tensile stresses) occurs, which causes a water inflow, particularly from the zone below the 0°C isotherm, i.e. frost depth. This water, after its arrival into the frost zone, increases the ice amount in a form of ice lenses and layers, which dig up and lift the frozen material. In this way, frost heaves take rise.

The increase of ice quantity (ice lenses) in the material depends on the following:

- water content,
- diameter and content of fine grains,
- water permeability of material,
- freezing duration, and
- frost severity.

Indirectly, it depends on the negative pressure having come into existence.

It can be derived from the abovementioned considerations that neither well permeable coarse-grained materials nor soils, e.g. clays, are sensitive to freezing, as they make the pore water inflow into the frost zone difficult. By increasing the content of fine grains (silt sands, silts), the danger of occurrence of excessive ice amounts in material becomes mightily greater.

When the pore water freezes in cohesive materials and moist rocks, the volume becomes greater, and a phenomenon occurs, which is similar as in case of blasting, if the bond strength does not resist the crystallization pressure. However, such a phenomenon only appears at temperatures substantially below 0° C, as the water freezing point drops due to increased pressure.

2.2.5.1.3 Frost line penetration

During freezing period the frost line penetrates into the material. The more severe the frost and the longer it acts, the deeper and the faster the frost penetration.

The informative frost line penetration depth is approximately proportional to square root of the penetration time.

2.2.5.1.4 Thawing

The ice, which has been formed in the road body during freezing period, begins to thaw, when the heat is conducted to it

- from above due to warming, and
- from below from Earth.

By thawing of increased amounts of ice, i.e. ice lenses or layers, plastic properties and consistency of material (soil), which has been loosened on freezing, are changing. In certain circumstances the material can become mushy or even liquid, thus its bearing capacity is essentially changed.

Thawing of moist material from above is often faster than from below, so that a thawed material enriched with water rests on a still frozen substrate, thus is cannot be drained downwards. In such conditions a mushy or liquid material can also be pressed through cracks onto pavement surface.

Only when the water evacuation downwards is made feasible, the excessive water can flow away, and the material can gradually regain its original properties.

2.2.5.2 Damage due to freezing and thawing

2.2.5.2.1 General conditions in which damages can occur

Damages to pavement structures can occur in the following conditions:

- sufficiently severe and persistent frost,
- pavement material sensitive to freezing placed in the frost zone,
- water can enter the pavement up to the frost line,
- traffic loading,
- insufficient pavement bearing capacity.

As a rule, damages to pavements can only occur, when all the abovementioned factors arise. Protective measures against damage due to freezing and thawing are generally intended for only one of the conditions indicated.

It has been proven by different tests that no damage, or only insignificant damage will occur due to freezing, when

- the frost does not reach below the lower edge of the frost resistant pavement,
- a severe frost appears suddenly, and there is not enough time for the ice accumulation in the zone of sensitive material; damages can only result from hardening and shrinkage of asphalt mix (crack opening).

Minor damage due to freezing can occur, when the freezing is fast, and the frost is persistent and reaches deeply; an ice accumulation usually arises so deep, that, except causing wide frost heaves, it does not affect adversely the pavement surface.

Major damage to the pavement occurs, when the frost penetrates only few below the pavement, i.e. into a frost sensitive material, but it persists for a longer period, and causes significant ice accumulation directly below the pavement.

2.2.5.2.2 Occurrence of damages

A destructive effect of the water upon freezing can destroy material bonds. However, such changes are, as a rule, of a minor extent, and without any significant influence on the road condition.

Prevailing **damages** to roads upon freezing occur **due to non-uniform frost heaves** resulting from an increased water or ice quantity, generally in the substrate.

Non-uniform frost heaves particularly occur, when the frost penetrates along the road edge less deep than in the middle of the road. This is the case, when

- the heat conductivity of mainly overgrown topsoil is lower than of the pavement, or

- the frost penetration into a sensitive soil is different due to the snow accumulation at the carriageway edges.

Non-uniform or differential frost heaves can also result from locally different pavements, or from a different water inflow into the frost zone.

Frost heaves cause convexities on the pavement surface; in asphalt carriageways such convexities cause open longitudinal cracks; in stiff cement concrete surfacing, these convexities provoke more or less non-uniform lifting of slabs (formation of steps). This unevenness also impairs the carriageway serviceability.

A reduced bearing capacity of the pavement **during thawing** can cause both deformations and cracks under traffic loading. Cracks can be formed as a fine net called alligator cracking, or as larger clods (blocks), and they signify a commencement of pavement destruction.

Days, or even weeks elapse before the original bearing capacity condition is established.

A pavement can also be destructed, when a relatively thin asphalt surfacing placed on a substrate of lower bearing capacity, is exposed to a transient severe frost. Due to bituminous binder hardening caused by a low temperature, and due to thermally induced stresses, the pavement surfacing (crust) can crush and fly out in pieces due to mechanical loading.

2.2.5.2.3 Influence of material sensitivity to freezing

A material shall be assessed as sensitive to freezing, if, due to frost action, ice lenses or layers occur in it, being more or less parallel with the frost line, and causing frost heaves on the carriageway; a material is also sensitive to freezing, when its bearing capacity is reduced upon thawing.

The material sensitivity to freezing depends on the following:

- granulometric composition,
- grain form,
- compaction,
- mineral types in fine grain-groups, and
- mineralogical-chemical properties.

On the basis of the granulometric composition criteria, and mineralogical criteria, materials used in road construction are classified into three classes of their sensitivity to freezing (Table 2).

On the basis of the quotient of non-uniformity of the material grading curve $U = d_{60}/d_{10}$, and on the basis of fines content (of up to 0.063 mm), a more detailed definition of material F1 and F2 sensitivity classes (Diagram 2) has been carried out.

According to the material classification indicated in Table 1, all the coarse-grained aggregates containing up to 5 % by mass of grains of up to 0.063 mm are insensitive to freezing.

Moreover, mixed aggregates, containing up to 15 % by mass of fines, are insensitive or only insignificantly sensitive to frost, if the quotient of non-uniformity of the grading curve amounts to $U = d_{60}/d_{10} \leq 6$.

When the quotient U amounts to 6 - 15, the admissible content of fine particles in the material shall be interpolated between 15 % by mass and 5 % by mass. If mixed materials do not fulfill this criterion, they shall be classified in the frost sensitivity class F2.

Table 2: Classification of materials (aggregates) on the basis of their sensitivity to freezing

Class	Sensitivity	Content of grains up to 0.063 mm % by mass	Classification[1]
F1	insensitive	< 5	GW, GP SW, SP
F2	less to medium sensitive	5 ... 15	GC[2], GM[2] SC[2], SM[2] CL, CH
F3	very sensitive	> 15	SM – ML ML, MH CL - ML

Legend: ali so to angleške oznake?

[1] - *Classification according to the DIN 18 196 and USCS respectively*

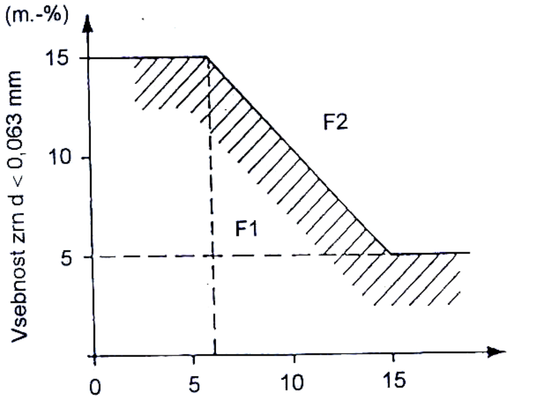
G – gravel

S – sand

M – silt

C – clay

[2] - *Classified into F1, provided that the condition indicated in Diagram 2 is implemented*



m.-% = % by mass

os y = content of grains of d < 0.063 mm (fines)

os x = quotient of material non-uniformity U = ...

Diagram 2: Definition of sensitivity classes F1 and F2 in dependence on the quotient U, and content of fines

The sensitivity of mixed and fine-grained aggregates to freezing results from a complex simultaneous action of

- arising negative pressure upon pore water freezing at the boundary ice - water,
- admixtures of different clay minerals,
- material permeability to water in connection with the compaction rate,
- water mobility,
- plasticity of fines, and
- conditions of material sedimentation (naturally, or destructed by excavation and placing).

Therefore, comprehensive laboratory investiga-tions are required to become familiar with the sensitivity of local materials to freezing, i.e.:

- CBR values after freezing and thawing (CBR_3), and in special cases also
- frost heaves on material samples of different water content.

On the basis of results of measurements according to the CBR_3 method (after freezing and thawing), the materials can be classified in freezing sensitivity classes as indicated in Table 3.

Table 3: Material classification by sensitivity to freezing on the basis of CBR_3 values

	Material sensitivity class		
	F1	F2	F3
CBR_3 value	> 30 %	8 .. 30 %	< 8 %

2.2.5.2.4 Water influence

Damages due to freezing occur as a result of simultaneous action of both frost and water. Therefore, the water itself, as well as its inflow and outflow, are of particular importance to pavement structures.

To preserve the material bearing capacity and to prevent damage to the material sensitive to freezing the latter shall be protected from water penetration, and its drainage shall be made effective. This applies to all the construction methods carried out on a substrate sensitive to freezing.

Depending on the water source, effects of surface water, of water penetrating laterally, and of ground water shall be distinguished.

Through unprotected shoulders and central reserve the precipitation water can seep into the pavement and substrate (substructure, fill, subgrade).

The water penetrating laterally can be successfully and completely led off only by effective deep drainage.

The interdependence of the distance between the frost line and the ground water, and the damage due to freezing is extremely complex. The ground water can be lifted as capillary water very high in cohesive soils of low permeability. However, as the soil permeability is low, the quantity of the lifted capillary water is more or less limited, thus the damage hazard is reduced.

The water influence can be assessed as substantial, when the ground water level is permanently or even periodically less than 2 m below the substructure formation in the freezing period. However, the water influence cannot be entirely excluded even in case that the ground water level is much deeper. Just a small portion of water (moisture) in the substrate material is sufficient to cause water concentration in a form of an ice layer.

2.2.5.2.5 5.2.2.3 Traffic influence

During freezing, the material bearing capacity is increased. Consequently there is no direct danger of damage due to traffic loading.

During thawing, traffic loading can cause damage to the carriageway of insufficient bearing capacity. In such conditions the vehicle axle load is particularly decisive, whilst the traffic density is of minor importance.

2.2.5.3 5.3 Measures against damage

A road shall be protected from damage due to freezing and thawing, when the following two conditions are present at the same time:

- the material is sensitive to freezing (F2, F3)
- the free capillary water, or the water bound by adsorption reaches into the freezing zone, and its quantity is sufficient for ice formation.

Protective measures shall be so planned as to exclude at least one of the two conditions indicated above.

2.2.5.3.1 Reducing frost effects

Frost effects can be reduced, if the snow remains on the carriageway, and mineral strewing materials are used instead of de-icing salt. However, this is only feasible on roads of lower categories (of relative low traffic intensity), on condition that the traffic safety is not jeopardized.

2.2.5.3.2 Drainage

An effective drainage of pavement surface, pavement structure, substructure, fills, and subgrade shall always be ensured. For this purpose, dewatering installation and devices shall be properly maintained.

Damages due to freezing can be reduced or even eliminated by regular road maintenance. Unfortunately, this does not apply to the sources of damages.

For materials, which are less sensitive to freezing, the formation cross fall shall amount to at least 2.5 %, whereas for more sensitive materials (cohesive soils), it shall be 4 % minimum. The greater the hydraulic fall, the shorter the water evacuation time.

Where a pavement surface is not watertight, the precipitation water can penetrate into the substrate thus accelerating damage due to freezing. Suitable waterproofing can also be a provisional maintenance measure.

2.2.5.3.3 Pavement

When executing a pavement, special attention shall be paid to the quality of mineral aggregates for subbases, as provided by the current technical regulations.

The water absorption capacity of an unbound aggregate of grain size above 4 mm shall amount to $\leq$ 0,5 % by mass.

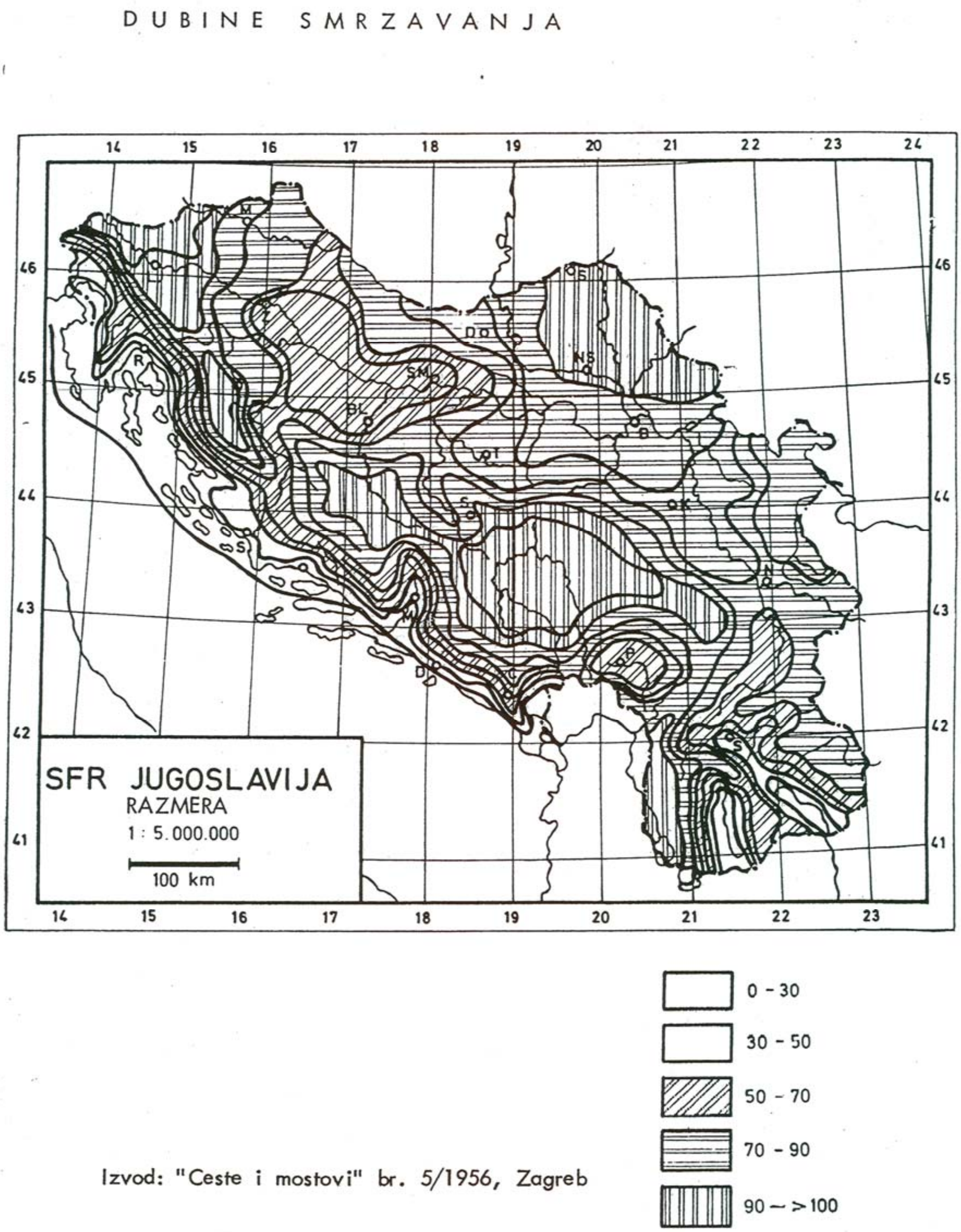
Where a pavement material is insufficiently resistant to freezing and thawing, so that its portion or the complete pavement shall be replaced by an insensitive material, it shall be considered that the heat conductivity of the substitutive material is greater than that of the existing material sensitive to freezing. This means that, after the materials have been substituted, the frost depth is generally increased, which shall be particularly taken into account at stable structures.

2.2.5.3.4 Traffic loading limitation

Roads where materials non-resistant to freezing are placed in the zone of the frost penetration depth for economical or any other reasons, it is possible to protect the road from damage during freezing and thawing to a great extent by limiting vehicle axle loads, or even by a complete road closure for traffic. The duration of such limitations depends on the course of thawing, the extent of substrate softening, and the condition of drainage devices. Necessary information on how the bearing capacity varies by the time can be obtained by harmonized deflection measurements with a Benkelman-beam; during thawing, this shall be carried out daily.

Annex I

Map of informative frost penetration detphs in former SFRJ



Izvod: "Ceste i mostovi" br. 5/1956, Zagreb

2.3 CHARACTERISTICS OF PAVEMENT STRUCTURE MATERIALS

2.3.1 Subject of specification

The present specification provides bases for performing measurements of moduli of deformation, as well as for evaluation of measurement results.

The intention of measurements of moduli of deformation is to specify the course and magnitude of settlement of surface of placed layer consisting of granular material. The characteristics of settlement (both elastic and plastic component) enable an evaluation of bearing capacity, as well as verifying compressibility and compaction of the material placed.

In road construction, measurements of moduli of deformation represent a constituent part of the quality control of executed earth works, and pavement subbases; the moduli of deformation themselves are one of the bases to asses the required dimensions of pavement structures.

Depending of the intention for use, and measurement methods, the following quantities are measured in the road construction:

- static moduli of deformation E_{vs},
- dynamical moduli of deformation E_{vd},
- moduli of compressibility M_E,
- moduli of subgrade reaction k_s, and
- CBR values.

2.3.2 Reference documents

The present specification is based on the following technical reference documents:

BAST – Empfehlungen (E 1), Ausführung von Plattendruckversuchen, BAST, Köln, 1968 (*BAST – Recommendations (E 1), Execution of Plate Bearing Tests, BAST, Cologne, 1968*)

BAST – Empfehlungen (E 4), Ausführung von Plattendruckgeräten, BAST, Köln, 1969 (*BAST – Recommendations (E 4), Performance of Plate Bearing Testing Devices, BAST, Cologne, 1969*)

BAST – Empfehlungen (E 8), Plattendruck-versuch mit Hilfe des Benkelman – Balkens für die Erdbaukontrolle (Ein-Uhr-Messverfahren), BAST, Köln, 1970 (*BAST – Recommendations (E 8), Plate Bearing Test by Means of Benkelman Beam for Earth Works Quality Control, BAST, Cologne, 1970*)

DIN 18 134: 1993 Baugrund, Versuche und Versuchsgeräte, Plattendruckversuch (*DIN 18 134: 1993 Foundation Soil, Tests and Testing Devices, Plate BearingTest*)

Siedeck P in R. Voss, **Die Bodenprüf-verfahren bei Strassenbauten**, BASt, Werner-Verlag, Düsseldorf, 1966 (*Soil Testing Methods in Road Construction*, *BASt, Werner-Verlag, Düsseldorf, 1996*)

SNV 670 316: 1975 Versuche, CBR-Penetrometer, Feldversuch (*Testing, CBR Penetrometer, In Place Test*)

SNV 670 318: 1980 Versuche, Schneller M_E – Versuch (*Testing, Fast M_E – Test*)

SNV 70 312: 1959 Versuche, VSS-Gerät (M_E und CBR) (*Tests, VSS-Device (M_E and CBR)*)

SNV 70 317: 1959 Versuche, Plattenversuch (*Testing, Plate Bearing Test*)

SNV 70 319: 1972 Versuche, Plattenversuch nach Westergaard (*Testing, Westergaard Plate Test*)

TP BF-StB: 1992 B 8.3, Boden /Fels, Prüfung, Dynamischer Plattendruckversuch mit Hilfe des Leichten Fallgewichtsgerätes (*Soil/Rock, Testing, Dynamical Plate Bearing Test by Means of Light Falling Weight Device*)

The specification includes provisions of other publications, either by dated or undated references. For dated references, subsequent supplements or modifications shall be considered, if they are included by a supplement or revision. For undated references the latest edition of the reference publication is valid.

2.3.3 Explanation of terms

The terms in this technical specification shall be understood as indicated below. The terms in bold face are in Slovenian language, whilst the first term in brackets is in English, and the second one in German).

Dynamic modulus of deformation, (dynamischer Verfor-mungsmodul, dinamički deformacijski modul Evd) is a characteristic value of material deformability at specified shock loading of a circular plate by a falling light weight, determined on the basis of the measured amplitude "s" of the plate settlement.

Modulus of subgrade reaction k_s, (Bettungsmodul $k_{s,}$ modul reakcije tla "k_s") is a characteristic value of material deformability at certain loading applied to a circular plate; it is determined on the basis of resulting settlement.

Modulus of compressibility M_E, (Zusammendrückungsmodul $M_{E,}$ modul stišljivosti M_E) is a chracteristic value of material deformability at gradual single loading applied to circular plate; it is determined on the basis of inclination of secant of the settlement curve in a certain range of loading.

Bearing capacity, (Tragfähigkeit, nosivost) is a mechanical resistance of the formation of material built-in to transient loading.

Plate bearing test, (Platten-druckversuch, ispitivanje s pločom) is a test, where the material is loaded with a circular plate and adequate additional equipment, and unloaded; average loading with the plate "p" and corresponding settlements "s" determine the settlement (deformation) curve.

Static modulus of deformation E_{vs}, (statischer Verformungs modul $E_{vs,}$ statički deformacijski modul Evs) is a characteristic value of material deformability at gradual multiple loading of circular plate; it is determined on the basis of inclination of secant of the settlement curve in a certain range of the first, second, or third loading.

California Bearing Ratio, CBR-Wert, vrijednost CBR) is a characteristic value of material deformability at settlement of a pressed-in beetle, determined on the basis of a loading, which causes a settlement specified in advance.

Compaction (degree of), (Verdichtungsgrad, stepen zbijenosti) signifies an attained density of placed material after completion of the compaction procedure.

2.3.4 Bases of measurements

2.3.4.1 Physical bases

2.3.4.1.1 General

In a homogeneous elastic isotropic foundation soil the settlement **s** below a circular plate is defined by the following equation:

$$s = \frac{\pi}{2} \cdot \left(1 - \mu^2\right) \cdot \frac{p \cdot r}{E}$$

where:

μ – *Poisson's ratio (μ = 0.5)*
p – *uniform vertical loading (normal stress σ)*
r – *circular plate radius*
E – *material modulus of elasticity*

The modulus of elasticity **E** of material in a homogeneous foundation soil is, in relation to the modulus of deformation $\mathbf{E_v}$, defined by the equation below:

$$E = \frac{\pi}{3} \cdot \left(1 - \mu^2\right) \cdot E_V$$

In this way the equation for the modulus of deformation $\mathbf{E_v}$ gets its basic form as follows:

$$E_V = \frac{3}{2} \cdot \frac{p \cdot r}{s} = 0{,}75 \frac{p}{s} \cdot D$$

where:

D – circular plate diameter

2.3.4.1.2 Basic methods of measurements

For basic methods, equations to assess characteristic values of deformability of placed material are either adopted directly, or basic forms have been modified:

- **for the static modulus of deformation $\mathbf{E_{vs}}$:**

$$E_{VS} = 0{,}75 \frac{\Delta\sigma}{\Delta s} \cdot D \qquad \left[MN / m^2\right]$$

where:

$\Delta\sigma$ – difference between two assumed levels of vertical loading (=Δp) [MN/m^2]

Δs – difference between two settlements of circular plate at the change of specific loading by Δp [mm]

D – circular plate diameter [mm]

- **for the dynamical modulus of deformation $\mathbf{E_{vd}}$:**

$$E_{vd} = 1{,}5 \cdot r \cdot \frac{\sigma}{s} \qquad \left[MN / m^2\right]$$

where:

σ - mean normal stress below the plate loaded with the maximum force F_s:

$$\sigma = \frac{F_S}{\pi \cdot r^2} \qquad \left[MN / m^2\right]$$

- **for the modulus of compressibility $\mathbf{M_E}$:**

$$M_E = \frac{\Delta\sigma}{\Delta s} \cdot D \qquad \left[MN / m^2\right]$$

- **for the modulus of subgrade reaction $\mathbf{k_s}$:**

$$k_S = \frac{\sigma_o}{s} \qquad \left[MN / m^3\right]$$

- **for the CBR value:**

$$CBR = \frac{\sigma}{\sigma_s} \cdot 100 \qquad [\%]$$

where:

σ – loading for standardized impressing of ram into the investigated material

σ_s – loading for normal impressing of ram into a standard material (crushed stone)

2.3.4.2 Equipment

The measuring equipment to determine substrate settlement by means of a circular loading plate consists of the following three basic components:

- equipment for mechanical loading, i.e.:
- stiff circular loading plate, and
- loading devices,

- equipment to measure settlements, and
- equipment to evaluate the executed measurements.

In addition to the equipment stated above, a suitable counterweight is required for all the measurements with a circular loading plate, except for assessment of the dynamical modulus of deformation.

2.3.4.2.1 General

The stiff loading plate shall be made of steel Če 52.

The plate dimensions depend on the settlement measurement method. The allowable tolerance of plate dimensions must not exceed 1 %.

The lower (bearing) surface of the stiff circular loading plate shall be even and smooth. The mean roughness of the surface shall not be greater than 6.3 μm.

On the upper side of the plate a spirit level shall be fixed. It shall be of such a construction as to allow its horizontal placing on a plate surface inclined by up to 7°.

Two handles shall be fixed to the plate.

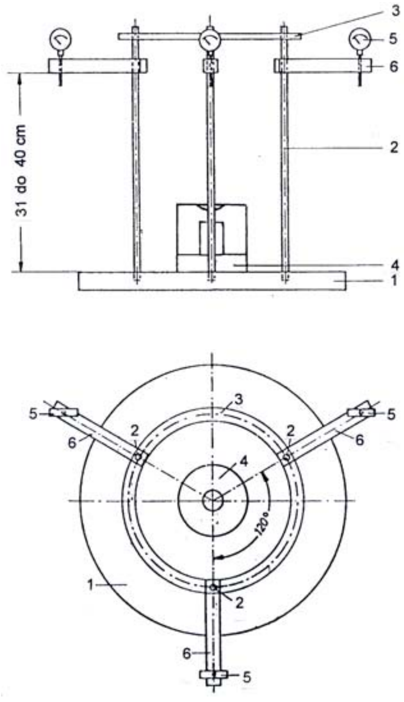
The counterweight mass required for settlement measurements by means of a stiff circular loading plate shall be by at least 1,000 kg greater than the maximum one required to perform the measurement.

2.3.4.2.2 Static modulus of deformation E_{vs}

The scheme of the part of the equipment to assess the static modulus of deformation E_{vs} is shown in Fig. 1.

2.3.4.2.2.1 Circular loading plate

The circular loading plate shall measure 300 mm in diameter, and at least 25 mm in thickness. On the plate three supports to place measuring gauges (2), a linking ring (3), and an additional back-up plate (4) shall be fixed. On the lower side of the latter, there shall be an opening to allow measurements by means of a single measuring gauge (movement measuring device), whereas on its upper side a pan for a hinge shall be provided. For a stable and centrical placing of an additional back-up plate, a suitable groove shall be foreseen on the loading plate.



do = to

Fig. 1: Scheme of a circular loading plate including equipment to measure settlements

2.3.4.2.2.2 Loading devices

Devices to load the circular plate shall enable loading and unloading by degrees. They are composed of the following components:

- oil pressure pumps with a valve to adjust the pressure,
- pressure hose, and
- hydraulic ram.

To achieve a perfect load transfer, a two-sided hinge shall be fixed to the hydraulic ram. If necessary, the ram can be extended by means of adequate elements, however to a length not exceeding 1 m. The ram stroke shall amount to at least 150 mm.

A measuring toll for mechanical and/or electrical measuring of loading belongs to loading devices as well.

2.3.4.2.2.3 Equipment to measure settlements

The equipment to measure settlements depends on the measurement method:

- for measurements carried out at three locations, three measuring gauges (5) of a range of at least 10 mm (20 mm is recommendable), and of reading accuracy (scale) of 0.01 mm are required; the measuring gauges shall be fixed to the accessories on the circular plate (2) fixed by means of suitable handles (6) (Fig. 1); they shall rest on supports fixed to a stable stand (tripod);
- for measurements of settlements at a single place, i.e. in an opening located in the middle of the back-up plate (4) (Fig. 1), in addition to the measuring gauge or electrical measuring tool to measure the movement, a suitable bearing framework is required to place and fix these measuring devices (Fig. 2).

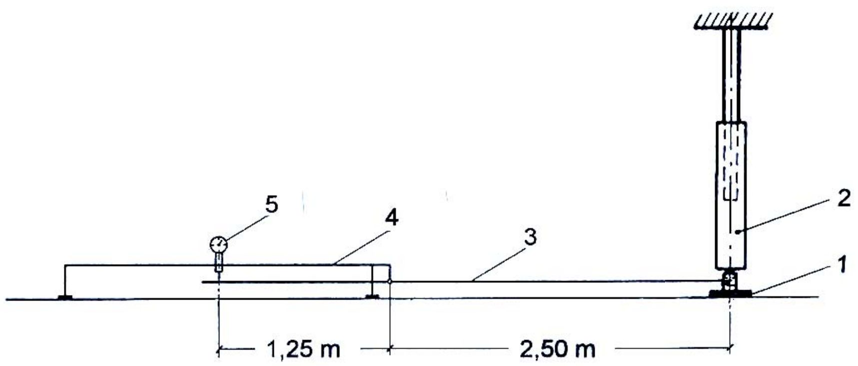


Fig. 2: Scheme of settlement measurements using single measuring gauge (with Benkelman beam)

Legend:

1 - circular loading plate with an additional back-up plate with an opening for measurements
2 - hydraulic ram
3 - measuring arm
4 - bearing framework
5 - measuring gauge

2.3.4.2.3 Dynamical modulus of deformation E_{vd}

A scheme of equipment for assessment of the dynamical modulus of deformation E_{vd} is shown in Fig. 3.

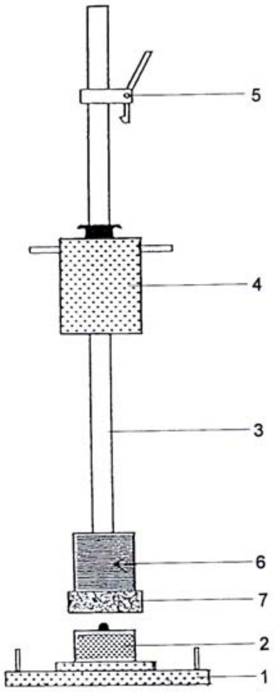


Fig. 3: Scheme of circular loading plate with equipment for dynamical loading, and for settlement measurements

2.3.4.2.3.1 Circular loading plate

A circular loading plate (1) shall measure 300 mm in diameter, and 20 mm (17 mm, 15 mm) in thickness. A casing with a sensor and plug (2) to adjust a guiding bar (3) shall be fixed on the plate.

The mass of the circular loading plate including all elements fixed to it, and including the sensor to measure settlements, shall amount to 15 kg ± 0.25 kg.

2.3.4.2.3.2 Loading devices

Devices to load the circular loading plate shall allow an impulsive (dynamical) loading.

They are composed of the following components:

- a guiding bar (3) over which a free falling weight with a ring-shaped handle (4) can slide; the following components are fixed to the bar:
- on the lower side, a steel circular spring with a casing (6), and a device to prevent overturning (7) are placed,
- on the upper side, a locking device (5) is placed.

Loading devices shall met the following requirements:

- falling weight mass (4)	10 kg ± 0.1 kg
- total mass of guiding bar including steel spring with casing, device to prevent overturning, and locking device	5 kg ± 0.25 kg
- maximum impulse force F_s	7.07 kN
- impulse duration t_s	18 ms ± 2 ms

Spring elements and the weight falling height shall be so adjusted as to ensure the required impulse force F_s of an accuracy of ± 1 % in the temperature range between 0°C and 40°C.

The falling weight (made of Je 52 steel) shall be designed in such a way that it can be intercepted after recoil. A minimum friction between the falling weight and the guiding bar made of polished stainless steel shall be ensured for a long duration.

2.3.4.2.3.3 Equipment to measure settlements

The equipment to measure settlements consists of the following components:

- a sensor with a port for connection; the sensor is fixed to a stiff circular loading plate, and
- an electronic measuring instrument.

The sensor or acceleration measuring device (accelerometer) shall ensure a measuring result accuracy with an error of maximum 2 % in the temperature range between 0°C and 40°C, and in the frequency range between 8 Hz and 100 Hz. The measurement accuracy depends on the settlement range and amounts to:

- in the range between 0.2 mm and 1 mm minimum ± 0.02 mm,
- in the range between 1 mm and 2 mm minimum ± 2 %

The electronic measuring instrument for data recording shall generally be fed from an accumulator (NC block), which must be loaded automatically. Voltage and other characteristics of the electronic equipment for settlement measurement shall be adequately harmonized.

2.3.4.2.4 Modulus of compressibility M_E

2.3.4.2.4.1 Circular loading plate

A circular loading plate shall have an area of 200 cm^2 (D = 15.96 cm), or 700 cm^2 (D = 29.86 cm).

2.3.4.2.4.2 4.2.4.3 Equipment to measure settlements

The equipment for settlement measurement is the same.

2.3.4.2.5 Modulus of subgrade reaction k_s

2.3.4.2.5.1 Circular loading plate

A circular loading plate to measure the modulus of subgrade reaction k_s shall measure 600 mm or 762 mm in diameter. On the plate, ribs to increase its stiffness shall be arranged symmetrically in radii. On the upper side the ribs shall be machined plane-parallel to the contact surface to allow placing the circular plate of diameter of 300 mm onto the ribs. To achieve centrical placing of the plate, plugs and, if necessary, retaining ties shall be fixed to the lower plate.

2.3.4.2.5.2 Loading devices

Devices to load the circular loading plate to measure the modulus of subgrade reaction k_s are the same, and shall, taking account of the circular plate characteristics, meet the same requirements.

2.3.4.2.5.3 Equipment to measure settlements

The equipment to measure settlements shall be, taking account of the circular plate characteristics, similar to the equipment just mentioned.

2.3.4.2.6 CBR value

2.3.4.2.6.1 4.2.6.1 Hydraulic ram

A hydraulic ram for the determination of the CBR values (of predominantly cohesive soils) shall have a contact surface of 20 cm^2, which acts directly on the measured substrate.

Lateral pressing-out of the soil shall be prevented by means of lead cylinders of 150 mm of outer diameter, of 52 mm of inner diameter, and of 10 mm of height. These cylinders shall be put on the hydraulic ram.

2.3.4.2.6.2 4.2.6.2 Loading devices

Considering the fact that the hydraulic ram is directly used to load the measured substrate, the loading devices are the same as defined in 2.3.4.2.2.

2.3.4.2.6.3 Equipment to measure settlements

To assess CBR values, predominantly the same equipment for settlement measurements is required as indicated in 2.3.4.2.3.

It is recommendable to introduce a measuring gauge graduated in inches, so that one circle of the pointer amounts to 1/20 of an inch (approximately 1.25 mm = prescribed impress in a minute), which enables that the second-pointer on a stopwatch, and the pointer of ram impression on the measuring gauge run synchronically.

2.3.5 Measurement execution

The execution of settlement measurement with stiff circular loading plates is determined by the following:

- basic conditions to carry out the measurements, and
- approved measuring methods, including recording measurement results.

2.3.5.1 Conditions to execute measurements

Settlements can be measured with a stiff circular loading plate on the following materials:

- coarse-grained materials,
- mixed materials, and
- cohesive soils of high-plasticity to solid consistency.

In the material, the portion of grains of size greater than 63 mm or more than ¼ of the plate diameter may only be negligible in the material.

In case of fast-drying materials, of single-sized sand, of soils with a crust, of soils, which are softened or soaked for the moment, or of materials, which upper part has changed for any reason, the measurements shall be performed by means of a plate below the changed portion. The density of the tested material shall remain unchanged to the greatest possible extent.

In fine-grained soils (silts, clays) the measurement with a slab can be perfectly executed and evaluated, if the materials are of a high-plasticity to solid consistency. In dubious cases the soil water content shall be assessed, as the latter has a crucial impact on the measurement result. This shall be carried out at different depths up to 3r below the measuring place surface.

The results of measurements carried through with a plate are generally not realistic or applicable, when the measurements have been executed on a frozen material.

2.3.5.2 Measurement procedures

The method of settlement measurement by means of a stiff circular loading plate consists of the following three characteristic stages:

- preparation of surface for the measurement
- placing the measuring
- measurement itself.

2.3.5.2.1 Preparation of surface for measurement

The surface to be measured shall be adjusted to the circular loading plate size.

A suitable evenness of the surface to carry out the measurement shall be ensured by means of suitable tools (steel ruler, trowel). Loose material particles shall be removed by sweeping.

When the substrate inclination exceeds 5°, a horizontal measuring surface shall be ensured as follows:

- in cohesive soils by removing the layer upper part, by removing of exposed coarse particles, and by filling up local cavities with sand or gypsum mash;
- in non-cohesive materials by applying a layer/wedge of sand or gypsum mash up to the horizontal substrate level.

The centre of the surface to be measured shall be preliminarily determined by means of a vertical line below the force application point of the hydraulic cylinder on the counterweight.

2.3.5.2.2 Placing measuring equipment

2.3.5.2.2.1 Measurement of static modulus of deformation E_{vs}

Over its entire contact surface, the circular loading plate shall firmly rest on the prepared substrate. Eventual cavities shall be filled up with a layer of dry, medium-grained sand, or gypsum mash in a thickness of some millimetres.

Where a gypsum mash is used, which is applicable to non-cohesive material only, the contact surface of the circular loading plate shall be lubricated with oil.

The plate shall be placed horizontally to the prepared substrate. By rotating around the vertical axis, and by tapping the plate shall be so impressed into the sand or the gypsum mash as to prevent occurrence of cavities between the plate and the substrate.

The gypsum mash pressed out at the plate edges shall be cut off by means of a trowel prior to hardening.

Before commencement of the measurement the gypsum mash shall harden. This can be verified by monitoring the gypsum swelling, which ceases after hardening, or by notching into the cut gypsum mash.

Then, the hydraulic ram shall be placed in the middle of the loading plate below the counterweight, and secured against overturning. The distance between the loading plate and the counterweight shall amount to at least 75 cm.

A tripod or any other bearing framework for settlement measurements by means of a circular loading plate shall be so placed as to ensure location the supporting legs out of the counterweight influence, as well as out of the influence on the and by the circular loading plate (at least 50 cm away).

The measuring gauges shall be placed perpendicularly to the measured surface, and in such a way as to enable a perfect reading.

To the scope of placing the measurement equipment, a short-term preliminary loading of the circular (approximately 30 seconds with 0.01 01 MN/m^2) belongs as well. After this period, measuring gauges or a movement measuring device shall be set to a zero reading.

If appropriate, the measuring equipment shall be protected from weather actions such as sun radiation, and wind.

The placed measuring equipment as well as the counterweight shall not be exposed to vibrations during measurements.

2.3.5.2.2.2 Measurement of dynamical modulus of deformation E_{vd}

The surface preparation procedure to measure the dynamical modulus of deformation is the same as described in item 2.3.5.2.1, provided that dry, medium-grained sand is used to fill up the cavities.

Onto the placed circular loading plate, the guiding bar shall be placed both centrically and vertically, including all the components to perform the measurement.

To ensure a perfect contact between the circular loading plate and the substrate, a pre-loading shall be executed at the measuring place by three shocks (impulses) of a freely falling weight from a specified (calibrated) height. After each shock (recoil) the weight shall be intercepted, and the equipment to measure settlements shall be introduced.

2.3.5.2.2.3 Measurement of modulus of compressibility M_E

The procedure of placing the equipment to measure the modulus of compressibility is the same as described in item 2.3.5.2.2

To ensure a perfect contact between the circular loading plate and the measured surface, the plate shall be pre-loaded applying 0.02 MN/m^2. In this value the plate and the ram dead weight is included.

2.3.5.2.2.4 Measurement of modulus of subgrade reaction k_s

The procedure of surface preparation to measure the modulus of subgrade reaction is the same as described in 2.3.5.2.1.

The distance between the circular loading plate and the counterweight shall amount to:

- for a plate of diameter D = 600 mm minimum 1.10 m
- for a plate of diameter D = 762 mm minimum 1.30 m

To measure the modulus of subgrade reaction a stiff circular loading plate measuring 762 mm in diameter shall be introduced.

To ensure a perfect contact between the circular loading plate and the measured surface, the plate shall be pre-loaded applying 0.01 MN/m^2. In this value the plate and the ram dead weight is included. The pre-loading shall continue until the difference between settlements measured in the last minute does not exceed 0.05 mm.

2.3.5.2.2.5 Measurement of CBR value

For the circular contact surface of the hydraulic ram (of an area of 20 cm^2), and for lead cylinders located at the ram, suitable evenness of the surface to be measured shall be ensured using adequate tools; in exceptional cases, cavities shall be filled up with dry, medium grained sand.

The dead weight of lead cylinders installed to prevent a lateral pressing-out of the soil shall be similar to the pavement weight to be constructed above the measured surface.

By means of an appropriate tripod or any other suitable bearing framework, as well as by fastenings, a possibility of measuring settlements with a single measuring gauge shall be provided.

2.3.5.2.3 Measurements

The maximum loading upon measurement, and/or the maximum settlement to be achieved depend on the goal of the test, as well as on the material properties, and the circular loading plate size.

When some unusual settlements are noticed, e.g. in terms of a substantial inclination of the loading plate, the material under the loading plate shall be excavated up to a depth equal to the plate diameter, and the findings shall be recorded.

2.3.5.2.3.1 Measurement of static modulus of deformation E_{vs}

When measuring the static modulus of deformation with a stiff circular loading plate, the loading shall increase until the following values are achieved:

- settlement of up to 2 mm, or
- normal stress below the plate of up to 0.5 MN/m^2.

However, the measurement shall already be interrupted at lower stress or smaller settlement, when, on increasing the loading, an excessive remodelling is noticed indicating failure of the material placed.

The loading shall be performed in at least six levels, where the difference between two successive levels is approximately the same for the entire loading range. When it is established during the test that the originally selected intervals between successive loading degrees are too big or too small, they shall be adequately modified.

The transition from one loading level to another shall generally be executed within a minute.

Upon both loading and unloading, the next loading degree (level) may be carried out only after the difference in the settlements read on the individual measuring gauge is not greater than 0.02 mm. At one loading level, the loading shall be constant.

When measurements are performed by means of three measuring gauges, the first reading shall be made 10 seconds prior to expiry of the waiting time. Attention shall be paid to the fact that the plate loading, after the first reading has been carried out, is always increasing in equal time intervals.

When, by mistake, the plate has been loaded higher than it had been planned, the loading must not be reduced; however, this shall be recorded.

As a rule, the loading levels (degrees) shall be as follows:

- for cohesive soil	0.02 to 0.03 MN/m^2
- for mixed material	0.03 to 0.04 MN/m^2
- for gravel	0.05 to 0.06 MN/m^2
- for crushed stone	0.06 to 0.07 MN/m^2

The circular loading plate shall be unloaded in three degrees: to 50 %, 25 %, and 0 % of the maximum loading. After a complete unloading is accomplished, a repeated loading cycle shall be executed, however to the last but one loading level of the first loading cycle.

2.3.5.2.3.2 Measurement of dynamical modulus of deformation E_{vd}

After the settlement measuring equipment is turned on, loading shall be carried through by three shocks (impulses) of a falling weight. The settlement amplitude shall be measured to an accuracy of at least ± 0.02 mm. Attention shall be paid to ensuring the calibrated height of the weight free fall permanently, and to intercepting the weight after recoiling.

2.3.5.2.3.3 Measurement of modulus of compressibility M_E

By a uniform increase of loading of the circular loading plate, the pressure for the first degree (level) shall be created, i.e. 0.05 MN/m^2 (the value read on the pressure gauge shall amount to 0.05 MN/m^2 – pressure due to dead weight).

As soon as the stress for this level is attained, settlements shall be read on the measuring gauges in the following way:

- for cohesive soils after 3, 6, 9 minutes, etc.,
- for non-cohesive materials after 2, 4, 6, 8 minutes, etc.

The read settlements shall be adequately recorded.

The loading for the subsequent level (degree) can start as soon as the settlement after 3 or 2 minutes amounts to less than 0.05 mm.

The loading time required at the first loading level (e.g. 9 minutes) shall be kept for all subsequent levels (degrees) as well.

The following loading degrees (levels) shall be applied:

- on foundation soil and fills by degrees of 0.05 MN/m^2 each up to the final loading of 0.25 MN/m^2

- on substructure by degrees of 0.1 MN/m^2 each, from 0.05 MN/m^2 to the final loading of 0.45 MN/m^2
- on subbase by degrees of 0.1 MN/m^2 each, from 0.05 MN/m^2 to the final loading of 0.55 MN/m^2.

2.3.5.2.3.4 Measurements of modulus of subgrade reaction k_s

The stiff circular loading plate shall be loaded (0.01 MN/m^2) until the settlement change in the last minute is greater than 0.02 mm. The next loading levels (degrees) are 0.04 MN/m^2, 0.08 MN/m^2, 0.14 MN/m^2, and 0.20 MN/m^2. At each loading level it shall be waited until the settlement is reduced to below 0.02 mm/min. Therefore, settlements shall be read every minute. Upon unloading one intermediate level (degree) at 0.08 MN/m^2 is sufficient.

2.3.5.2.3.5 Measurements of CBR values

Through the opening, i.e. through the placed lead cylinders, the hydraulic ram shall be inserted and loaded with 0.1 MN/m^2. Then, it shall be so unloaded that the ram contact surface is touching the substrate. Then, the ram shall be impressed into the substrate with a uniform speed of 1.27 mm per minute. Up to the depth of 2.54 mm, the pressure shall be read every 30 seconds. Afterwards, up to the depth of 5.08 mm, the pressure shall be read every minute.

2.3.6 Measurement evaluation

2.3.6.1 Measurement record

To ensure the required characteristic information on the measurement, a record shall be kept, which shall generally contain the following detailed data:

- information on the measuring place, and
- information on mutual dependence of increasing the loading, and of settlements arisen below the stiff circular loading plate, or below the hydraulic cylinder.

The required information on the measuring place are particularly the following:

- exact location
- material type in the substrate
- weather conditions, temperature
- date and time of measurement performed
- peculiarities

In addition to the abovementioned information, the record shall also comprise the chracteristics of the method (loading plate diameter, type of measurement).

For each loading level (degree), all the settlements shall be recorded, i.e. all the readings on the measuring gauge, or any other measuring equipment introduced. Moreover, average values of settlements at certain loading level shall be evaluated.

Settlements at individual loading levels, and the course of settlement shall be clearly indicated in a graphical form as a rule.

2.3.6.2 Calculation of moduli of deformation

2.3.6.2.1 Static modulus of deformation E_{vs}

The base to calculate the static modulus of deformation E_{vs} is the equation indicated in 4.1.2:

$$E_{VS} = 0,75 \cdot \frac{\Delta p}{\Delta s} \cdot D \qquad \left[MN / m^2 \right]$$

To assess the substrate bearing capacity, the static modulus of deformation E_{vs2} shall be calculated from this equation; for the material compaction estimation, the static modulus of deformation E_{vs1} and the ratio E_{vs2}/E_{vs1} shall be calculated as well.

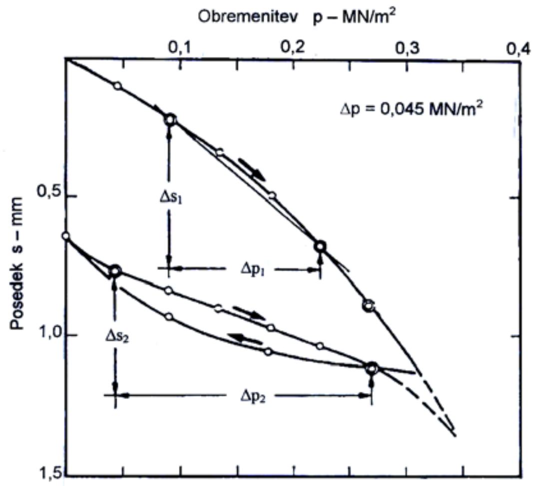
The Δs values shall generally be determined within the range of a uniform course of the substrate settlement at loading in levels (degrees). This range is predominantly the following:

- for the first loading between the 2nd and 5th level, and
- for the second loading between the 2nd and 6th level.

In Fig. 4, values to calculate the moduli of deformation are indicated:

$$E_{vs1} = 0{,}75 \cdot \frac{\Delta p_1}{\Delta s_1} \cdot D \qquad \left[MN/m^2\right]$$

$$E_{vs2} = 0{,}75 \cdot \frac{\Delta p2}{\Delta s2} \cdot D \qquad \left[MN/m^2\right]$$



posedek = settlement; obremenitev = loading

Fig. 4: Diagram of settlements of circular loading plate »s« in dependence on the loading »p«

The indicated limiting values of settlements s and loading p can also be assumed from the particular measurement record.

2.3.6.2.2 Dynamical modulus of deformation E_{vd}

With a dynamical impulse force $F_s \cong 7$ kN the circular loading plate of radius of r = 150 mm, and of area of 700 cm^2, is loaded with a normal stress of σ = 0.1 MN/m^2. According to the fundamental equation for the dynamical modulus of deformation (item 4.1.2)

$$E_{vd} = 1{,}5 \cdot r \cdot \frac{\sigma}{s} \qquad \left[MN/m^2\right]$$

or

$$E_{vd} = 22{,}5/s \qquad \left[MN/m^2\right]$$

it is possible, by means of an electronic measuring instrument, which assesses the maximum settlement by double integration of the measured acceleration, to assess the value of the dynamical modulus of deformation E_{vd} as well. For the assessment, the average value of three tests is relevant.

2.3.6.2.3 Modulus of compressibility M_E

In the fundamental equation for calculation of the modulus of compressibility (item 4.1.2),

$$M_E = \frac{\Delta\sigma}{\Delta s} \cdot D \qquad \left[MN/m^2\right]$$

the value Δs established in the following stress ranges $\Delta\sigma$ shall be considered:

- for foundation soils and fills between 0.05 and 0.15 MN/m^2,
- for substructure between 0.15 and 0.25 MN/m^2,
- for subbase between 0.25 and 0.35 MN/m^2.

2.3.6.2.4 Modulus of subgrade reaction k_s

From the settlement diagram, the stress σ_o shall be determined corresponding to the mean settlement of s = 1.25 mm (Fig. 5).

The modulus of subgrade reaction k_s shall be calculated from the basic equation (item 4.1.2).

$$k_S = \frac{\sigma_o}{s} = \sigma_o / 0{,}00125 \qquad \left[MN/m^3\right]$$

In dependence on the course of the settlement curve, the starting point of the settlement shall be corrected by means of a tangent in the curve turning point.

2.3.6.2.5 CBR value

The CBR values shall be calculated from the basic equation (item 4.1.2), indicated below, by introducing the stress values σ, measured at the ram impression up to the specified depth of 2.54 mm or 5.08 mm, and the standardized values for the crushed stone σ_s:

$$CBR = \frac{\sigma}{\sigma_s} \cdot 100 \qquad [\%]$$

The lower CBR value is shall be considered.

Where the settlement curve course is concave at the beginning of loading, the staring point of the settlement shall be corrected by means of a tangent in the curve turning point.

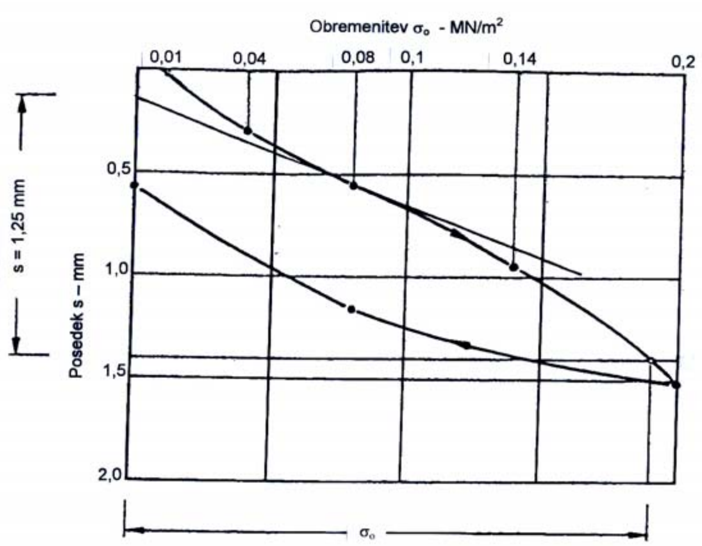


Fig. 5: Diagram of compressive stress »σ_o« under circular loading plate in dependence on assumed settlement »s«

2.4 PAVEMENT STRUCTURE BEARING CAPACITY

2.4.1 Subject of specification

The present specification provides technical bases for evaluation of pavement bearing capacity.

The intention of pavement surface deflection measurement is to define the condition and durability of a pavement.

Pavement surface deflection measurements are suitable particularly to the following:

- establishing conformity and uniformity of executed works in new constructions,
- monitoring condition of existing pavements within the scope of road management,
- assessing actual condition of existing pavements, and
- evaluating adequate strengthening of existing pavements for the design life time

The executed works meet the requirements, if design values of pavement surface deflection are ensured.

To establish the actual condition of an existing pavement, and to specify suitable measures, the following shall be evaluated

- design deflection of pavement surface of a homogeneous road section d_m
- life time of a (strengthened) pavement.

The specification is intended to define the condition of pavements with asphalt surfacing.

2.4.2 Reference documents

The specification is based on the following reference documents:

COST 324, Long Term Performance of Road Pavements, Final Report, EEC, Brussels, 1997

COST 325, New Road Monitoring Equipment and Methods, Final Report, EEC, Brussels, 1997

DYNATEST 8000 FWD Test System – Owner's Manual and Operating Instructions, Technical description

FEHRL Technical note ISSN 1362-6019: 1996 Harmonisation of the Use of the Falling Weight Deflectometer on Pavements; Harmonisation of

FWD measurements and data processing for flexible road pavement evaluation

Merkblatt über Einsenkungsmessungen mit dem Benkelman – Balken, FGSV, Köln, 1991 (*Instructions to Deflection Measurements with Benkleman – Beam*)

SNV 640 330: 1974 Deflektionen, Allgemeines, VSS, Zürich (*Deflections, General*)

The specification includes provisions of other publications, either by dated or undated references. For dated references, subsequent supplements or modifications shall be considered, if they are included by a supplement or revision. For undated references the latest edition of the reference publication is valid.

2.4.3 Explanation of terms

The terms in this technical specification shall be understood as indicated below.

Benkelman-beam, (Benkelman-Balken, Benkelmanova greda) is a measuring device to determine elastic or total settlement of the traffic surface under a vehicle wheel static load of 50 kN as a rule.

Deflectograph, (Deflektograph, deflektograf) is a measuring device for a continuous automatic determination (measurement and record) of total settlements of pavement surface under certain wheel load during a vehicle drive.

Deflectometer, (Deflektometer, deflektometar) is a measuring device for an automatic determination (measurement and record) of pavement surface settlements under certain dynamical loading.

Homogenous section, (homogener Abschnitt, homogen odsjek) is defined by the selected variation coefficient. i.e. the ratio of a standard deviation of measured deflection to their mean value.

Calibrate / adjust, (kalibrieren /justieren, kalibrirati) means to verify suitability of certain properties of equipment, and/or to calibrate them to the required dimension.

Wheel pass, (Radspur, trag točkova) is that area on a carriageway where the traffic frequency is the maximum; there are two wheel passes on one traffic lane.

Rut, (Spurrinne) is a longitudinal gutter occurring in the wheel pass area due to strain acting to the pavement structure and/or to the substrate of the material built-in.

Surfacing, (Decke, zastor) is the pavement upper layer, generally built of a wearing course and bound pavement base-bearing course with a suitable binder.

Design deflection, (mass-gebende Durchbiegung, mjerodavan ugib/defleksija) is a settlement of pavement surface under certain load, taking account of effects on measurement results (corrections).

Modulus of elasticity, (Elastizitätsmodul, modul elastičnosti) is a ratio of normal stress to elastic elongation (under dynamic load).

Nominal axle load, (nominelle Achslast, nazivno (nominalno) osovinsko opterećenje (NOO)) is a standard/nominal single axle load of 81.6 (82) kN, transferred by double wheels of 4 x 20.4 kN to the pavement surface; it is defined as a base to compare effects of different axle loads.

Bearing capacity, (Tragfähigkeit, nosivost) is a mechanical resistance of the formation of material built-in to transient loading.

Deflection,(Durchbiegung, ugib, defleksija) is a surface settlement under certain loading; it is a criterion for the structural condition (available bearing capacity) at the time of measurement; the deflection consists of both elastic and plastic component.

Poisson's ratio, (Poissonsche Querdehnungszahl, Poissonov količnik) is the ratio of the material transversal elongation strain to the longitudinal elongation strain.

Residual lifetime, (Restlebensdauer, preostali period trajanja) is a period between the measurement carried through (e.g. of pavement surface deflection) and fatigue (failure) of the material placed/built-in.

Durability, (Dauerhaftigkeit, trajnost) is a period between placing and fatigue (failure) of the material placed, e.g. to the pavement structure.

Pavement structure, (Fahrbahnbefestigung, kolovozna konstrukcija) is a part of traffic surface consolidation, consisting of one or more bearing courses and a wearing course.

Pavement surface, (Fahrbahn-oberfläche, vozna površina) is a uniformly and continuously consolidated surface of the pavement wearing course, on which the traffic is running.

2.4.4 Basic methods of deflection measurements

Approved methods of existing pavement deflection measurements are based on static or dynamical loading of the measuring spot.

The basic methods are defined as pavement surface deflection measurements performed by

- Benkelman-beam (under static load),

- Lacroix deflectograph (under rolling load), and
- Dynatest 8000 FWD deflectometer (with falling weight – under dynamical load).

For special purposes and under special conditions the pavement surface deflection can also be determined by introducing some other method, e.g. by means of an optical deflectometer, measuring sound (built-in into the pavement structure), vibrators (to measure oscillation amplitudes), etc.

Different loadings specified for the abovementioned basic measurement methods, result in different deflection values, which are not mutually comparable directly.

To establish the condition of an existing pavement, and to specify suitable measures, the following shall be assessed by approved methods of deflection measurement:

- design deflection of pavement surface on a homogeneous road section d_m
- pavement structure life time.

2.4.5 Deflection measuring equipment

The pavement surface deflection measuring equipment shall be such as to ensure:

- repeatability,
- accuracy of deflection measurement and recording in either graphical or digital record, and
- record durability.

All the deflection measuring equipment shall comply with the specified technical characteristics, and shall be calibrated according to a suitable method generally prescribed by the equipment producer; moreover, the equipment shall have a valid certificate.

2.4.5.1 Benkelman-beam

Benkelman-beam is a mechanical measuring device transferring vertical movements (deflection) of a pavement surface to a measuring clock (Fig. 1). It consists of the following components:

- transportable or mobile stand with three supports; the height of these supports is adjustable;
- vertically movable sensor arm, which can be folded and blocked;
- measuring clock (of diameter Ø 100 mm), of measuring range of 30 mm, and with scale division of 0.01 mm;
- adjustable vibrator to eliminate friction of the sensor arm, and in the measuring clock point.

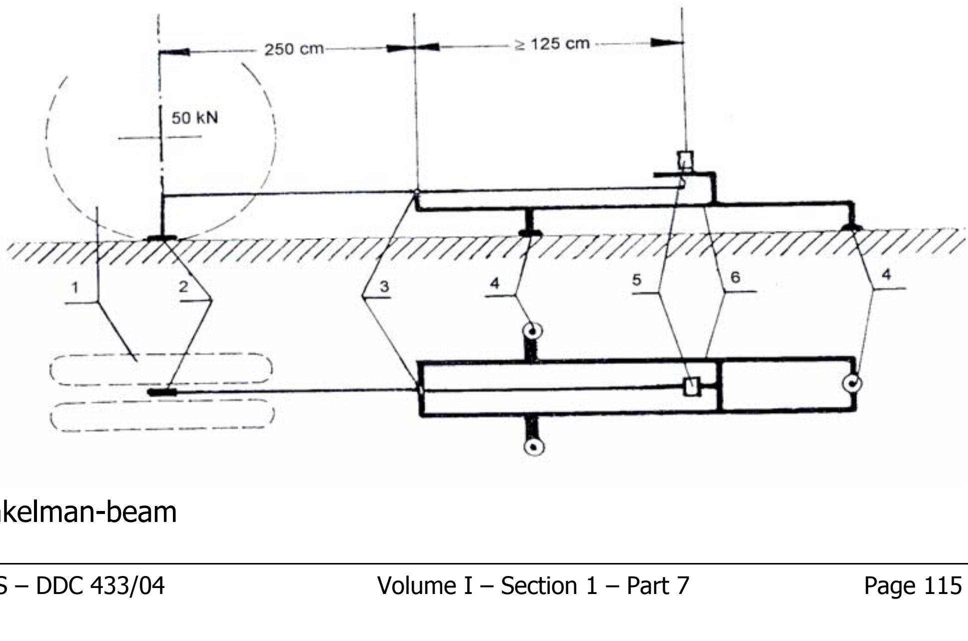


Fig. 1: Benkelman-beam

Legend:
1 – position of wheels
2 – sensor arm point
3 – supporting hinge
4 – adjustable support
5 – measuring clock
6 – stand

The length of the sensor arm from the sensor point to the bearing (250 cm), and from the bearing to the measuring clock (125 cm) shall generally be in the ratio of 2 : 1 (or 1 : 1). The distance of the sensor point from the adjacent supports shall amount to at least 270 cm.

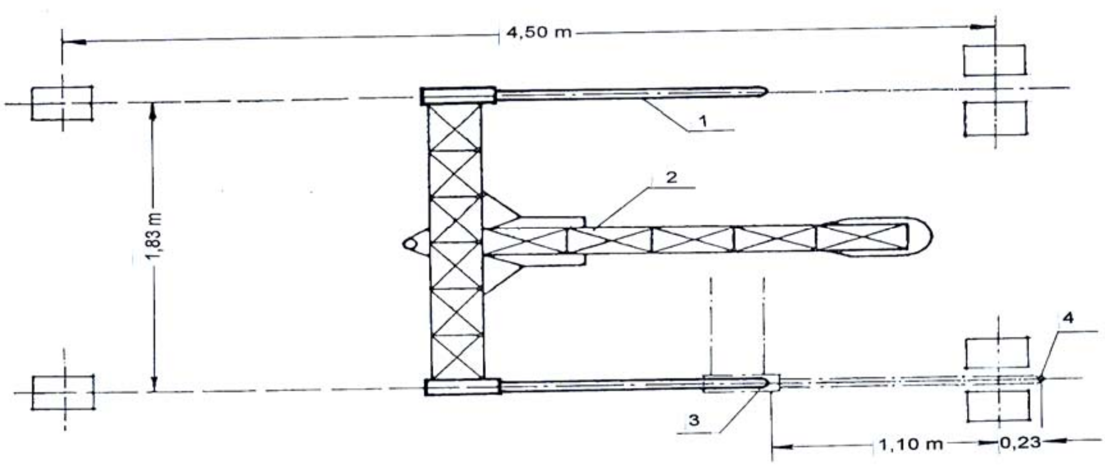
During any movement of deflection measuring device the sensor arm of the Benkelman-beam shall be folded and blocked.

Instead of a measuring clock a precise electronic measuring device, e.g. an inductive measurer of movements, can be used.

To load a measuring spot, a twin-axle lorry having two double wheels on the rear (measuring) axle is required. The spacing of inner edges of pneumatic tyres shall amount to 90 – 140 mm. The axle load shall be 100 kN. Prior to and after measurements the axle load shall be checked by means of a suitable weighing machine. In case that the wheel load deviates from the value of 50 kN, the measured deflection values shall be adequately corrected. The air pressure in the rubber tyres shall be the same amounting to approximately 0.7 MPa, however not less than 0.45 MPa.

To measure the asphalt surfacing temperature (as a rule, up to a depth of 4 cm), electric measuring devices with a sensor and a range between 0°C and 50°C are particularly suitable. The temperature measurement accuracy shall amount to $\pm$ 1°C.

A Benkelman-beam shall ensure an accuracy of the pavement surface deflection measurement of up to ± 0.05 mm.



Legend:
1 – sensor arm
2 – mobile bearing framework
3 – starting position of the sensor arm
4 – final position of the sensor arm

Fig. 2: Lacroix deflectograph – measuring equipment

2.4.5.2 Lacroix deflectograph

A Lacroix deflectograph consists of the following basic components:

- a lorry, which
- is the carrier of the measuring equipment, and
- represents a load to carry out the measurement,
- measuring equipment consisting of
- fixed bearing framework,
- mobile bearing framework with two sensor arms (Fig. 2),
- two inductive measurers of movements of sensor arms,
- computer aided system to monitor measurements, to control the movement of the mobile bearing framework, to record the deflection automatically, and to deliver the data to the computer,
- a computer with appurtenant software to record all the required information on measurements and results.

The equipment for an automatic recording of measurement results enables an electronic record or a record on a paper tape.

To load a measuring spot, the lorry rear axle shall be equipped with two double wheels. The axle load shall amount to 100 kN. It can be adjusted by regulating the water quantity in the tank placed on the lorry.

The pressure in rubber tyres on the measuring axle shall be the same amounting to 0.7 – 0.8 MPa.

To measure the asphalt surfacing temperature, a termometer of a range between 0°C and 50°C shall be used.

Deflectograph calibration shall be performed in compliance with the producer's instructions. This shall be done prior to each measurement, and for each sensor arm individually.

By calibrating the deflectograph, the accuracy of the ratio of sensor arm point movement to the recorded movement shall be ensured. Maximum admitted deviation amounts to ± 0.02 mm.

The adopted measuring equipment shall ensure the accuracy of deflection measurement within the range of up to ± 0.05 mm, and the accuracy of the distance measurement (between measuring spots and in total) of up to ± 3 %.

2.4.5.3 Dynatest 8000 FWD deflectometer

A Dynatest 8000 FWD deflectometer (Falling Weight Deflectometer) is composed of the following constituent parts:

- as a rule, a single-axle trailer (Fig. 3), which is
- the carrier of the equipment for dynamical loading (free falling weights, sensors, system of rubber springs, circular loading plate with a load cell, and
- the carrier of the measuring equipment (deflection measurers – geophones),
- a computer aided system to monitor the measurements, and to deliver the data to the computer, and:
- a computer with appurtenant software to control the entire procedure, and the equipment for recording and processing of all the required information on deflection measurements, and measurement results.

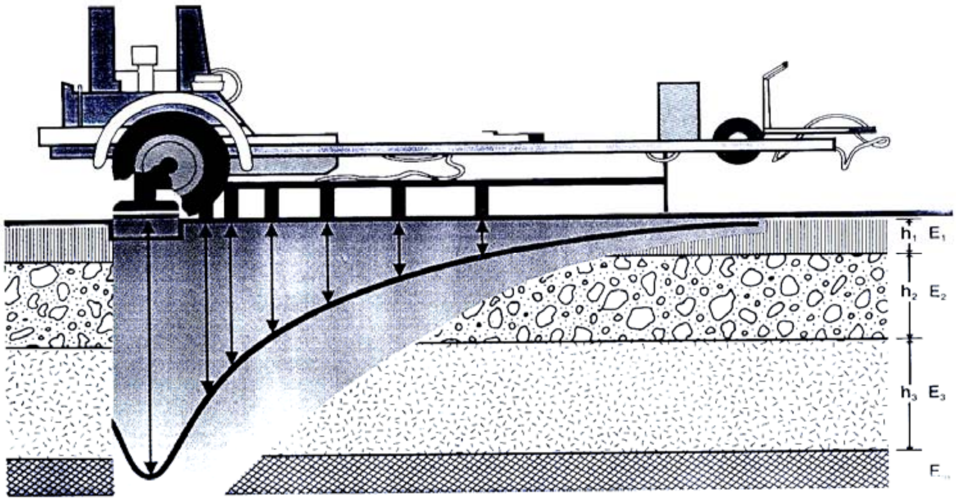


Fig. 3: Dynatest 8000 FWD deflectometer

Electrical – hydraulic equipment shall enable dynamical loading of a measuring spot in the range of 7 to 120 kN. A system of rubber springs above the loading plate shall ensure a sinusoidal form of the loading. The required technical characteristics of the loading control are as follows:

- accuracy: $<$ 0.5 %
- repeatability: $\pm$ 0.1 %
- time of load increase: 5 to 30 ms
- time of recording deflection: 20 to 60 ms

The technical characteristics of the loading plate are as follows:

- diameter: 30 cm
- thickness: 2 cm
- base: 5 mm thick deformed rubber

The technical characteristics of deflection measurers – geophones are as follows:

- deflection recording range: up to 2 mm
- accuracy: $<$ 2 % $\pm$ 1 μm
- repeatability: $\pm$ 2 μm $\pm$ 1 %

An absolute calibration of deflection measuring devices shall be carried out once a year, whereas a relative calibration after every 10,000 measurements (with an admissible deviation of $< \pm$ 1 %).

The dynamical range of deflection measuring devices shall be checked prior to each individual measurement.

General conditions to place deflection measurers – geophoes are as follows:

- placing: at a length of up to 250 cm
- number: 6 to 9
- standard spacing: 30 cm

The arrangement of deflection measuring devices from the centre of the loading plate depends on the pavement condition and on pavement surface deflection d_{mD}:

- $d_{mD} \leq 500$ μm:

arrangement at 0-30-60-90-150-210 cm

- 500 μm $< d_{mD} \leq 1000$ μm:

arrangement at 0-30-60-90-150-180 cm

- $d_{mD} > 1000$ μm:

arrangement at 0-30-60-90-120-150 cm

The temperature measuring device shall meet the following requirements:

- working range: - 10°C to 60°C
- divisibility: > 0.5°C
- accuracy: ± 1°C

2.4.6 Deflection measurement execution

2.4.6.1 Preparations for measurements

Prior to commencement of deflection measurements, and during performing the measurements, safety of the working staff, as well as of the traffic participants shall be ensured.

The entire measuring equipment shall be prepared and calibrated in compliance with clause 2.4.5 of this specification.

Prior to commencement of pavement surface deflection measurement, all the foreign matter shall be removed from the surface.

At the asphalt carriageway edge, holes to measure the surfacing temperature shall be prepared. Prior to measurement, they shall be filled up with glycerine.

Each individual deflection measurement shall be documented by means of the following data:

- place of measurement: road marking, mileage marking, location on traffic lane, type of wearing course, particularities
- date and duration time of measurement
- pavement structure
- asphalt surfacing temperature
- used measuring equipment:load type and characteristics
- design/maximum loading
- design deflection value including all corrections.

To assess the actual condition, pavement surface deflection shall be measured to an extent, which is, in view of the intention and the used measuring equipment, relevant to the entire measured pavement surface.

Pavement surface deflection measurements shall be carried out particularly in the outer rut, which is, as a rule, the most loaded one.

2.4.6.2 Benkelman-beam

Benkelman-beam enables measuring of the following:

- the total, i.e. both elastic and plastic deflection of a pavement surface (“at loading arrival” method), and only
- the elastic deflection (“at loading departure” method), which is predominantly considered to assess the pavement actual condition.

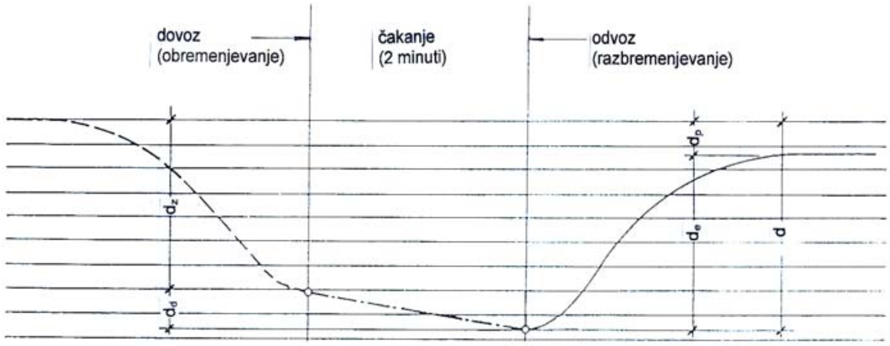
2.4.6.2.1 Measurement method

A Benkelman-beam shall be so placed onto the selected measuring spot as to ensure proper contact of all the three supports of the stand, and to achieve transverse horizontality of the stand. Prior to measuring deflection, the asphalt surfacing temperature shall be measured.

On measuring the deflection in accordance with the “at loading arrival” method, the lorry shall drive backwards with a speed of 0.5 m/s, and shall carefully approach the sensor point on the arm – Benkelman-beam. The pair of rear wheels of the lorry shall be, at the beginning of the measurement, 3 m faraway from the sensor point. During the test, at certain distances of the lorry rear axle from the sensor point (2, 1, 0.5, and 0.25 m), and when the sensor point is in the axis of the lorry rear wheels, deflection values shall be read on the measuring clock. After expiry of approximately two minutes, the lorry shall drive back to the starting point with a speed of approximately 0.5 m/s. Here, the deflection shall be measured when the rear axle is 1 m and 3 m far from the sensor point. The measurement procedure is schematically presented in Fig. 4.

When measuring the pavement surface deflection according to the "at loading departure" method, the rear axle of the lorry, i.e. both loading wheels, shall be placed onto the measuring spot (Fig. 5 - point A).

On a certain measuring spot on the asphalt surfacing, the lorry representing the load may stay maximum one minute.



Legend:

d – total deflection on certain measuring spot

d_z – starting deflection upon loading

d_d – additional deflection during waiting under load

d_e – elastic deflection

d_p – plastic deflection

Fig. 4: Schematic presentation of measurement of pavement surface total deflection

The sensor arm shall be pushed between the pair of loading wheels, so that the sensor point, after the lock-up has been released, comes in contact with the surface some centimetres before the wheel axis (in the lorry driving forward direction).

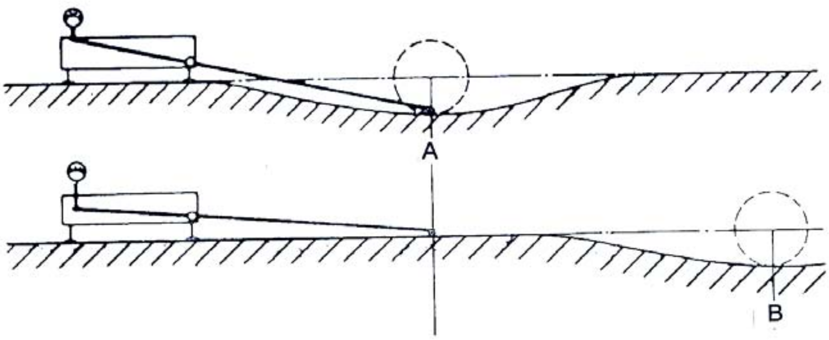


Fig. 5: Schematic presentation of pavement surface elastic deflection measurement with Benkelman-beam

After the vibrator is turned on and adequately set, the value shall be read on the measuring clock, and the lorry shall drive approximately 10 m ahead from the measuring place (point B), and the condition on the measuring clock shall be re-read, after the pointer has come to a standstill.

2.4.6.2.2 Evaluation of results

On the basis of results of deflection measurements with Benkelman-beam the following can be assessed:

- the design deflection, and
- the residual life time of the pavement, and an eventually required overlay.

The difference between the readings on the measuring clock, when the measuring spot is loaded with certain axle or wheel load, and when the measuring spot is unloaded or has come to standstill again, represents the basis for calculation of the pavement surface deflection on the particular measuring spot. The established deflection values shall be indicated in 0.01 mm.

Any eventual deviation

- of wheel load from the standard one (50 kN), and
- of surfacing temperature from the standard one (20°C),

as well as the impact of the critical season on the pavement surface design deflection shall be evaluated with suitable correction factors.

As the influence of the carriageway longitudinal fall on the loading is relatively insignificant (approximately ± 1 % at longitudinal fall of 8 %), comparative measurements in the same drive direction shall be carried out at greater longitudinal falls.

2.4.6.2.2.1 Wheel load effect

Factors of wheel loading effect k_{ko} (in the range of 30 to 70 kN) on the calculation of pavement surface deflection are indicated in Table 1. The indicated values take account of the following:

- the ratio of 2 : 1 of the vertical movement of the sensor point on the sensor arm to the measuring clock point, and
- the conversion of the deflection values read on the measuring clock into millimetres.

The maximum deflection d_i shall be calculated from the following equation:

$$d_i = k_{ko} \times (d_{To} - d_{Tr})$$

where:

d_{To} - reading on the measuring clock scale under load

d_{Tr} - reading after unloading

2.4.6.2.2.2 Temperature effect

To calculate the asphalt surfacing temperature effect (in the range of 5°C to 30°C) on the pavement surface deflection, correction factors k_T are indicated in Table 2. At temperatures out of the abovementioned range, deflection measurements are not appropriate.

The mean asphalt surfacing temperature shall be calculated from the following equation:

$$T_m = \frac{5T_o + (h-5)T_{10}}{h}$$

where:

- T_o - temperature on pavement surface (°C)

- T_{10} - temperature at depth of 10 cm (°C)

- h - asphalt surfacing thickness

Table 1: Factors of wheel load effect k_{ko} on calculation of pavement surface deflection

Wheel load [kN]	Factor k_{ko}	Wheel load [kN]	Factor k_{ko}	Wheel load [kN]	Factor k_{ko}	Wheel load [kN]	Factor k_{ko}
30.0	0.0333	40.0	0.0250	50.0	0.0200	60.0	0.0167
30.5	0.0328	40.5	0.0247	50.5	0.0198	60.5	0.0166
31.0	0.0323	41.0	0.0244	51.0	0.0196	61.0	0.0164
31.5	0.0318	41.5	0.0241	51.5	0.0194	61.5	0.0163
32.0	0.0312	42.0	0.0238	52.0	0.0192	62.0	0.0161
32.5	0.0308	42.5	0.0235	52.5	0.0190	62.5	0.0160
33.0	0.0303	43.0	0.0232	53.0	0.0189	63.0	0.0159
33.5	0.0298	43.5	0.0230	53.5	0.0187	63.5	0.0158
34.0	0.0294	44.0	0.2227	54.0	0.0185	64.0	0.0156
34.5	0.0290	44.5	0.0225	54.5	0.0183	64.5	0.0155
35.0	0.0286	45.0	0.0222	55.0	0.0182	65.0	0.0154
35.5	0.0282	45.5	0.0220	55.5	0.0180	65.5	0.0153
36.0	0.0278	46.0	0.0217	56.0	0.0179	66.0	0.0151
36.5	0.0274	46.5	0.0215	56.5	0.0177	66.5	0.0150
37.0	0.0270	47.0	0.0213	57.0	0.0175	67.0	0.0149
37.5	0.0267	47.5	0.0211	57.5	0.0174	67.5	0.0148
38.0	0.0263	48.0	0.0208	58.0	0.0172	68.0	0.0147
38.5	0.0260	48.5	0.0206	58.5	0.0171	68.5	0.0146
39.0	0.0257	49.0	0.0204	59.0	0.0169	69.0	0.0145
39.5	0.0253	49.5	0.0202	59.5	0.0168	69.5	0.0144

Table 2: Factors of temperature effect k_T on calculation of deflection of pavement with asphalt surfacing (of thickness h)

<table>
<tr><th rowspan="3">Mean temperature of asphalt surfacing T_m [°C]</th><th colspan="3">Asphalt surfacing thickness h</th></tr>
<tr><th>5 to 10 cm</th><th>10 to 20 cm</th><th>20 to 30 cm</th></tr>
<tr><th colspan="3">Factor k_T</th></tr>
<tr><td>5</td><td colspan="3">1.50</td></tr>
<tr><td>6</td><td colspan="3">1.335</td></tr>
<tr><td>7</td><td colspan="3">1.265</td></tr>
<tr><td>8</td><td colspan="3">1.205</td></tr>
<tr><td>9</td><td colspan="3">1.165</td></tr>
<tr><td>10</td><td colspan="3">1.135</td></tr>
<tr><td>11</td><td colspan="3">1.11</td></tr>
<tr><td>12</td><td colspan="3">1.09</td></tr>
<tr><td>13</td><td colspan="3">1.075</td></tr>
<tr><td>14</td><td colspan="3">1.06</td></tr>
<tr><td>15</td><td colspan="3">1.05</td></tr>
<tr><td>16</td><td colspan="3">1.04</td></tr>
<tr><td>17</td><td colspan="3">1.03</td></tr>
<tr><td>18</td><td colspan="3">1.025</td></tr>
<tr><td>19</td><td colspan="3">1.02</td></tr>
<tr><td>20</td><td>1.000</td><td>1.000</td><td>1.000</td></tr>
<tr><td>21</td><td>0.985</td><td>0.975</td><td>0.99</td></tr>
<tr><td>22</td><td>0.98</td><td>0.955</td><td>0.975</td></tr>
<tr><td>23</td><td>0.975</td><td>0.94</td><td>0.955</td></tr>
<tr><td>24</td><td>0.975</td><td>0.925</td><td>0.935</td></tr>
<tr><td>25</td><td>0.97</td><td>0.91</td><td>0.915</td></tr>
<tr><td>26</td><td>0.97</td><td>0.895</td><td>0.89</td></tr>
</table>

Mean temperature of asphalt surfacing T_m [°C]	Asphalt surfacing thickness h 5 to 10 cm	10 to 20 cm	20 to 30 cm
	Factor k_T		
27	0.97	0.88	0.87
28	0.97	0.865	0.845
29	0.97	0.85	0.825
30	0.97	0.835	0.80

The value of pavement surface deflection d_{20}, i.e. evaluated for the temperature of 20°C, shall be calculated for pavements with asphalt surfacing, and

- unbound base bearing course from the equation

$d_{20} = d_i \times k_T$ (mm)

- hydraulically bound base bearing course from the equation

$d_{20} = d_i + k_h$ (mm)

where:

- k_h - correction value indicated in Table 3.

Table 3: Correction value of effect of pavement structure (hydraulically bound base bearing course) on calculation of pavement surface deflection at different temperatures

Asphalt surfacing mean temperature Tm [°C]	Correction value k_h [mm]
5	0.05
6	0.05
7	0.05
8	0.04
9	0.04
10	0.04
11	0.03
12	0.03
13	0.03
14	0.02
15	0.02
16	0.01
17	0.01
18	0.00
19	0.00
20	0.00
21	- 0.01
22	- 0.02
23	- 0.02
24	- 0.03
25	- 0.03
26	- 0.04
27	- 0.05
28	- 0.05
29	- 0.06
30	- 0.06

2.4.6.2.2.3 Season effect

The season effect on the calculation of pavement surface deflection depends particularly on the following:

- carriageway condition,

- sensitivity of materials used in frost areas to adverse freezing effects, and
- climatic and hydrological conditions.

Informative values of factors *c* of the season effect are presented in Table 4.

Table 4: Factors *c* of season effects on calculation of deflection of pavement with asphalt surfacing

Factor c	Characteristic conditions to assess informative values
1.0	Measurement carried out in period of lowest bearing capacity (at thawing)
1.1 – 1.2	Pavement not sensitive to frost effects, climatic and hydrological conditions favourable
1.2 – 1.4	Pavement contains a bearing layer of unbound mineral aggregate of moderate sensitivity to frost effects, climatic and hydrological conditions favourable
1.6 – 2.0	Pavement surfacing cracked; pavement contains a bearing layer of unbound mineral aggregate of medium sensitivity to frost effects, climatic and hydrological conditions unfavourable.

2.4.6.2.2.4 Determination of homogeneous sections

A condition for a homogeneous pavement section in view of the deflection is the variation factor k_v to be calculated from the equation below:

$$k_v = \frac{s}{\overline{d}} \leq 0{,}35$$

where:

- s – standard deviation of deflection value calculated from the following equation:

$$s = \sqrt{\frac{d_{20}^2 - \overline{d}\sum d_{20}}{n-1}}$$

- $\overline{d}$ - mean deflection value calculated from the equation below:

$$\overline{d} = \frac{\sum d_{20}}{n}$$

The length of a homogeneous section shall generally not be less than 100 m in settlements, and 200 m out of settlements respectively.

2.4.6.2.2.5 Determination of design elastic deflection

The design elastic deflection of a pavement with asphalt surfacing assessed on the basis of results of deflection measurements carried out with Benkelman-beam, shall be calculated from the following equation:

$$d_{mB} = c \cdot \left(\overline{d} + k_{pr} \cdot s\right)$$

where:

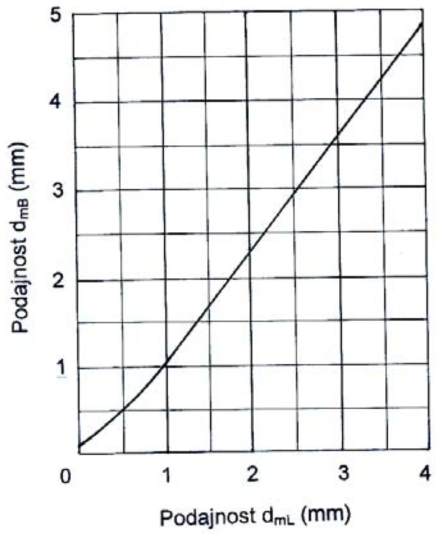
- k_{pr} - factor depending on the road type and required safety degree, amounting to:

- *k_{pr} = 2.0: for motorways, expressways, and main roads (heavy traffic)*
- *k_{pr} = 1.6: for regional roads (medium traffic)*
- *k_{pr} = 1.3: for local roads (light traffic)*

The design elastic deflection d_{mB} is the base to assess the pavement actual condition.

In the procedure of assessing the pavement condition, and determining eventually required measures to settle the situation, the deflection d_{mL} established with a Lacroix deflectograph (TSC 06.541 is specified as relevant.

The correlation of deflection values evaluated on the basis of results of measurements with a Benkelman-beam (d_{mB}), and with a Lacroix deflectograph (d_{mL}), is indicated in Fig. 6.



podajnost = deflection

Fig. 6: Correlation of deflection values evaluated on the basis of results of measurements with a Benkelman-beam (d_{mB}), and with a Lacroix deflectograph (d_{mL})

2.4.6.3 Lacroix deflectograph

A deflectograph is a measuring device for continuous automatic assessment (measurement and recording) of a pavement surface deflection under certain wheel loading during driving.

The device is so conceived as to adopt – same as it applies to the Benkelman-beam – the "at loading arrival" principle.

The deflection measuring method with the Lacroix deflectograph enables assessment of the pavement actual condition.

2.4.6.3.1 Measurement method

A deflectograph, i.e. a lorry carrying the measuring equipment, shall be, prior to commencement of measuring works, set with its rear axle onto the beginning of the road section to be measured. Both sensor arms with the mechanism for recording the pavement surface deflection shall be released, so that they can rest properly on the surface. The sensor arms are in this position approximately 100 cm far from the middle of the lorry rear axle (Fig. 7). Before starting the deflection measurement, the asphalt surfacing temperature shall be measured.

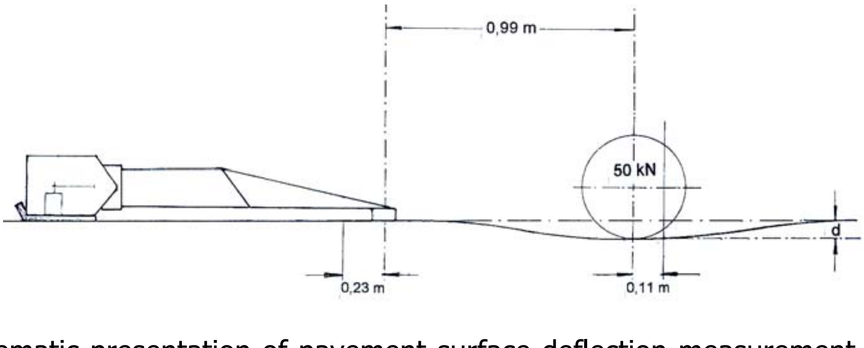


Fig. 7: Schematic presentation of pavement surface deflection measurement with Lacroix deflectograph

The entire procedure of measuring pavement surface deflection with a Lacroix deflectograph is electronically controlled (via computer keyboard).

During the measuring procedure, when the lorry is moving continuously with a speed of approximately 2 km/h, the following steps are performed alternately:

- bearing framework movement control; movement between 3.50 m and 5.50 m in length (into the starting position for the next measurement cycle);
- movement of lorry rear wheels towards the sensor arm point, which follows settlements under rear wheel load;
- automatic recording of deflection by means of two inductive devices to measure movements of sensor arms.

The results of deflection measurements carried out with a Lacroix deflectograph are recorded in digital form.

2.4.6.3.2 Evaluation of results

The following can be assessed on the basis of results of deflection measurements with a deflectograph:

- design deflection,
- residual life time of the pavement, as well as an eventually required overlay.

The procedure of evaluation the results of pavement surface deflection d_{mL} with a Lacroix deflectograph is basically the same as in case of measurements carried out with a Benkelman-beam. All the effects discussed in item 6.2.2 should be considered to a suitable extent, including determination of both homogeneous road sections and design deflection.

2.4.6.4 Dynatest 8000 FWD deflectometer

Pavement surface deflection measurements performed with a Dynatest 8000 FWD deflectometer shall be carried out in compliance with individual detailed instructions of the measuring equipment producer.

Pavement surface deflection measurements carried out with a deflectometer with falling weight enable an assessment of the measuring spot characteristics (actual condition, durability), as well as the actual condition of the pavement structure materials.

2.4.6.4.1 Measurement method

The pavement surface deflection measurement method is based on dynamical loading of a circular slab with a falling weight. Both time course and loading force shall be similar as in case of the loading with a lorry wheel.

Prior to commencement of the deflection measurement the following shall be carried out:

- ensuring a perfect resting of the loading plate and all the deflection measuring devices on the pavement surface;
- measuring asphalt surfacing temperature, and
- setting individual basic and eventual additional parameters to the measurement (method, loading, number of measuring devices).

Deflection measurements with a deflectometer shall generally be executed in the middle of the loading plate, as well as on 6 spots within the range of the measuring equipment bearing framework. The three outer measuring devices, which are more distant from the loading plate, shall be placed at distances generally greater than the equivalent thickness of the pavement structure. The distance between individual measuring spots shall be determined in view of the measurement purpose, and shall amount up to 50 m for planning suitable measures, and up to 200 m for carriageway management purposes.

The entire procedure of pavement surface deflection measurement by means of a deflectometer with a falling weight is electronically controlled via computer. In the computer all the measurement data, which correctness shall be verified in accordance with a special program, shall be stored.

2.4.6.4.2 Evaluation of results

The following can be assessed on the basis of results of pavement surface deflection measurements by means of a falling weight deflectometer:

- design deflection d_{mD},
- dynamic moduli of elasticity of pavement layer material, and
- residual life time of the pavement, and eventually required overlay.

2.4.6.4.2.1 Design deflection

To assess the design deflection of a pavement with asphalt surfacing, dynamical loading of the plate equivalent to a wheel load of 50 kN shall be ensured.

The value established by the third falling weight test shall generally be taken as design deflection.

The temperature effect on the pavement surface deflection shall be considered in evaluating the modulus of elasticity. The season effect on the pavement surface deflection measured by means of a deflectometer, as well as on further evaluations of the pavement condition characteristics is not taken into account.

2.4.6.4.2.2 Moduli of elasticity of pavement layers

To evaluate dynamical moduli of elasticity of pavement material layers, which indirectly characterize the pavement condition, the pavement structure shall be defined. The calculation of thicknesses of equivalent layers is based on the Odemark's "semi-space" theory, and on special ELMOD software. The following additional input data are required:

- plate loading (kN/m^2)
- diameter of loading plate (standard diameter amounts to 30 cm)
- number of pavement layers (maximum 4)
- thicknesses of individual layers ($h \geq 6$ cm)
- Poisson number of materials placed
- spacing of deflection measuring devices – geophones.

2.4.6.4.3 Residual life time

On the basis of results of pavement surface deflection measurements by means of a deflectometer, and of the evaluated dynamical moduli of elasticity of pavement layers, the special ELMOD software also enables a calculation of the pavement residual life time for the foreseen traffic loading, i.e. the number of passages of the nominal axle load of 82 kN, as well as a determination of eventually required overlay or strengthening.

2.4.7 Criteria for assessment of actual condition

To assess the pavement actual condition, homogeneous road sections shall be determined first. The length of such section shall amount to at least 200 m out of settlements, and 100 m in settlements.

The design traffic loading for assessment of the pavement condition shall be specified in accordance with the technical specification TSC 06.511. An informative classification of traffic loading into groups is indicated in Table 5.

2.4.7.1 Design deflection on newly constructed roads

Design values of pavement surface deflection on newly constructed roads with an asphalt surfacing (limiting - d_{mm} and threshold – d_{ms}) are indicated in Table 6.

2.4.7.2 Design deflection on existing roads

Design values of pavement surface deflection on existing roads with an asphalt surfacing (d_{mm}) are indicated in Table 7. Threshold values d_{sm} can be, as a rule, by up to 0.1 mm greater.

Table 5: Informative classification of traffic loading into groups

Traffic loading group	Number of passages of nominal axle loading of 82 kN per day	in 20 years
- exceptionally heavy	above 3,000	above 2 x 10^7
- very heavy	above 800 to 3,000	above 6 x 10^6 to 2 x 10^7
- heavy	above 300 to 800	above 2 x 10^6 to 6 x 10^6
- medium	above 80 to 300	above 6 x 10^5 to 2 x 10^6
- light	above 30 to 80	above 2 x 10^5 to 6 x 10^5
- very light	up to 30	up to 2 x 10^5

Table 6: Design values of pavement surface deflection on newly constructed roads with asphalt surfacing

Traffic loading group	Design life time 10 years		20 years	
	Deflection value (mm)			
	limiting – d_{mm}	threshold – d_{ms}	limiting – d_{mm}	threshold – d_{ms}
- exceptionally heavy	0.45	0.55	0.40	0.45
- very heavy	0.60	0.70	0.50	0.60
- heavy	0.75	0.85	0.65	0.75
- medium	0.90	1.00	0.80	0.90
- light	1.05	1.15	0.95	1.05
- very light	1.20	1.30	1.10	1.20

Table 7: Design values of pavement surface deflection on existing roads with asphalt surfacing

Traffic loading group	Design life time 5 years	10 year	15 years	20 years
	Deflection limiting value d_{mm} (mm)			
- exceptionally heavy	0.8	0.7	0.6	0.5
- very heavy	0.9	0.8	0.7	0.6
- heavy	1.2	1.0	0.9	0.8
- medium	1.5	1.2	1.1	1.0
- light	1.7	1.4	1.2	1.1
- very light	1.8	1.6	1.4	1.2

2.5 NEW ASPHALT PAVEMENT STRUCTURES

2.5.1 Subject of specification

The present specification provides pavement dimensions on all the traffic surfaces intended for the motor traffic, and constructed on a substructure. The dimensions of asphalt surfacing on bridges and in tunnels shall be assessed taking account of specific conditions.

The present technical specification is intended to determine the following:

- total thickness of pavements, and
- thicknesses of layers of individual materials,

in dependence on the following factors:

- effect of traffic loading on fatigue of pavement materials,
- substrate (substructure) bearing capacity, and
- hydrological and climatic conditions.

The design of new asphalt pavements is based on the assumption that all the factors (effects) are similar on a road section under consideration, and that they will not change significantly compared with the foreseen ones. In such a case, the design life time and the serviceability of an asphalt pavement are ensured; however, the serviceability gradually decreases with the time.

The contents of this technical specification cannot be so interpreted and implemented as to prevent or condition a suitable application of construction products approved for the use in compliance with the provisions of the Law of construction products.

2.5.2 Reference documents

The present technical specification is based on the following reference documents:

AASHTO Interim Guide for Design of Pavement Structures, AASHTO, Washington, D.C., 1974

Richtlinien für die Standardisierung des Oberbaues von Verkehrsflächen – RStO 86, FGSV, Köln, 1989 (*Guidelines for Standardization of Pavements of Traffic Surfaces)*

Dimensionierung des Strassen-oberbaues (Vorträge 1972), VSS, Zürich, 1972 (*Road Pavement Design)*

Road Note 29: 1970 A guide to the structural design of pavements for new roads, Road Research Laboratory, London

RVS 3.63: 1997 Strassenplanung, Bautechnische Details, Oberbaube-messung (*Road Design; Constructive Technical Details; Pavement Design*)

SN 640 324: 1988 Dimensionierung, Strassenoberbau (*Design, Road Pavement*)

The specification includes dated provisions of other publications. Subsequent supplements or modifications shall be considered, if they are included by a supplement or revision.

2.5.3 Explanation of terms

The technical terms used in this specification shall be understood as indicated below:

Asphalt surfacing, (Asphaltdecke, Asfaltni (habajući) zastor) is pavement upper layer consisting of wearing course and bound base-bearing or base-wearing course made of asphalt mixture.

Asphalt pavement, (Asphalt – Fahrbahnbefestigung, asfaltna kolovozna konstrukcija) is a part of traffic surface stabilization with asphalt surfacing; the type of other pavement bearing courses is not defined.

Asphalt concrete, (Asphalt-beton, bitumenski beton) consists of bituminous binder and aggregate of certain grain size intended for execution of wearing and sealing courses.

Bituminous well graded crushed stone, (bituminiertes Brechkorn – Mischgut, bitumenizirani drobljeni agregat) is an asphalt mixture used for bearing courses; it consists particularly of totally rushed stone grains entirely coated with bituminous binder.

Pavement thickness-index, (Dickenindex der Fahrbahnbefestigung, debljinski indeks kolovozne konstrukcije (D)) is a sum of products of equivalency factors (= fatigue resistances) of individual materials (a_i), built-in into a pavement, and thicknesses of layers of those materials (d_i).

Pavement life time, (Lebensdauer der Fahrbahnbefestigung, vrijeme trajanja kolovozne konstrukcije) is the design time of an adequate serviceability of a pavement surface in view of traffic safety, comfort, and economy.

Hydrological conditions, (hydrologische Verhältnisse, hidrološki uslovi) are conditions defining water conditions in the ground next to road.

Climatic conditions, (klimatische Verhältnisse, klimatski uslovi) are conditions, defined by the air temperature in certain time period at certain location or in certain area, where a road is situated.

Design traffic loading, (massgebende Verkehrsbelas-tung, mjerodavno saobraćajno opterećenje) is a characteristic value of traffic loading acting on the pavement structure of one traffic lane in the design life; it is determined on the basis of the average annual daily traffic (number of vehicles) and of its growth, as well as of the additional factors: traffic lane number and width, maximum longitudinal fall of the carriageway, and eventual dynamical effects; it is a sum of passages of nominal axle load of 82 kN.

Nominal axle load, (nominelle Achslast, nazivno (nominalno) osovinsko opterećenje (NOO)) is a standard/nominal single axle load of 81.6 (82) kN, transferred by double wheels of 4 x 20.4 kN to the pavement surface; it is defined as a base to compare effects of different axle loads.

Bearing capacity, (Tragfähigkeit, nosivost) is a mechanical resistance of the formation of material built-in to transient loading.

capping layer, (verfestigter Unterbau, posteljica) is tje upper (finishing) layer of a fill or subgrade, of thickness up to 50 cm, having special properties (increased bearing capacity, reduced sensitivity to frost effects) attained by adequate constructional-technical measures (improve-ment, consolidation, stabilization).

Average annual daily traffic (ADT), (durchschnittlicher täglicher Verkehr (DTV), prosječni godišnj dnevni saobraćaj (PLDP)) is the average daily number of motor vehicles having passed the selected road cross section in a specified year, assessed on the basis of traffic counting.

2.5.4 Bases for design

2.5.4.1 General

The present guideline is based on the results of an AASHO test (American Association of State Highway Officials) supplemented by verification of relevant stresses and deformations on boundary surfaces of individual pavement layers.

The fundamental parameters in this empirical method of assessing pavement dimensions are as follows:

- pavement life time,
- serviceability of pavement surface upon the expiry of the life time (p),
- bearing capacity of substrate = substructure (CBR),
- design daily traffic loading (T_d),
- climatic and hydrological conditions (R)

- characteristics of materials to be used for the pavement foreseen (a_i, d_i).

The pavement serviceability as a target value is determined by the traffic capability index p amounting to the following:

- for new, ideally even asphalt carriageways p = 5.0
- for completely worn out (destroyed) carriageways, on which the traffic is no more feasible p = 0.

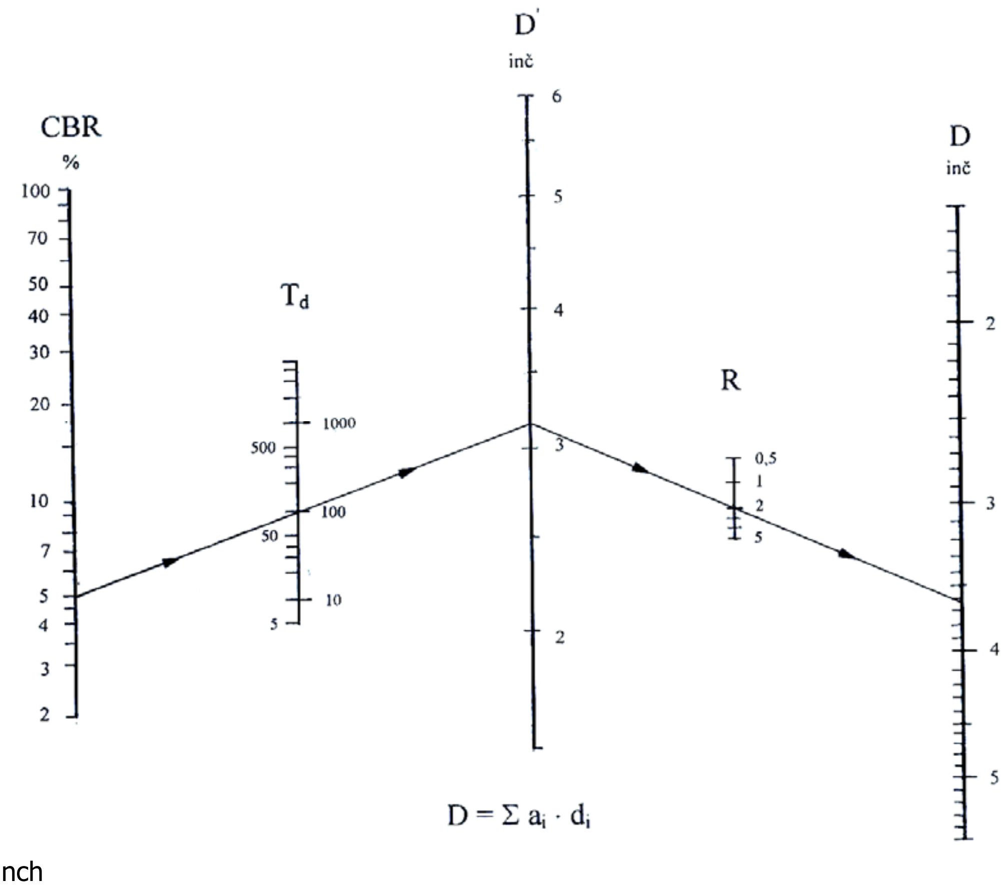
Measurements of pavement surface service-ability, or assessment of the traffic capability index on the basis of measurement results are not applicable in practice. As design limiting value of the traffic capability index upon expiry of the pavement life time, $p_k = 2.0$ has been adopted signifying still serviceable, but an ultimate limit state of the pavement surface.

The interdependence of other basic parameters According To AASHO is indicated in the nomogram in Fig. 1.

2.5.4.2 Substrate (substructure) bearing capacity

2.5.4.2.1 Assessment method

For assessment of pavement dimensions the California Bearing Ratio (CBR) is relevant. Informative correlative values of the CBR_2 ratio, of moduli of deformation E_{v2}, and of moduli of compressibility M_E are indicated in Table 1.



inč = inch

Fig. 1: Nomogram to assess the pavement thickness index D (p = 2.0)

Table 1: Informative correlations of values of bearing capacity for characteristic materials in substrate/subgrade

Material classification by USCS	CBR_2 value (%)	Modulus of compressibility M_E (MN/m^2)	Modulus of deformation E_{v2} (MN/m^2)
ML, MH, CH	3	4	15
CL, SC	5	8	20
GC, SM	7	13	45
GC, SP	10	20	60
SW, GM	15	35	80
GP, GW	20	50	100

2.5.4.2.2 Criteria

A fundamental condition for the substrate below the pavement structure is the material soil mechanic characteristics, which shall be uniform as possible, thus enabling a suitably uniform bearing capacity.

Where an adequate bearing capacity cannot be attained with natural materials, suitable methods for improvement, consolidation, and/or stabilization shall be introduced. As these procedures are relatively inexpensive, a maximum possible bearing capacity shall be attained, however not less than CBR = 7 %.

Road sections of a uniform bearing capacity shall be as long as possible. As a rule, the bearing capacity of the substrate below the pavement should be uniform on the entire section of a new road under consideration, however on a section not shorter than 500 m.

2.5.4.3 Design traffic loading

2.5.4.3.1 Assessment method

The design traffic loading T_n of the pavement in the design life time of *n* years shall be assessed according to the procedure indicated in detail in the guideline 2.1.

The total number of passages of nominal axle load of 82 kN shall be determined for each individual traffic lane.

The design life time of pavements with asphalt surfacing shall amount to 20 years as a rule. In certain cases it can also be shorter, however not less than 5 years.

2.5.4.3.2 Classification

The classification of average daily and design (total) traffic loading into characteristic groups in the pavement design life time (n = 20 years) is shown in Table 2.

Table 2: Classification of traffic loading into groups

Traffic loading group	Number of passages of nominal axle load of 82 kN	
	per day	in 20 years
- exceptionally heavy	above 3,000	above 2×10^7
- very heavy	above 800 up to 3,000	above 6×10^6 up to 2×10^7
- heavy	above 300 up to 800	above 2×10^6 up to 6×10^6
- medium	above 80 up to 300	above 6×10^5 up too 2×10^6
- light	above 30 up to 80	above 2×10^5 up to 6×10^5
- very light	up to 30	up to 2×10^5

2.5.4.4 4.4 Climatic and hydrological conditions

2.5.4.4.1 Assessment methods

The method of assessing dimensions of new asphalt pavements takes account of both climatic and hydrological conditions by the following:

- the adopted value of regional factor R = 2.0 in assessing dimensions to ensure suitable fatigue resistance of the planned materials, and
- a certain limiting thickness h_{min} of the pavement for protection from freezing and thawing effects.

2.5.4.4.2 Criteria

The regional factor values amount to R = 0.5 for the most severe climatic and hydrological conditions up to R = 5 for the most favourable ones. For the conditions prevailing in our environment R = 2.0 has been assumed as a basic value.

Relevant effects of climatic and hydrological conditions on assessing limiting thicknesses of pavement structures for protection from freezing and thawing effects shall be determined on the basis of an analysis of protection conditions and guidelines, which are indicated in detail in the giudeline 2.2.

2.5.4.5 Basic materials

2.5.4.5.1 General

When selecting materials for pavement construction, the following shall be considered:

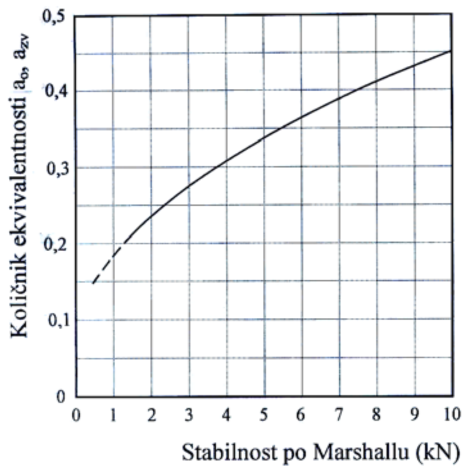
- role of individual material type and layer,
- material quality, and
- application economy.

The quality of materials intended for new asphalt pavements shall meet the requirements specified by the current technical regulations. Interrelations of these materials with regard to resistance to fatigue caused by both traffic and climatic loading, i.e. material equivalency factors or substitution factors (a_i) enable required comparisons on specifying both type and dimensions of individual pavement layers.

2.5.4.5.2 Quality definition

Average (informative) values of material equivalency factors, predominantly used for new asphalt pavements, are indicated in Table 3. In case of substantial quality deviations of asphalt mixtures for wearing and bearing courses, of cement stabilized mineral aggregates, and unbound mineral aggregates from average values, adequate material equivalency factors shall be determined using diagrams shown in Figs. 2 to 5.

Fig. 2: Equivalency factors for bituminous concrete, and bituminous well graded crushed stone



y = equivalency factor

x = stability by Marshall

Fig. 3: Equivalency factors for bituminous gravel, and mineral aggregate stabilized with bitumen

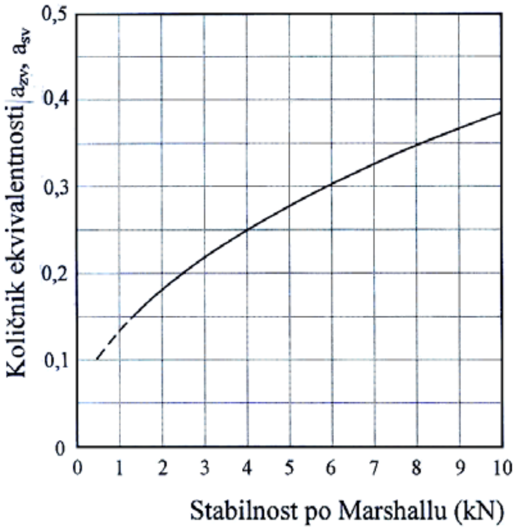


Fig 4: Equivalency factors for mineral aggregate stabilized with cement

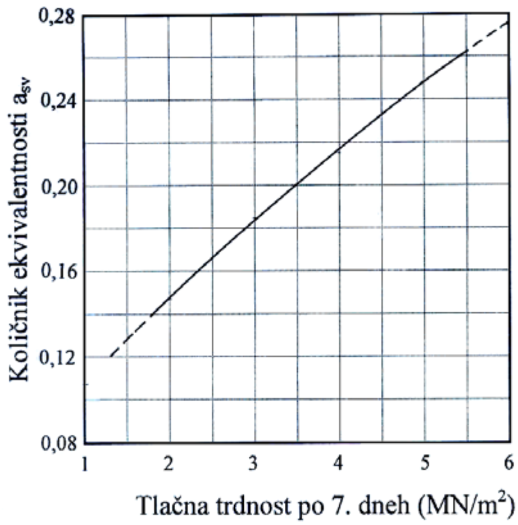
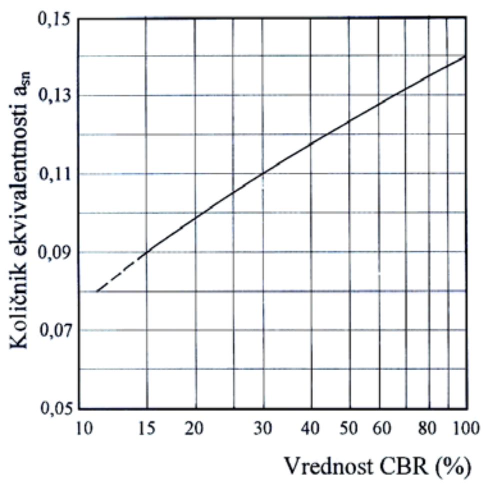


Fig. 5: Equivalency factors for unbound mineral aggregate (crushed stone, gravel)

x = compressive strength after 7 days



x = CBR value

Table 3: Average values of equivalency factors of basic road construction materials

Material type	Equivalency factor a_i
- for wearing course:	
- bituminous concrete	$a_o = 0.42$
- crushed stone with bituminous mastic	$a_o = 0.42$
- for upper roadbase:	
- bituminous crushed stone	$a_{zv} = 0.35$
- bituminous gravel	$a_{zv} = 0.28$
- for base course:	
- mineral aggregate stabilized with	
- bitumen	$a_{sv} = 0.24$
- cement	$a_{sv} = 0.20$
- for subbase:	
- crushed stone	$a_{sn} = 0.14$
- gravel	$a_{sn} = 0.11$ *

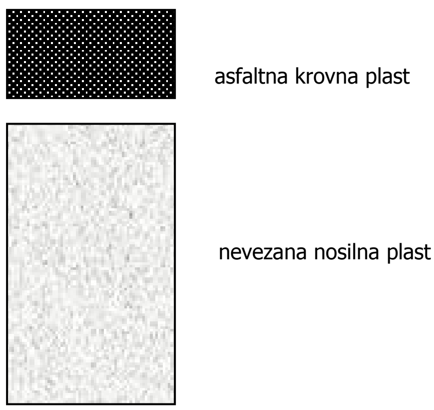
* limited by layer thickness of 40 cm

2.5.5 Basic types of asphalt pavements

Asphalt pavements can be constructed in three characteristic structures, differing one from another in bearing courses.

In view of the type of used material, bearing courses can be executed of

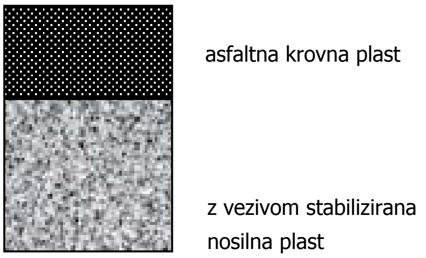
- unbound mineral aggregate (Fig. 6),
- mineral aggregate stabilized with either bitumen or cement (Fig. 7), or
- mineral aggregate stabilized with either cement or bitumen, and unbound mineral aggregate (Fig. 8).



asfaltna krovna plast = asphalt surfacing

nevezana nosilna plast = unbound base-bearing course

Fig. 6: Asphalt pavement with base-bearing course of unbound mineral aggregate



z vezivom stabilizirana nosilna plast = base-bearing course stabilized with binder

Fig. 7: Asphalt pavement with base-bearing course of mineral aggregate stabilized with either bitumen or cement

nevezana nosilna plast = unbound base-bearing course

Fig. 8: Asphalt pavement with two base- bearing courses: mineral aggregate stabilized with either bitumen or cement, and unbound mineral aggregate

In certain conditions, an asphalt pavement can be so executed, that the position of base-bearing courses is interchanged.

2.5.6 Method of assessing dimensions

2.5.6.1 General

The method of assessing new asphalt pavements comprises the following:

- assessment of relevant bases for dimensioning in compliance with procedures indicated in chapter 4, and
- assessment of both thickness and type of individual layers taking account of material properties.

To determine layer dimensions of basic new asphalt pavement consisting of an asphalt surfacing and unbound base-bearing course (Fig. 6), the diagram shown in Fig. 9 shall be adopted.

For typical new asphalt pavements, presented in Fig. 7 and Fig. 8, the dimensions of base-bearing courses shall be assessed by taking account of appropriate equivalency factors for the materials selected (Table 3).

2.5.6.2 Assessment of layer thicknesses

The required thicknesses of both asphalt surfacing and unbound mineral aggregate layer for the design traffic loading T_n in the pavement life time, and for a certain CBR value shall be assessed on the basis of the diagram indicated in Fig. 9.

2.5.6.2.1 Asphalt surfacing

The total required thickness of the asphalt surfacing d_k, i.e. of the asphalt wearing course and the asphalt upper roadbase is, in the diagram shown in Fig. 9, assessed for an average asphalt mix quality of a design equivalency factor of $a_{rk} = 0.38$.

The selection of asphalt mixes for both wearing course and upper road base depends on specific conditions of application, i.e. particularly on the foreseen traffic loading, climatic conditions, and road alignment course, to which both mineral aggregate composition and bituminous binder type shall be adapted.

The quality of the selected asphalt mixes shall meet the requirements provided by the current technical regulations for both produced and placed asphalt mixtures.

To assess the thickness of both wearing course d_o and upper roadbase d_{zv}, equivalency factors a_o and a_{zv} indicated in Table 3, as well as limiting values depending on the technology shall be considered by the following equation:

$$D_k = a_{rk} \cdot d_k = 0.38 \cdot d_k = a_o \cdot d_o + a_{zv} \cdot d_{zv}$$

Asphalt mixtures for wearing courses of new pavements to carry very heavy and exceptionally heavy traffic loading shall generally contain a modified bituminous binder.

For upper roadbases of new asphalt pavements to carry heavy, very heavy, and exceptionally heavy traffic loading, bituminous crushed stone shall be used; in addition, a modified bituminous binder is recommended as well. In particular for a light and very light traffic loading, an asphalt mix with bituminous gravel can be foreseen.

2.5.6.2.2 Unbound base-bearing course

The thickness of the unbound aggregate mixture in a base-bearing course is, in the diagram in Fig. 9, specified for a mixture of natural gravel grains of design equivalency factor of $a_{rn} = 0.11$.

For new asphalt pavements, the design thickness of an unbound base-bearing course of gravel grain mixture shall amount to as indicated below:

- heavy traffic loading min. 25 cm
- medium or light traffic loading min. 20 cm

Where, due to a poor bearing capacity of the substructure and to a heavy traffic loading, a layer of an unbound gravel grain mixture thicker than 40 cm is required (refer to the diagram shown in Fig. 9), the substructure bearing capacity shall be adequately increased as a rule.

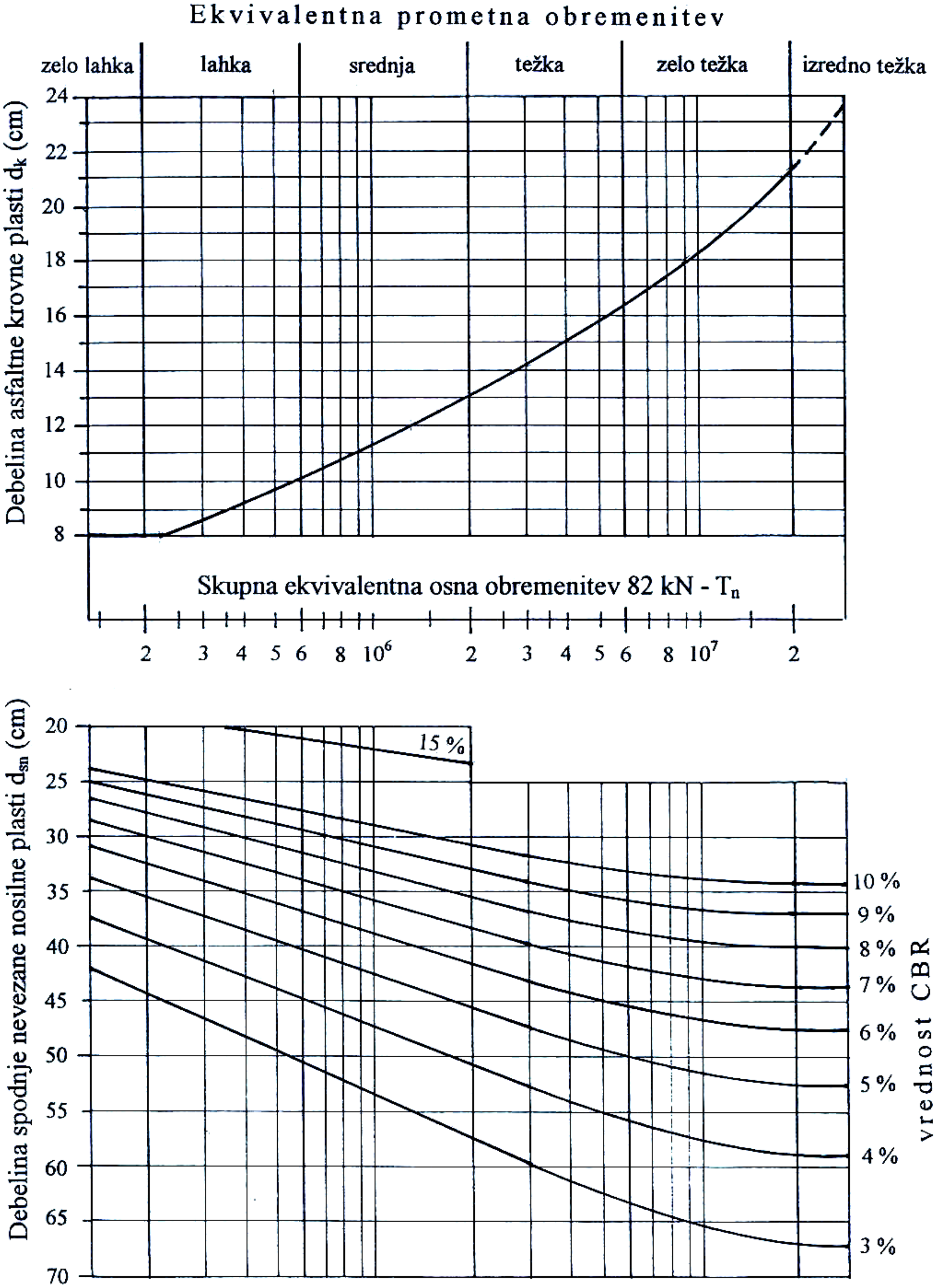
A portion of the thickness, or the complete thickness of an unbound gravel base-bearing course can be replaced by crushed stone mix, taking account of the design equivalency factor $a_{sn} = 0.14$. As the thickness of the unbound crushed stone base-bearing course is also limited to 40 cm, it can replace the equivalent design thickness of the gravel layer amounting to:

$$40 \times 0.14/0.11 \cong 50 \text{ cm.}$$

The mineral aggregate type foreseen for unbound base-bearing courses shall be adapted to both traffic loading and economical conditions. For new asphalt pavement carrying heavy, very heavy, and exceptionally heavy traffic loading, crushed stone aggregates shall be used for the unbound base-bearing course as a rule.

The quality of mineral aggregates for unbound base-bearing courses shall meet the requirements of the current technical regulations for both produced and placed mineral aggregate.

Fig. 9: Diagram for assessment of dimensions of basic layers of new asphalt pavements



debelina asfaltne krovne plasti = asphalt surfacing layer
skupna ekvivalentna osna obremenitev = total equivalent axle
ekvivalentna prometna obremenitev = equivalent traffic loading
debelina spodnje nevezane nosilne plasti = thickness of unbound base-bearing course
vrednost CBR = CBR value
zelo lahka = very light, lahka = light, srednja = medium, težka = heavy, zelo težka = very heavy, izredno težka = exceptionally heavy

2.5.6.2.3 Base course

An unbound gravel base-bearing course can be partly or completely replaced by a base course, i.e. a mixture of crushed stone or gravel, stabilized with either cement or concrete. The corresponding equivalency factor is indicated in Table 3.

The minimum design thickness of a base course for new asphalt pavements amounts to the following:

- when they are loaded with heavy traffic, and stabilized with
  - cement min. 18 cm,
  - bituminous binder min. 14 cm,
- when they are loaded with medium or light traffic, and stabilized with
  - cement min. 15 cm,
  - bituminous binder min. 12 cm.

The material type for a base course, i.e. a mineral aggregate stabilized with either bituminous binder or cement, shall be adapted to traffic and climatic conditions, road alignment course, and economical circumstances.

The quality of the asphalt mix, or the mineral aggregate stabilized with cement in the base course shall comply with the provisions of the current technical regulations for both produced and placed mix.

2.5.6.3 Stage construction

When a stage construction of a new asphalt pavement is foreseen, it shall be considered that the base-bearing course (made either of an unbound mineral aggregate, or of mineral aggregate stabilized with a binder, or of a combination of both) must be executed at a time for the entire design life time, whereas the asphalt surfacing only for the first partial life time.

The required asphalt layer thickness of the overlay, which shall be applied onto the first partial life time layer of the planned pavement, shall be, for the remaining life time, assessed from the difference between the required asphalt layer thicknesses for the entire life time, and the asphalt thickness for the first partial life time.

Prior to execution of the final design asphalt pavement, i.e. of the second stage, the bearing capacity of the existing pavement (of the first partial life time) shall be checked by deflection measurements (e.g. by means of a Benkelman beam, deflectometer, or deflectograph), and the required thickness of the additional asphalt surfacing shall be specified.

2.5.7 Verification of freezing effect

For a new asphalt pavement assessed on the basis of both traffic loading and substructure bearing capacity (chapter 2.5.6), the freezing and thawing effect shall be verified as well.

With regard to the substrate resistance, i.e. the resistance of the material below the road pavement, as well as to the hydrological conditions, the minimum required pavement thicknesses h_{min} are specified (Table 4).

In case that the total thickness of a new asphalt pavement, i.e. d_k (surfacing) + d_{sn} (base-bearing course) is less than the specified minimum pavement thickness h_{min}, the following shall be carried out:

- either to increase adequately the base-bearing course thickness, or
- to ensure suitable quality of substructure material in a required thickness.

Table 4: The minimum required pavement thicknesses h_{min}

Resistance of materials below the pavement to freezing and thawing effects	Hydrological conditions	Pavement thickness h_{min}
resistant	favourable unfavourable	$\geq 0.6\ h_m$ [1)] $\geq 0.7\ h_m$
non-resistant	favourable unfavourable	$\geq 0.7\ h_m$ $\geq 0.8\ h_m$

Legend:

[1)] *h_m – frost depth*

2.5.8 Example

2.5.8.1 Basis

2.5.8.1.1 Traffic load

Average annual daily traffic (AADT) assessed on the basis of traffic counting:

- motor car + caravan	3974 vehicles
- bus	27 vehicles
- lorry:	
- light	245 vehicles
- medium	80 vehicles
- heavy	67 vehicles
- heavy with trailer	72 vehicles

2.5.8.1.2 Substrate (substructure) bearing capacity

Considering substrate earth charasteristics determined through geomechanical investigations minimal bearing capacitty is determinated as: CBR = 8 %.

2.5.8.1.3 Design characteristics of road

- the width of traffic lines	3,25 m
- max longitudinal vertical aligment	5,5 %
- design lifetime of carriageway	20 years
- average annual daily traffic growth	4 %
- frost penetration depth h_m	85 cm
- hydrological conditions	unfavourable

2.5.8.1.4 Assessment of design traffic loading

Equivalent daily traffic load determined on the base of data in guideline 2.1. table 3.

representative vehicle	no. of vehicles	equivalency factor	sum of passages of nominal axle load NOO
- motor car and caravan	3974	0,00006	0,2
- bus	27	1,20	32,4
- lorry:			
- light	245	0,01	2,5
- medium	80	0,20	16,0
- heavy	67	1,10	73,7
- heavy with trailer	72	2,00	144,0
Ukupno:	4465		268,8

Traffic load influence factors:

- Factor of distribution of traffic loading on traffic lanes (see 2.1, table 5)	f_{pp} = 0,5
- Factor of the effect of traffic lane width on traffic loading (see 2.1, table 6)	f_{st} = 1,40
- Factor of the effect of carriageway vertical alignment on traffic loading (see 2.1, table 7)	f_{nn} = 1,09
- Factor of traffic loading increase (see 2.1 - table 8)	f_{po} = 31

Planned pavement traffic load during design lifetime of 20 years:

$T_n = 365 \times T_d \times f_{pp} \times f_{št} \times f_{nn} \times f_{po} = 365 \times 268{,}8 \times 0{,}5 \times 1{,}40 \times 1{,}09 \times 31 = 2{,}3 \times 10^6$ passing of nominal axle load (NOO) 82 kN

what means heavy traffic loading (see 2.1, table 9).

2.5.8.2 Pavement design

Equivalence factors of basic road construction materials are given in guideline 2.5, table 3. Average value of equivalency factor of asphalt surfacing mixtures is :

$a_{pz} = 0{,}38$.

2.5.8.2.1 Assessment of dimensions of basic courses (see 2.5 figure 9)

For planned traffic load: $T_n = 2{,}3 \times 10^6$ NOO and minimal bearing capacitty CBR = 8 % the following pavement courses arenecessary:

- asphalt surfacing in depth d=13,5 cm
- subbase composed of unbound sand d=36,5 cm

Assessment of the pavement thickness-index:

$D = 13{,}5 \times a_{pz} + 36{,}5 \times a_{š} = 13{,}5 \times 0{,}38 + 36{,}5 \times 0{,}11 = 9{,}15$ cm

2.5.8.2.2 Designing of pavement courses

For planned traffic load T_n pavement could be composed of the following courses:

- asphalt concrete BB 8s or BB 11s (see Special technical conditions, chapter 2.2.2.12.2 table 3.25 or bituminous mastic chippings SBM 8s or SBM 11s (see Special technical conditions, chapter 2.2.2.12.6 table 3.61) d= 3,5 cm,
- bituminous well graded crushed stone VGNS 22S or VGN 32S (see Special technical conditions, chapter 2.2.2.11.4 table 3.12) d= 10 cm and
- subbase composed of crushed stone (see Special technical conditions, chapter 2.2.2.11 d = h_d :

$$h_d = \frac{D - a_o \cdot h_o - a_{zv} \cdot h_{zv}}{a_d} = \frac{9{,}15 - 0{,}42 \cdot 3{,}5 - 0{,}35 \cdot 10}{0{,}14} = 30cm$$

Unfavorable hydrological conditions require the depths of courses composed of frost resistant materials: (see 2.2, table 1)

$h_{min} = h_m \times 0{,}7 = 85 \times 0{,}7 = 60$ cm

On condition that the substrate material to the depth of:

$h = h_{min} - h_o - h_{zv} - h_d = 60 - 3{,}5 - 10 - 30 = 17$ cm

is not containing more than 8 m.-% grains d< 0,063 mm, pavement may be composed of:

- 3,5 cm bituminous concrete or bituminous mastic chippings
- 10 cm bituminous crushed stone and
- 30 cm subbase composed of crushed stone.

In the case of substrate sensitivity to frost effects, the depths of subbase shall be increased for 17 cm to total depth of d= 47 cm.

2.6 NEW CONCRETE PAVEMENT STRUCTURE

2.6.1 Subject of specification

The present specification provides pavement dimensions on all the traffic surfaces intended for the motor traffic, and constructed on a substructure.

The dimensions of cement concrete surfacing on bridges and in tunnels shall be assessed taking account of specific conditions.

The present technical specification is intended to determine the following:

- total thickness of pavements, and
- thicknesses of layers of individual materials,

in dependence on the following factors:

- effect of traffic loading on fatigue of pavement materials,
- substrate (substructure) bearing capacity, and
- hydrological and climatic conditions.

The design of new cement concrete pavements bases on the assumption that all the factors (effects) are similar on a road section under consideration, and that they will not change significantly compared with the foreseen ones. In such a case, the design life time and the serviceability of a cement concrete pavement are ensured; however, the serviceability gradually decreases with the time.

The contents of this technical specification cannot be so interpreted and implemented as to prevent or condition a suitable application of construction products approved for the use in compliance with the provisions of the Law of construction products.

2.6.2 Reference documents

The present specification is based on the following reference documents:

AASHTO Interim Guide for Design of Pavement Structures, AASHTO, Washington, D.C., 1974

Dimensionierung des Strassen-oberbaues (Vorträge 1972), VSS, Zürich, 1972 (*Road Pavement Design)*

Richtlinien für die Standardisierung des Oberbaues von Verkehrsflächen – RStO 86, FGSV, Köln, 1989 (*Guidelines for Standardization of Pavements of Traffic Surfaces)*

Road Note 29: 1970 A guide to the structural design of pavements for new roads, Road Research Laboratory, London

RVS 3.63: 1997 Strassenplanung, Bautechnische Details, Oberbaube-messung (*Road Design; Constructive Technical Details; Pavement Design*)

SN 640 324: 1988 Dimensionierung, Strassenoberbau (*Design, Road Pave-ment*)

SNV 640 326: 1971 Dimensionierung, Oberbau mit Zementbetonbelag (*Design, Pavement with cement concrete surfacing*)

The specification includes dated provisions of other publications. Subsequent supplements or modifications shall be considered, if they are included by a supplement or revision.

2.6.3 Explanation of terms

See (General technical conditions, chapter 2.1.3)

2.6.4 Bases for design

2.6.4.1 General

The present specification is based on the results of an AASHO test (American Association of State Highway Officials) supplemented by verification of relevant stresses and deformations on boundary surfaces of individual pavement layers.

The fundamental parameters in this empirical method of assessing pavement dimensions are as follows:

- pavement life time,
- serviceability of pavement surface upon the expiry of the life time (p),
- bearing capacity of substrate = substructure (CBR),
- design daily traffic loading (T_d),

- climatic and hydrological conditions (R)
- characteristics of materials

2.6.4.2 Subgrade (substructure) bearing capacity

2.6.4.2.1 Assessment method

For assessment of pavement dimensions the California Bearing Ratio (CBR) is relevant. Informative correlative values of the CBR_2 ratio, of moduli of deformation E_{v2}, and of moduli of compressibility M_E are indicated in Table 1.

Table 1: Informative correlations of values of bearing capacity for characteristic materials in substrate/subgrade

Material classification by USCS	CBR_2 value (%)	Modulus of compressibility M_E (MN/m^2)	Modulus of deformation E_{v2} (MN/m^2)
ML, MH, CH	3	4	15
CL, SC	5	8	20
GC, SM	7	13	45
GC, SP	10	20	60
SW, GM	15	35	80
GP, GW	20	50	100

2.6.4.2.2 Criteria

A fundamental condition for the substrate below a cement concrete pavement structure is the material soil mechanic characteristics, which shall be uniform as possible, thus enabling a suitably uniform bearing capacity.

Where an adequate bearing capacity cannot be attained with natural materials, suitable methods for improvement, consolidation, and/or stabilization shall be introduced. As these procedures are relatively inexpensive, a maximum possible bearing capacity shall be attained, however not less than CBR = 10 %.

Road sections of a uniform bearing capacity shall be as long as possible. As a rule, the bearing capacity of the substrate below the pavement should be uniform on the entire section of a new road under consideration, however on a section not shorter than 500 m.

2.6.4.3 Design traffic loading

2.6.4.3.1 Assessment method

The design traffic loading T_n of the pavement in the design life time of *n* years shall be assessed according to the procedure indicated in detail in the guideline 1.1.7.2.1.

The total number of passages of nominal axle load of 82 kN shall be determined for each individual traffic lane.

The design life time of pavements with cement concrete surfacing shall amount to 20 years as a rule. In certain cases it can also be shorter, however not less than 10 years.

2.6.4.3.2 Classification

The classification of average daily and design (total) traffic loading into characteristic groups in the pavement design life time (n = 20 years) is shown in Table 2.

Table 2: Classification of traffic loading into groups

Traffic	Number of passages

loading group	of nominal axle load of 82 kN per day	in 20 years
- exceptionally heavy	above 3,000	above 2 x 10^7
- very heavy	above 800 up to 3,000	above 6 x 10^6 up to 2 x 10^7
- heavy	above 300 up to 800	above 2 x 10^6 up to 6 x 10^6
- medium	above 80 up to 300	above 6 x 10^5 up too 2 x 10^6
- light	above 30 up to 80	above 2 x 10^5 up to 6 x 10^5
- very light	up to 30	up to 2 x 10^5

2.6.4.4 Climatic and hydrological conditions

2.6.4.4.1 Assessment methods

The method of assessing dimensions of new cement concrete pavements takes account of both climatic and hydrological conditions by the following:

- the adopted value of regional factor R = 2.0 in assessing dimensions to ensure suitable fatigue resistance of the planned materials, and
- a certain limiting thickness h_{min} of the pavement for protection from freezing and thawing effects.

2.6.4.4.2 Criteria

The regional factor values amount to R = 0.5 for the most severe climatic and hydrological conditions up to R = 5 for the most favourable ones. For the conditions prevailing in our environment R = 2.0 has been assumed as a basic value.

Relevant effects of climatic and hydrological conditions on assessing limiting thicknesses of pavement structures for protection from freezing and thawing effects shall be determined on the basis of an analysis of protection conditions and guidelines, which are indicated in detail in the TSC 06.512.

2.6.4.5 Basic materials

2.6.4.5.1 General

When selecting materials for pavement construction, the following shall be considered:

- role of individual material type and layer,
- material quality, and
- application economy.

The quality of materials intended for new cement concrete pavements, shall meet the requirements specified by the current technical regulations.

When assessing both type and dimensions of unbound bearing course, material equivalency factors, or substitution factors (a_{sn}) shall be considered. The mentioned factors enable a required comparison of interrelations of these materials with regard to resistance to fatigue caused by both traffic and climatic loading.

2.6.4.5.2 Cement concrete

For the assessment of a cement concrete surfacing, the flexural tensile strength is relevant. The characteristic strength of the 28-days cement concrete shall be assumed. The characteristic flexural tensile strength of the cement concrete is defined as a 5 %-fractil value.

If the characteristic compressive strength $f_{ck,cyl}$ is only specified, the estimated mean flexural tensile strength f_{cfm} of cement concrete may be calculated from the equation below:

$$f_{cfm} = 0{,}5\left(f_{ck,cyl}\right)^{2/3} \quad (\text{N/mm}^2) \qquad (1)$$

The characteristic flexural tensile strength f_{cfk} may be calculated from the following equation:

$$f_{cfk} = 0{,}7 f_{cfm} \quad (\text{N/mm}^2) \qquad (2)$$

The characteristic compressive strength $f_{ck,cyl}$ shall be assessed on the basis of results of compressive strength testing on cylinders of 15 cm in diameter, and 30 cm in height. The interconnection of cement concrete strength classes (according to the SIST EN 206-1), concrete grades MB (according to the PBAB – Rulebook of concrete and reinforced concrete), and cylinder compressive strengths $f_{ck,cyl}$ is indicated in Table 3.

Table 3: Interconnection of strength classes, cement concrete grades MB, and characteristic compressive strengths of cylinders $f_{ck,cyl}$

Strength class according to SIST EN 206-1	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
Concrete grade MB according to PBAB*	MB 30	MB 35	MB 40	MB 45	MB 55	MB 60	-
$f_{ck,cyl}$	20	25	30	35	40	45	50

**Rulebook of concrete and cement concrete*

If bending tests are carried out, the following equation may be adopted to calculate the characteristic flexural tensile strength f_{cfk} of cement concrete:

$$f_{cfk} = f_{cfms} - \frac{s_p t_{10}}{\sqrt{n}} - 1{,}645 s_p \left(1 + \frac{s_p t_{10}}{f_{cfms}\sqrt{n}}\right) \qquad (3)$$

where:

f_{cfk} characteristic value (N/mm^2)

f_{cfms} mean value of testing series (N/mm^2)

s_p standard deviation (N/mm^2)

t_{10} value for Student's distribution at 10 %-fractil (in dependence on the number of specimens; basic values are indicated in Table 4)

n number of specimens

- mean value f_{cfm}:

$$f_{cfm} = f_{cfms} - \frac{s_p t_{10}}{\sqrt{n}}$$

- standard deviation s_p:

$$s_p = \sqrt{\frac{\Sigma\left(f_{cfms} - f_{cfm}\right)^2}{(n-1)}} \qquad (4)$$

Values t_{10} are indicated in Table 4 in dependence on the number of specimens.

Table 4: Values t_{10} in dependence on the number *n* of specimens

- **n**	3	4	5	6	8	10	12	15

t_{10}	1,89	1,64	1,53	1,48	1,42	1,38	1,36	1,34

2.6.4.5.3 Asphalt mixture

As a direct base for the cement concrete surfacing, a layer of an asphalt mixture of bituminous crushed stone or bituminous gravel shall be placed.

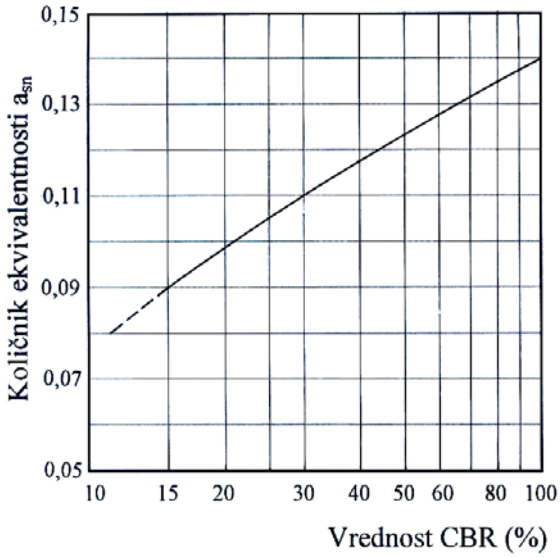
2.6.4.5.4 Mineral aggregate

The unbound base-bearing course of a cement concrete pavement shall be executed in compliance with the requirements indicated in this guidelines.

Average (informative) values of equivalency factors of materials used for the unbound base-bearing course are as follows:

- for crushed stone: a_{sn} = 0.14
- for gravel: a_{sn} = 0.11; however, this value is limited by the layer thickness of 40 cm.

In case of substantial deviations of unbound mineral aggregates from the average values, suitable equivalency factors shall be assessed using the diagram in Fig. 1.



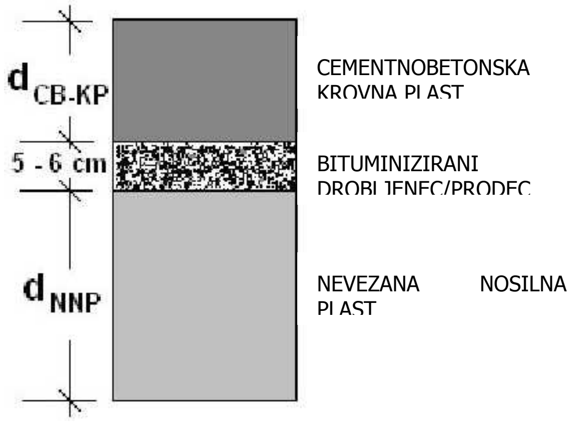
y = equivalency factor, x = CBR value

Fig. 1: Equivalency factors for unbound mineral aggregate (crushed stone, gravel)

2.6.5 Cement concrete pavement composition

Cement concrete pavements are composed of the following layers (Fig. 2):

- cement concrete surfacing
- intermediate binding course of bituminous crushed stone or gravel in a thickness of 5 to 6 cm, and
- unbound base-bearing course



cement concrete surfacing
bituminous crushed stone/gravel
unbound base-bearing course

Fig. 2: Chracteristic composition of cement concrete pavements

2.6.5.1 Method of assessing dimensions

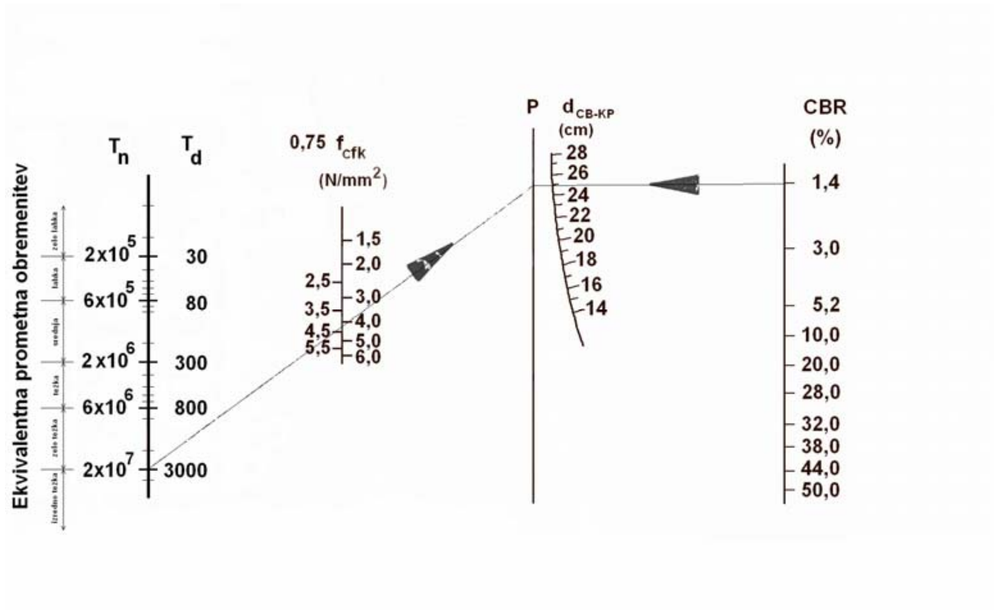
2.6.5.1.1 General

The method of assessing new cement concrete pavements comprises the following:

- assessment of relevant bases for dimensioning in compliance with procedures indicated in chapter 4, and
- assessment of both thickness and type of individual layers taking account of material properties.

2.6.5.1.2 Assessment of layer thicknesses

The required thicknesses of the cement concrete surfacing $d_{CB\text{-}KP}$, and of the unbound mineral aggregate layer for the design traffic loading T_n in the pavement life time, and for a certain CBR value shall be assessed on the basis of the nomogram in Fig. 3, and the diagram in Fig. 4.



equivalent traffic loading, very light, light, medium, heavy, very heavy, exceptionally heavy

Fig. 3: Nomogram for assessment of cement concrete surfacing thickness $d_{CB\text{-}KP}$

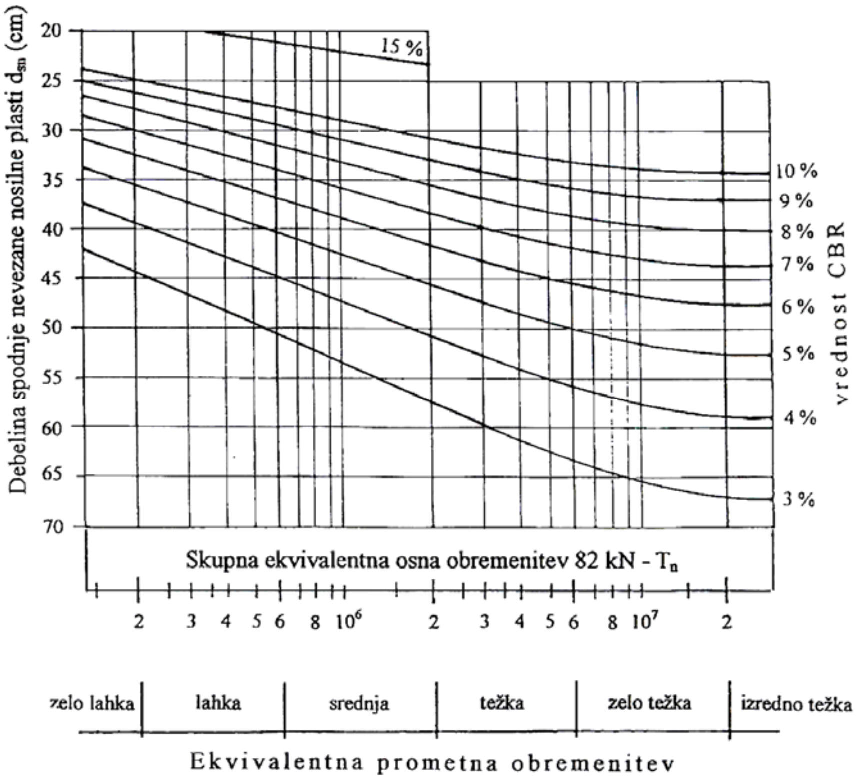


Fig. 4: Diagram for assessment of thickness of mineral aggregate layer of an unbound base-bearing course

y-levo = thickness of unbound base-bearing course, y-desno = CBR value, X = total equivalent axle load of 82 kN;

ekvivalentna prometna obremenitev = equivalent traffic loading; very light, light, medium, heavy, very heavy, exceptionally heavy.

2.6.5.1.3 Cement concrete surfacing

The required cement concrete surfacing thickness $d_{CB\text{-}KP}$ is, in the nomogram indicated in Fig. 2, determined for an average quality of a cement concrete mix.

The selection of cement concrete mixes for both wearing and upper unbound bearing course, which can differ one from the other, but are placed according to a "fresh to fresh" procedure, depends on specific service conditions, in particular on the foreseen traffic loading, climatic conditions, and road alignment course, to which the composition of a fresh cement concrete mix shall be adapted. The cement concrete mix quality shall meet the requirements provided by the current technical regulations for both produced and placed cement concrete mix.

Cement concrete mixes for wearing courses of new pavements shall be air-entrained, or their composition shall be so modified as to ensure, in the hardened condition, their prescribed rate of resistance to freezing and thawing in presence of de-icing salt; in addition, they shall also be adequately resistant to polishing of the carriageway surface.

2.6.5.1.4 Unbound base-bearing course

The thickness of the unbound aggregate mixture d_{NNP} in a base-bearing course is, in the diagram in Fig. 4, specified for a mixture of natural gravel grains of design equivalency factor of a_{rn} = 0.11.

For new cement concrete pavements, the design thickness of an unbound base-bearing course of gravel grain mixture shall amount to as indicated below:

- heavy traffic loading min. 25 cm
- medium or light traffic loading min. 20 cm

Where, due to a poor bearing capacity of the substructure and to a heavy traffic loading, a layer of an unbound gravel grain mixture thicker than 40 cm is required (refer to the diagram shown in Fig. 4), the substructure bearing capacity shall be adequately increased as a rule.

A portion of the thickness, or the complete thickness of an unbound gravel base-bearing course can be replaced by crushed stone mix, taking account of the design equivalency factor a_{sn} = 0.14. As the thickness of the unbound crushed stone base-bearing course is also limited to 40 cm, it can replace the equivalent design thickness of the gravel layer amounting to:

$40 \times 0.14/0.11 \cong 50$ cm.

The mineral aggregate type foreseen for unbound base-bearing courses shall be adapted to both traffic loading and economical conditions. For new cement concrete pavement carrying heavy, very heavy, and exceptionally heavy traffic loading, crushed stone aggregates shall be used for the unbound base-bearing course as a rule.

2.6.6 Verification of freezing effect

For a new cement concrete pavement assessed on the basis of both traffic loading and substructure bearing capacity (chapter 6), the freezing and thawing effect shall be verified as well.

With regard to the substrate resistance, i.e. the resistance of the material below the road pavement, as well as to the hydrological conditions, the minimum required pavement thicknesses h_{min} are specified (Table 5).

In case that the total thickness of a new cement concrete pavement, i.e. d_{CB-KP} (cement concrete surfacing) + bituminous crushed stone/gravel layer (5 – 6 cm) + d_{NNP} (unbound base-bearing course) is less than the specified minimum pavement thickness h_{min}, the following shall be carried out:

- either to increase adequately the base-bearing course thickness, or
- to ensure suitable quality of substructure material in a required thickness.

Table 5: The minimum required pavement thicknesses h_{min}

Resistance of materials below the pavement to freezing and thawing effects	Hydrological conditions	Pavement thickness h_{min}
resistant	favourable unfavourable	$\geq 0.6\ h_m$ [1)] $\geq 0.7\ h_m$
non-resistant	favourable unfavourable	$\geq 0.7\ h_m$ $\geq 0.8\ h_m$

Legend:

[1)] *h_m – frost depth (frost penetration depth)*

2.7 PAVEMENT STRUCTURE STRENGTHENING

2.7.1 Subject of specification

The presentl specification provides dimensions of planned strengthening of existing pavements on all the traffic surfaces intended for the motor vehicle traffic.

Dimensions of strengthening of existing pavements on bridges and in tunnels shall be assessed taking account of specific conditions.

The presentl specification is intended for assessment of the

- total thickness of strengthening, and
- thicknesses of individual layers,

for the planned strengthening of an existing pavement. A pavement determined in this way is, in dependence on the

- foreseen traffic loading during life time,
- existing pavement,
- quality of materials used, and
- hydrological and climatic conditions,

required to prevent an excessive fatigue (destruction) of the material structure in the existing pavement, as well as to maintain the pavement surface serviceability at an adequate level to ensure a safe, comfortable, and economical travel at optimum fund consumption.

The design of strengthening of existing pavements is based on the assumption that all the abovementioned factors are similar on the considered road section, and that they will not significantly change with regard to the foreseen ones. In such a case the life and service time of a strengthened pavement is ensured. However, its serviceability gradually decreases with the time.

The contents of this technical specification cannot be so interpreted and implemented as to prevent or condition a suitable application of construction products approved for the use in compliance with the provisions of the Law of construction products.

2.7.2 Reference documents

The present technical specification is based on the following reference documents:

AASHTO Interim Guide for Design of Pavement Structures, AASHTO, Washington, D.C., 1974

Asphalt Overlays and Pavement Rehabilitation, MS-17, The Asphalt Institute, College Park, Maryland, 1977

Richtlinien für die Standardisierung des Oberbaus von Verkehrsflächen – RStO 86, FGSV, Köln, 1989 (*Guidelines for Standardization of Pavements of Traffic Surfaces)*

Evalution of AASHO Interim Guides for Design of Pavement Structures, NCHRP Report 128, HRB, National Academy of Science, Washington D.C., 1972

Fahrbahnverstärkungen OECD, Bundes-amt für Strassenbau, Bern, 1982 (*Pavement Strengthening)*

Haas R., Hudson W.R., **Pavement Management Systems**, Mc Graw-Hill Book Company, New York, 1978

Rational Pavement Management, Studie Centrum Wegebouw, SCW Record 1, Arnhem, 1975

Road Note 29, A guide to the structural design of pavements for new roads, Road Research Laboratory, London, 1970

RVS 3.54: 1992 Strassenplanung, Bautechnische Details, Oberbau-verstärkung von Asphaltstrassen (*Road Design, Constructive Details, Asphalt Road Pavement*

Strengthening)

SNV 640 738: 1977 Reparatur und Erneuerung von Fahrbahnen, Oberbau-verstärkung in bituminöser Bauweise (*Repair and Restoration of Carriageways, Pavement Strengthening with Bitumen)*

TGL 22 853: 1969 Anlagen des Strassen-verkehrs, Bemessung flexibler Befestigungen, Kriterium der zulässigen Durchbiegung (*Road Traffic Equipment, Design of Flexbile Pavements, Criterion of Admissible Deflection)*

ThE specification includes provisions of other publications, either by dated or undated references. For dated references, subsequent supplements or modifications shall be considered, if they are included by a supplement or revision. For undated references the latest edition of the reference publication is valid.

2.7.3 Explanation of terms

The terms in this technical specification shall be understood as indicated in general technical conditions.

2.7.4 Strengthening in general

An existing asphalt pavement can be strengthened by means of

- an overlay,
- a partial replacement, or
- a complete replacement.

When the overlay procedure is selected, one or more new asphalt mix layers are placed onto the existing pavement.

The method of partial replacement includes

- a replacement of a portion of damaged pavement (e.g. of substantially cracked, crushed, or deformed asphalt layers) with new layers of suitable materials, or
- a treatment of a portion of existing pavement using adequate procedure for restoration of specified material properties (e.g. stabili-zation of unbound aggregate, remix, etc.).

In case of a complete replacement, the entire damaged pavement is removed, and a new pavement shall be constructed on a newly arranged substructure formation. The materials of the removed pavement can be reused, on condition that they are adequately recycled.

The decision whether to apply only one overlay, or to foresee the strengthening, depends on the following:

- suitability of existing layers for a portion of a new pavement,
- limitations specified in advance (e.g. limited carriageway height, bridge bearing capacity, etc.),
- impacts on the environment, and
- economy.

In this technical specification particularly such asphalt pavement strengthening is discussed, where asphalt overlays are applied. The basic condition is to be familiar in detail with all the particularities of the existing conditions of a pavement, and to ensure optimum drainage conditions.

As a rule, very deformable materials in the existing pavement shall be removed. They can only be used on condition that they are adequately recycled, or overlaid by layers of sufficient stability.

On road sections where the traffic of heavy lorries is slowed down or guided, or where the effect of horizontal forces on the pavement is high (due to braking and accelerating), these effects shall be considered by a proper selection of materials for strengthening.

The fundamental condition for the required analysis of causes of damages, and for the decision upon necessity and feasibility of an adequate strengthening of an existing pavement, is an investigation and evaluation of the actual condition of the entire existing pavement, as well as of the individual materials placed into it.

The following activities are especially important:

- to determine visually the magnitude and extent of damages,
- to establish deformations and uniformity of the pavement structure by measuring both longitudinal and transverse evenness,
- to ascertain, by measuring deflection of the existing pavement, the bearing capacity and its uniformity, and in certain cases also values of E-moduli of materials built-in (measurements with falling weight deflectometers with a falling weight – FWD),

and/or, in an adequate way (e.g. by a test pit)

- to establish the adhesion strength of asphalt layers placed,
- to investigate serviceability of existing materials (asphalt mixtures, unbound aggregate mixes) for the new pavement, or their suitability for an overlay,
- to carry through measurements of bearing capacity on the unbound material layer formation.

2.7.5 Bases of strengthening design methods

To assess the required dimensions of layers for strengthening of existing asphalt pavements, suitable methods shall be adopted. These methods are based on the following:

- on deflection, or exceptionally
- on the condition

of existing pavement.

For a comparison or a verification analytical methods for evaluation of the required strengthening.

The basic parameters assumed in assessing the required dimensions of strengthening are as follows:

- properties of pavement materials,
- deflection of existing pavement,
- traffic loading,
- pavement life time,
- pavement surface serviceability at expiry of the life time,
- climatic and hydrological conditions.

2.7.5.1 Material properties

When selecting material for strengthening of existing asphalt pavement, the following shall be considered:

- material quality,
- role of individual material type and layer in the pavement, and
- economy of the use.

The quality of materials intended for strengthening of existing asphalt pavements, shall meet the requirements specified in the current technical regulations.

Interrelations of resistance of these materials to fatigue, which is conditioned by both traffic and climatic loading, i.e. the equivalency factors for the material (a_i) assessed on the basis of the AASHO test results, enable the required comparisons on assessing the type and dimensions of individual layers for strengthening the existing asphalt pavements, as well as an estimation of their residual capacity to take the traffic loading (i.e. the residual values of the thickness-index).

Average (informative) values of equivalency factors of newly produced materials are indicated in Table 1.

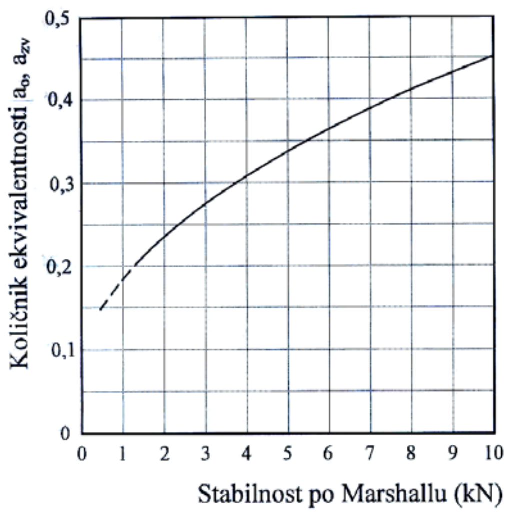
In case of substantial deviations of quality of newly produced materials from the average values, adequate equivalency factors shall be assessed using the diagrams shown in Figures 1 to 4.

Table 1: Average values of equivalency factors of basic road construction materials

Material type	Equivalency factor - a_i
- for wearing course:	
- bituminous concrete	$a_o = 0.42$
- crushed stone with bituminous mastic	$a_o = 0.42$
- for upper roadbase:	
- bitumininous crushed stone	$a_{zv} = 0.35$
- bituminous gravel	$a_{zv} = 0.28$
- for base course:	
- stabilized mineral aggregate	
- with bitumen	$a_{sv} = 0.24$
- with cement	$a_{sv} = 0.20$
- for subbase:	
- crushed stone	$a_{sn} = 0.14$
- gravel	$a_{sn} = 0.11$ [1)]

Legend:

1) limited by layer thickness of 40 cm



količnik ekvivalentnosti = equivalency factor
stabilnost po Marshallu = stability by Marshall

Fig. 1: Equivalency factors for bituminous concrete a_o and bituminous crushed stone a_{zv}

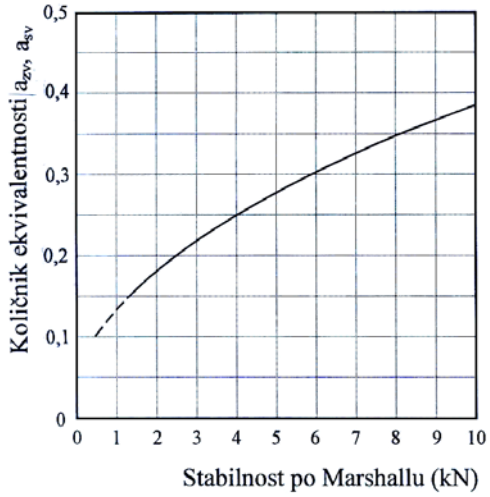
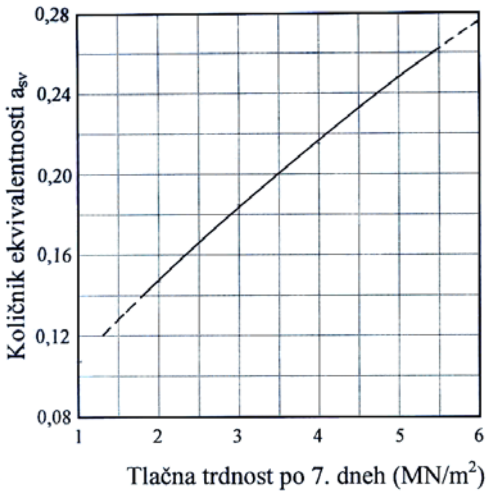
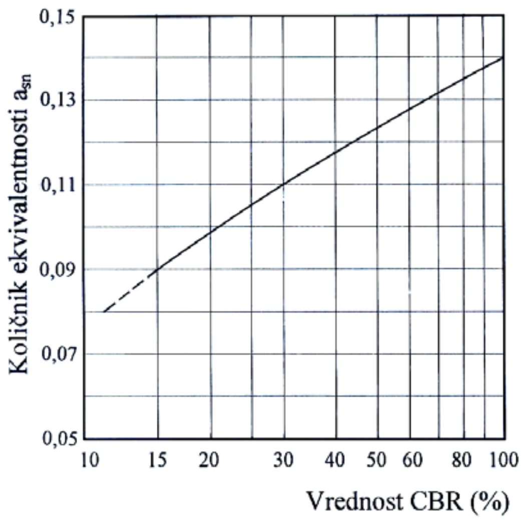


Fig. 2: Equivalency factors for bituminous gravel a_{zv} and mineral aggregate stabilized with bitumen a_{sv}



tlačna trdnost po 7 dneh = compressive strength after 7 days

Fig. 3: Equivalency factors for mineral aggregate stabilized with cement



vrednost CBR = CBR value

Fig. 4: Equivalency factors for unbound mineral aggregate (crushed stone, gravel)

Percentages of residual capacity of existing materials to take the traffic loading can be assessed, on the basis of the evaluation of the present condition of an asphalt pavement, by means of informative factors indicated in Table 2, using the equation below:

$$D_{ob} = \sum a_i \cdot d_i \cdot u_i \qquad [cm]$$

where:

D_{ob} - *thickness-index of existing pavement*

a_i - *equivalency factor of new material (Table 1)*

d_i - *layer thickness*

u_i - *factor of residual serviceability of material (Table 2)*

Table 2: Informative factors of residual serviceability of materials of existing layers

Material classification	Description of asphalt pavement condition (visual evaluation)	Factor u
IV	**Subbase:**	
	- aggregate mixture of gravel (a_{sn} = 0.11):	
	- non-resistant to heterogeneous freezing – CBR ≥ 10 %	0.5
	- resistant to heterogeneous freezing – CBR ≥ 40 %	0.9
	- aggregate mixture if crushed stone (a_{sn} = 0.14):	
	- non-resistant to heterogeneous freezing – CBR ≥ 10 %	0.6
	- resistant to heterogeneous freezing – CBR ≥ 40 %	0.9
III	**Base course:**	
	- stabilized with cement (a_{sv} = 0.20):	
	- very cracked	0.7
	- little cracked	0.9
	- stabilized with bitumen (a_{sv} = 0.24):	
	- very cracked	0.6
	- little cracked	0.9
II	**Upper roadbase:**	
	- bituminous gravel (a_{zv} = 0.28):	
	- very cracked and deformed	0.4
	- very cracked	0.5
	- very deformed	0.65
	- little cracked and/or deformed	0.8
	- undamaged	0.9
	- bituminous crushed stone (a_{zv} = 0.35):	
	- very cracked and deformed	0.4
	- very cracked	0.5
	- very deformed	0.65
	- little cracked and/or deformed	0.8
	- undamaged	0.9
I	**Wearing and sealing course** (a_o = 0.42):	
	- very cracked, it peels off and/or crumbles	0.3
	- very cracked and deformed	0.4
	- very cracked	0.5
	- very deformed	0.65
	- little cracked and/or deformed	0.8
	- undamaged	0.9

The thickness-index of an existing asphalt pavement D_{ob} can also be assessed using a modified Swiss index (MSI), which includes both magnitude and extent of characteristic damages to the asphalt carriageway (cracks, wear and tear, potholes, and patches), according to the equation below:

$$D_{ob} = D_{no} \cdot k_{š} \qquad [cm]$$

where:

D_{no} - thickness-index of new pavement (on completed construction, assessed according to TSC 06.520)

$k_{š}$ - damage factor

Damage factors $k_{š}$ are defined in dependence on the MSI values and traffic density on national roads (Table 3).

Table 3: Damage factors $k_{š}$ of existing asphalt pavement

Traffic density		MSI values		
Description	ADT limiting values			
extremely high	> 20,000	< 2.2	2.2 to 2.8	> 2.8
very high	> 10,000 to 20,000	< 2.3	2.3 to 2.9	> 2.9
high	> 5,000 to 10,000	< 2.4	2.4 to 3.0	> 3.0
medium	> 2,000 to 5,000	< 2.5	2.5 to 3.1	> 3.1
low	> 1,000 to 2,000	< 2.6	2.6 to 3.2	> 3.2
very low	< 1,000	< 2.7	2.7 to 3.3	> 3.3
	damage factor - $k_{š}$	0.7	0.7 to 0.4	0.4

2.7.5.2 Deflection

The deflection of an existing asphalt pavement shall generally be established with a Lacroix deflectograph. Other methods are also feasible, e.g. FWD or Benkelman beam, for which an applicable correlation is ascertained, and which are mentioned in the relevant technical regulations.

Deflection measurements shall be carried out in the period of the pavement minimum bearing capacity, i.e. in the spring thawing period. When these measurements are performed out of the thawing period, the results shall be corrected by a factor c which informative values amount to:

c = 1.2 - 1.6 the wearing course is non-cracked; mineral aggregate being little to medium sensitive to freezing (F2) is used for the subbase (unbound bearing course)

c = 1.6 - 2.0 the wearing course is cracked; mineral aggregate being medium sensitive to freezing is used for the subbase (unbound bearing course)

When selecting the correction factor c, climatic and hydrological conditions shall be considered as well.

Where the wearing course is substantially cracked, and the climatic and hydrological conditions are unfavourable, the correction factor value shall be assessed by means of measurements.

No deflection measurement must be carried through, when any layer of the existing pavement is frozen, or when the wearing course temperature exceeds 25 °C.

Homogeneous sections of pavement deflection shall be determined on the basis of the condition that the variation factor amounts to:

$$k_V = \frac{s}{\bar{d}} < 0{,}35$$

where:

standard deviation $s = \sqrt{\frac{\sum\left(d_i - \bar{d}\right)^2}{n-1}}$ (mm/100)

average deflection $\bar{d} = \frac{\sum d_i}{n}$ (mm/100)

The design deflection of existing pavement d_m shall be calculated from the equation below:

$$d_m = c \cdot \left(\bar{d} + k_{pr} \cdot s\right) \quad \text{(mm/100)}$$

where:

k_{pr} – traffic loading effect factor amounting to:

- *k_{pr} = 2.0 for roads with heavy traffic*
- *k_{pr} = 1.6 for roads with meadium traffic*
- *k_{pr} = 1.3 for roads with light traffic*

2.7.5.3 Traffic loading

The design traffic loading T_n for assessment of the required strengthening of an existing pavement for the planned life time of n years shall be determined in compliance with this specification

The classification of average daily and design (total) traffic loading into characteristics groups in a life time of 20 years is indicated in Table 4.

Table 4: Classification of traffic loading into groups

Traffic loading group	Number of passages of nominal axle load of 82 kN	
	per day	in 20 years
- exceptionally heavy	above 3,000	above 2 x 10^7
- very heavy	above 800 up to 3,000	above 6 x 10^6 up to 2 x 10^7
- heavy	above 300 up to 800	above 2 x 10^6 up to 6 x 10^6
- medium	above 80 up to 300	above 6 x 10^5 up to 2 x 10^6
- light	above 30 up to 80	above 2 x 10^5 up to 6 x 10^5
- very light	up to 30	up to 2 x 10^5

2.7.5.4 Pavement surface serviceability

As a target value, the pavement surface serviceability is determined with the driving capacity index p amounting to:

- for new, ideally even asphalt carriageways p = 5.0
- for completely worn out (destroyed) carriageway on which the traffic is no more possible p = 0.

As a design limiting value of the driving capacity index at the end of the pavement life time, a value of p_k = 2.0 has been adopted. It signifies the threshold limiting serviceability of the particular pavement surface.

2.7.5.5 Climatic and hydrological conditions

The design effects of climatic and hydrological actions for assessment of limiting thicknesses of pavements shall be determined on the basis of an analysis of conditions and directives for the protection.

2.7.6 Methods of strengthening design

To specify strengthening of existing asphalt pavements, particularly methods based on

- the results of deflection measurements, and
- the visual assessment of the actual condition

shall be considered.

Analytical methods are intended especially for strengthening in special conditions.

2.7.6.1 Deflection based assessment

The entire required thickness of existing asphalt pavement strengthening h_{oj}, shall be assessed on the basis of

- evaluated design deflection of the existing pavement d_m,
- defined limiting value of deflection as a function of traffic loading d_{do},
- design traffic loading T_n,

and either

- on the basis of a suitable diagram, or
- by numerical method.

2.7.6.1.1 Diagram based assessment

On the basis of the interdependence of

- quality of characteristic pavement materials,
- traffic loading, and
- pavement deflection measured with Benkelman beam,

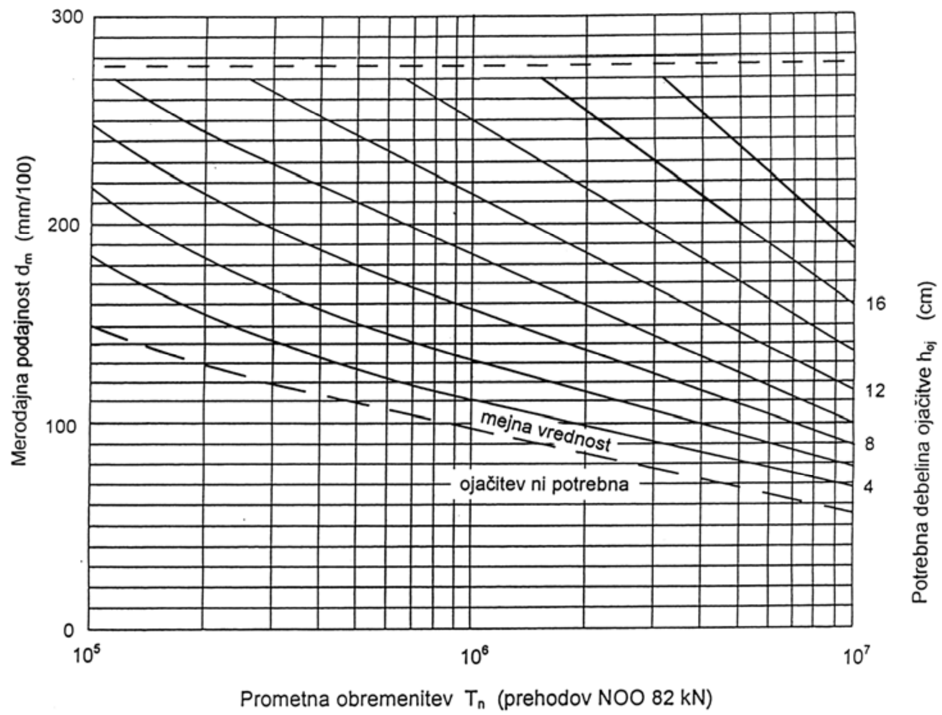
established on the basis of an AASHO test, a diagram has been arranged to assess the required thickness of strengthening of an existing asphalt pavement on the basis of deflection design values evaluated using the results of deflection measurements carried out with a Lacroix deflectograph (Fig. 5).

The required thickness-index of the strengthening layer D_{oj} shall be determined from the equation below:

$$D_{oj} = 0{,}42 \cdot h_{oj} = a_o \cdot h_o + a_{zv} \cdot h_{zv}$$

where

a_o	-	*equivalency factor of asphalt mixture for wearing course (Table 1)*
h_o	-	*thickness of asphalt wearing course*
a_{zv}	-	*equivalency factor of asphalt mixture for upper roadbase*
h_{zv}	-	*thickness of asphalt upper roadbase*



y = design deflection;

x = traffic loading (number of passages of nominal axle load of 82 kN);

potrebna debelina ojačitve = required thickness of strengthening;

mejna vrednost = limiting value;

ojačitev ni potrebna = no strengthening required

Fig.5: Diagram for assessment of required thickness of strengthening of existing asphalt pavement h_{od}

On the basis of the above diagram it is possible to assess the required strengthening for those asphalt pavements only, which subbase consists of mineral aggregate resistant to freezing.

In the diagram, the value of deflection of the strengthened asphalt pavement in the period of the lowest bearing capacity is marked as a limiting value.

The minimum thickness of a strengthening layer amounts to 4 cm.

As a required thickness of strengthening h_{oj} the average value is indicated. However, this value shall not be lower by more 20% at any location.

2.7.6.1.2 Numerical assessment

Where the design deflection d_m under the wheel load of 50 kN amounts to less than 2.5 mm, the logarithmic value of the deflection changes proportionally with the strengthening thickness. So the required thickness of the asphalt pavement strengthening can be calculated from the following equation:

$$h_{oj} = 50 \cdot \frac{\log d_m}{\log d_{do}}$$

where:

d_m – design deflection value of existing pavement,

d_{do} – admitted deflection defined in Table 5 in dependence on the traffic loading group and the design life time.

A numerical assessment of the required thickness of asphalt pavement strengthening is particularly suitable to an informative verification of the thickness assesses on the basis of the design deflection.

2.7.6.2 Assessment based on evaluation of actual condition

The required thickness of an existing asphalt pavement strengthening on the basis of the evaluation of actual condition can be assessed taking account of the following:

- the required pavement thickness (D_{po}) and serviceability of the existing pavement (D_{ob}), or
- the design traffic loading (T_{po}), and the traffic loading already taken (T_{ob}).

Table 5: Limiting values of deflection of apshalt pavement d_{do}

Traffic loading group	Design life time 5 years	10 years	15 years	20 years
	Admitted deflection d_{do} (mm)			
exceptionally heavy	0.8	0.7	0.6	0.5
very heavy	0.9	0.8	0.75	0.7
heavy	1.2	1.0	0.9	0.8
medium	1.5	1.2	1.1	1.0
light	1.7	1.4	1.2	1.1
very light	1.8	1.6	1.4	1.2

2.7.6.2.1 Pavement condition

The residual capacity of the existing pavement to take the traffic loading shall be defined in terms of the thickness-index D_{ob} as indicated in item 5.1.

The required pavement thickness to take the design traffic loading, expressed in terms of the thickness-index D_{po}, shall be assessed according to the method applicable to new constructions as indicated in the TSC 06.520). The bearing capacity of the existing pavement = substrate of CBR value = 15%.

The required thickness-index of the strengthening amounts to:

$D_{oj1} = D_{po} – D_{ob}$ (cm)

Where the thickness of the subbase executed of a suitable mineral aggregate in the existing pavement h_{snob} does not, in view of the substructure bearing capacity, meet the requirements of the increased traffic loading T_{po}, the required greater thickness of the subbase (unbound bearing course) h_{snpo} shall be considered by an additional thickness-index:

$D_{oj2} = 0{,}11 \cdot (h_{snpo} – h_{snob})$ (cm)

For the total required thickness-index

$D_{oj} = D_{oj1} + D_{oj2}$ (cm)

the thickness of an additional surfacing for the strengthening of the existing asphalt pavement shall be assessed using the equation below:

$D_{oj} = a_o \cdot h_o + a_{zv} \cdot h_{zv}$

2.7.6.2.2 Traffic loading condition

A method of assessing the required strengthening of an existing asphalt pavement on the basis of the traffic loading is only appropriate, when no fatigue damage to the placed materials have been noticed on the pavement surface.

As the existing asphalt pavement has already taken a portion T_{n1} of the design traffic loading T_n, it is also capable to take the remaining portion, i.e.:

$T_{n2} = T_n - T_{n1}$

This pavement capacity can be evaluated reasonably by means of a suitable thickness-index D_{ob}.

The further procedure of assessing the thickness of strengthening of an existing asphalt pavement is the same as described for new asphalt pavement

2.7.6.3 Analytical assessment method

In addition to the methods of assessing the required strengthening of existing asphalt pavements on the basis of their deflection and evaluation of their actual condition, analytical methods based on computer software can also be adopted.

The fundamental required information for an analytical method of evaluation of the planned strengthening is the following:

- existing pavement properties:
- layer thicknesses
- E-moduli of materials
- substrate (substructure) bearing capacity
- design traffic loading in the planned life time
- road serviceability at the end of the planned life time taking account of local conditions.

Analytical methods of assessing the required strengthening of existing pavements are particularly suitable to verify the stresses occurring due to flexural (tensile) loading of the strengthened asphalt pavement.

2.7.7 Verification of freezing effect

For a strengthened asphalt pavement both freezing and thawing effect shall be verified as well.

In Table 6 the minimum required pavement thicknesses h_{min} are indicated with regard to the resistance of the existing pavement and of the substructure (substrate) below it, and in view of the hydrological conditions.

Table 6: Minimum required thicknesses of strengthened asphalt pavements h_{min}

Resistance of material below the pavement to freezing and thawing effects	Hydrological conditions	Pavement thickness h_{min}
resistant	favourable unfavourable	$\geq 0.6\ h_m$ * $\geq 0.7\ h_m$
non-resistant	favourable unfavourable	$\geq 0.7\ h_m$ $\geq 0.8\ h_m$

** h_m – frost penetration depth*

In case that the total thickness of frost resistant materials in a strengthened asphalt pavement is smaller than the minimum required thickness h_{min} indicated in Table 6, the thickness of strengthening shall be adequately increased. If the strengthening with a thicker asphalt surfacing is uneconomical, first an unbound bearing layer (subbase) of mineral aggregates shall be placed to the existing pavement, followed by (in view of the required thickness-index of the strengthening) an appropriate surfacing (sandwich). The minimum thickness of such unbound mineral aggregate layer (subbase) shall amount to 10 cm.

GUIDELINES FOR ROAD DESIGN, CONSTRUCTION, MAINTENANCE AND SUPERVISION

VOLUME I: DESIGNING

SECTION 1: ROAD DESIGNING

Part 7: ROAD STRUCTURAL ELEMENTS

Chapter 3: DRAINAGE SYSTEM

Sarajevo/Banja Luka
2005

3 DRAINAGE SYSTEM

3.1.1 General

By appropriate drainage system, an effective and rapid evacuation of all the precipitation water shall be ensured:

- from the roadway, i.e. stabilized pavement surfaces (own water), and
- from all the surrounding surfaces from which the water flow towards the road (rear water).

Therefore, the following drainage types shall be distinguished:

- spread drainage, and
- spot drainage.

A spread drainage of precipitation water from roadways can be executed by spilling over shoulders, whereas a spot drainage by means of individual outlets or drainage devices, which collect the water and lead it to the common spot where it flows away from the road area.

All the precipitation water is collected in the ground, as well as in flowing and standing water, or in a controlled way in the sewage system.

In case of spot drainage of precipitation water from roadways it is not allowed to conduct the water directly into standing surface water, neither into water intended for the preparation of drinking water, nor into the ground water.

Where the criteria for the admitted contamination of water are exceeded, suitable precautions to retain the water on the roadway and to evacuate it from the roadway shall be taken within the scope of the spot drainage. These precautions depend on the type of contamination, which can be permanent of exceptional. A permanent contamination is particularly a periodical washing away of petrol derivates, as well as of rubber tyre, brake-lining, and de-icing salt residuals upon raining. It can be controlled by evacuating the water into adequate retarding structures. An exceptional contamination, which might occur in case of spilling of hazardous materials, can have catastrophic consequences on the ground water and wider area.

The following conditions are relevant to the spread drainage arrangement:

- on roads crossing inter-granular and inter-crack water-holders, if the daily average of passenger car units over a year does not exceed 12,000;
- on roads crossing karstic water-holders, if the daily average of passenger car units over a year does not exceed 6,000;
- on roads crossing areas of materials which permeability amounts to $k \leq 10^{-6}$ m/s, if the daily average of passenger car units over a year does not exceed 40,000;
- on roads where the precipitation water is drained directly into flowing or standing water, if the daily average of passenger car units over a year does not exceed 12,000;

In all other cases a spot drainage shall be foreseen for evacuation of the precipitation water.

When designing the drainage system for roadway and road surroundings, actual conditions at the construction location shall be considered: the source, quantity, and type of the water in all forms. All the water streams, as well as irrigation and draining devices and installations shall be checked (ditches, culverts, devices allowing water to disappear underground).

A general assessment of the ground water is often possible on the basis of the ground shape. A ground prone to sliding can be identified by typical humpy and wavy slope shape, and by sickle-shaped growth of tree trunks. Existence of certain types of plants is also a criterion to estimate hydrological ground conditions: natural vegetation on moist areas differs from that on dry ones. Soaked places or springs on slopes indicate seeping of the water between strata, which, however, is often unidentifiable in a dry season.

Where execution of sinkholes is planned it is reasonable to carry out a water sinking test, which enables an assessment whether draining devices are necessary, and, if so, of which type.

The ground water level is often different from season to season. It is essential to be familiar with the lowest and the highest expected level. It shall be verified, if the ground water levels match at different locations.

3.1.2 Bases for designing

If such provisions related to road drainage are designed that could affect the water, suitable protection of water from contamination or other adverse modifications of its properties shall be foreseen. In case that precipitations falling on the road or its surroundings, or the ground water could affect the road serviceability or durability, such water shall be captured and led to collecting ditch or a sinkhole.

Fig. 9.1 shows a flowchart of selection of road drainage measures.

As any water accumulated on the carriageway impedes the traffic, adequate road drainage shall be ensured, and the water located outside the carriageway shall be prevented to flow onto the carriageway.

The course of road axis and vertical alignment shall be so designed as to be separated as much as possible from the ground water, particularly in water accumulating areas. The ground water protection is discussed in detail in the Guidelines for determination of ground water protection method in motorway area.

As a rule, the position of a road in view of its height shall allow gravitational evacuation of all the water by the shortest way from the road area. The existing natural conditions for draining shall be preserved to the greatest possible extent.

All the devices and installations for road drainage shall be so chosen and designed as to be easily accessible and maintainable. Therefore, modes of road drainage above ground are preferential. On principle, it shall be ensured that the greatest possible percentage of surface water drainage is carried out by spilling over slopes, and by means of grassed curved channels. Where this is not possible for geological, paedological, hydrological, ecological, or constructive reasons, the precipitation water shall be conducted into sink holes – wells. Surface water that cannot disappear underground shall be drained away by means of ditches. If there are water-economy or ecological obstacles for a direct leading away of water into water streams, suitable structures for capturing, retaining, and sinking of precipitation water from pavement surfaces shall be built, and, if circumstances require, appropriate biological cleaning shall be arranged.

All the road drainage devices and arrangements shall be so designed as to resemble the surrounding nature to the greatest possible extent and to be located as close as possible to the road.

Therefore, the road drainage design shall take account of two levels of protection:

- hydraulic and traffic level of protection due to water amount on the carriageway upon heavy rain, and due to the road alignment course, and
- hydro-technical level of protection due to the recipient into which the captured water flows, to the ground water protection, and to the impact of the water recipient on the road vertical alignment (to prevent flooding).

3.1.3 Bases for dimensioning

3.1.3.1 General

Road drainage devices shall be capable to take the water, flowing to them under regular conditions, and to lead it away without any adverse effects. To assess dimensions of those devices it is indispensable to be acquainted with the amount of the water flowing off from the surfaces to be drained.

Dimension, mode, and portion of the overgrowth of surfaces determine different outflow quantities. The latter are also dependent on the ground conditions, growth height, inclination of surfaces, as well as rainfall duration and intensity.

3.1.3.2 Principles of Road Drainage

The following basic assumptions to calculate the road drainage shall be defined by the design:

- size of stabilized and non-stabilized contributory areas,
- differences in road position (road sections on fills or in cuts),
- distances and times of flowing off, and
- arrangement of ditches for precipitation water.

3.1.3.2.1 Bases for Calculation

The following bases apply to the calculation of precipitation water evacuation from a road:

- quantity (intensity) of precipitations,
- frequency of precipitations, and
- surface roughness coefficient.

The calculation of evacuation of precipitation water from a road is based on the experience that a strong rainfall is generally of short duration, and vice versa, a less intensive precipitation is of longer duration.

The precipitation quantity r [l/s · ha], or the precipitation intensity i [mm/min] decreases with an increased duration of the precipitation at the same statistical frequency n. An interdependence of the rainfall quantity r, precipitation frequency n, and duration of rainfall T can be defined by evaluating an appropriate record. The base for calculation of the amount of precipitation of certain duration and frequency is the rain lasting for 15 minutes at frequency n = 1 ($r_{15(n=1)}$), which is once a year reached or exceeded (one-year return period). In case that no information of such a rain is available, the data provided by relevant professional institution for a nearby location shall be considered.

In Table 1 return periods of rainfall for classified roads with regard to the road drainage characteristics are indicated.

Table 1: Rainfall return periods

Road position	Road/path category				
	MW, EW, M1	M2, R1, R2	LR	R3, TR	Public path
	Return period (years)				
- on a fill	5	2	1	0.2	0.2
- in a cut	20	10	5	2	1
- in a depression	50	20	5	2	1
- in a syncline	100	20	5	2	1

For a selected/required return period T the relevant rainfall quantity $r_{T(n)}$ can be assessed by means of the Reinhold equation as follows:

$r_{T(n)} = r_{15(n=1)} \cdot \varphi_{T(n)}$

The coefficient $\varphi_{T(n)}$ is indicated in Table 2.

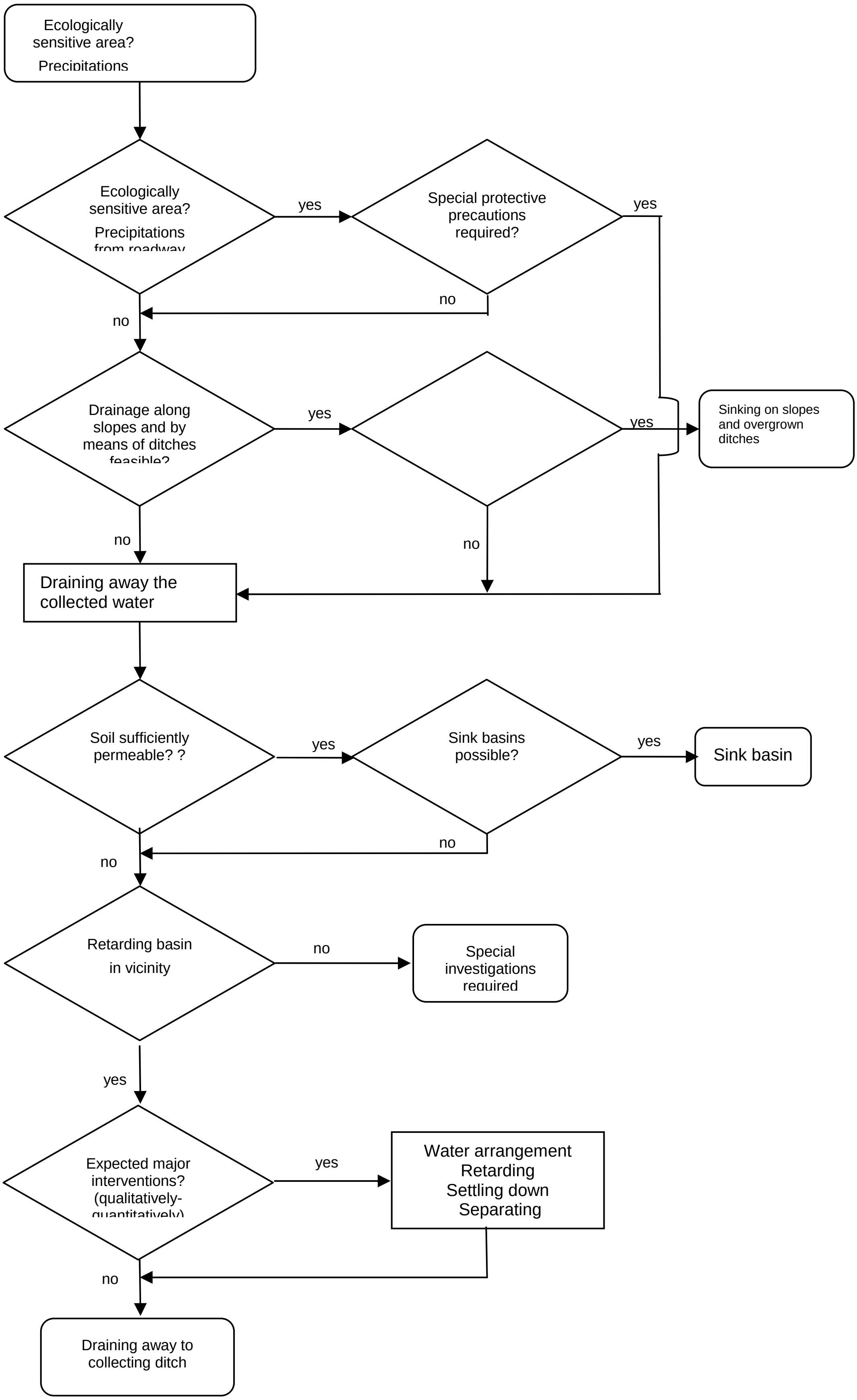


Fig. 1: Flowchart of selection of road drainage measures

The water quantity flowing off from the surface depends on losses, which may result from:

- wetting,
- filling up of depressions,
- sinking, or
- evaporation.

The capacity of draining surface water is defined by the outflow coefficient ψ_s calculated from the following equation:

$$\psi_s = \frac{količina\ odtoka}{količina\ padavin}$$ (outflow quantity/rainfall quantity)

Informative values of the outflow coefficient ψ_s are indicated in Table 3

Table 2: Coefficients φ_T of influence of changed rainfall duration in dependence on the frequency *n*

Rainfall duration	Coefficients φ_T for n =					
[mm]	0.05	0.1	0.2	0.3	0.5	1.0
5	4.740	3.827	3.059	2.666	2.228	1.714
6	4.424	3.572	2.855	2.488	2.079	1.600
7	4.148	3.348	2.676	2.333	1.949	1.500
8	3.904	3.151	2.519	2.196	1.835	1.412
9	3.687	2.976	2.379	2.074	1.733	1.333
10	3.493	2.820	2.254	1.964	1.642	1.263
11	3.318	2.679	2.141	1.866	1.559	1.200
12	3.160	2.551	2.039	1.777	1.485	1.143
13	3.016	2.435	1.947	1.697	1.418	1.091
14	2.885	2.329	1.862	1.623	1.356	1.043
15	2.765	2.232	1.784	1.555	1.300	1.000
16	2.654	2.143	1.713	1.493	1.248	0.960
17	2.552	2.061	1.647	1.436	1.200	0.923
18	2.458	1.984	1.586	1.382	1.155	0.889
19	2.370	1.913	1.529	1.333	1.114	0.857
20	2.288	1.847	1.477	1.287	1.076	0.828
22	2.141	1.728	1.381	1.204	1.006	0.774
24	2.011	1.623	1.298	1.131	0.945	0.727
26	1.896	1.531	1.224	1.066	0.891	0.686
28	1.794	1.448	1.157	1.009	0.843	0.649
30	1.702	1.374	1.098	0.957	0.800	0.615
32	1.619	1.307	1.044	0.910	0.761	0.585
34	1.543	1.246	0.996	0.868	0.725	0.558
36	1.475	1.191	0.952	0.829	0.693	0.533
38	1.412	1.140	0.911	0.794	0.664	0.511
40	1.354	1.093	0.874	0.762	0.637	0.490
42	1.301	1.050	0.840	0.732	0.612	0.471
44	1.252	1.011	0.808	0.704	0.588	0.453
46	1.207	0.974	0.779	0.679	0.567	0.436
48	1.164	0.940	0.751	0.655	0.547	0.421
50	1.125	0.908	0.726	0.633	0.529	0.407

Rainfall duration	Coefficients φ_T for n =					
[mm]	0.05	0.1	0.2	0.3	0.5	1.0
60	0.962	0.776	0.621	0.541	0.452	0.348
70	0.840	0.678	0.542	0.472	0.395	0.304
80	0.746	0.602	0.481	0.419	0.350	0.270
90	0.670	0.541	0.433	0.377	0.315	0.242
100	0.609	0.492	0.393	0.342	0.286	0.220
110	0.558	0.450	0.360	0.314	0.262	0.202
120	0.514	0.415	0.332	0.289	0.242	0.186
130	0.477	0.385	0.308	0.269	0.224	0.173
140	0.445	0.360	0.287	0.250	0.209	0.161
150	0.417	0.337	0.269	0.235	0.196	0.151

Table 3: Informative values of outflow coefficient ψ_s

Type of the ground	Coefficient ψ_s
- up-to-date carriageway	0.9 to 1.0
- ground of low permeability	0.5 to 0.8
- slope of a fill	0.3
- slope of a cut	0.3 to 0.5
- non-stabilized horizontal surface	0.05 to 0.1

3.1.3.2.2 Outflow Calculation

The amount of the precipitation water outflow from a surface shall be assessed by the following equation:

$$Q = r \cdot \varphi \cdot \sum_{i=1}^{n} A_E \cdot \psi_s$$

where:

Q - *outflow from surface [l/s]*

r - *rainfall amount [l/(s·ha)]*

φ - *coefficient of rainfall duration [-]*

A_E - *drained area [ha]*

ψ_s - *coefficient of peak outflow relating to* A_E

As the rainfall duration, the time in which the water to be drained away flows up to the spot relevant to the calculation shall be considered. If the flowing time amounts up to 15 minutes, the rainfall duration shall be assumed 15 minutes for gently sloping contributory areas. For steeper surfaces, shorter rainfall duration (10 minutes) shall be assumed as the water retardation is smaller.

3.1.4 DIMENSIONING OF DRAINING DEVICES

3.1.4.1 Open Ditches

3.1.4.1.1 Ditches and Channels

The quantity of water flow in an open ditch or channel shall be assessed by the Manning-Stickler continuity equation (a flow with a free water level):

$$Q = v \cdot A$$

$$v = k_{st} \cdot R^{2/3} \cdot J^{1/2}$$

$$Q = k_{st} \cdot R^{2/3} \cdot J^{1/2} \cdot A$$

$R = A / O$

where:

Q - quantity of water flow [m^3/s]

v - mean water flow velocity [m/s]

A - ditch cross-sectional area [m^2]

k_{st} - coefficient of roughness of ditch surface (by Strickler) [$m^{1/3}/s$]

R - hydraulic radius [m]

J - ditch vertical alignment fall [‰]

O - wetted circumference of ditch cross-section [m]

Values of coefficients of roughness k_{st} by Strickler are indicated in Table 4.

Table 4: Coefficients of roughness k_{st} by Strickler

Type of ditch/channel	Finishing of walls	Coefficient k_{st}
- natural ditch	solid bottom moderate sediments or overgrowth high capacity of carrying water stream	40 30 – 35 28
- torrent	rough rubble at rest/gravel-stone rough moving rubble/gravel-stone	25 – 28 19 – 22
- earth ditch	solid sand with clay or gravel bottom of sand and gravel, paved slopes rough gravel/balls (ϕ > 50 mm) cloddy clay sand, clay or gravel, extremely overgrown	50 45 – 50 35 30 20 – 25
- lined ditch	wall of brick/clinker, joints perfectly filled-up normal wall rough wall of quarry stone (with pavement)	75 60 50
- ditch finished with cement concrete	steel formwork or smooth flooring plaster wooden formwork without plaster old cement concrete, clean surface	90 65 – 70 60
- ditch finished with bituminous mixture	non-uniform cement concrete surface dense fine-grained surface	50 80
- curved channel	bottom with alluvial material grassed of gravel paved	30 – 50 20 – 30 25 – 30 40 – 50

The cross-sectional area A of the water flow, the wetted circumference O of the water flow, and the hydraulic radius R are indicated in Table 5 for some most frequent ditch and channel cross-sections.

Table 5: Characteristics of sections of ditches and channels for road drainage

Ditch/channel cross-section	Cross-sectional water flow area A	Wetted circumference of water flow O	Hydraulic radius R
- rectangular	b · h	b + 2h	$\frac{b \cdot h}{b + 2h}$
- triangular	m · h^2	$2h \cdot \sqrt{1 + m^2}$	$\frac{m \cdot h}{2\sqrt{1 + m^2}}$

- trapezoidal	h · (b + m · h)	$b+2h\cdot\sqrt{1+m^2}$	$\frac{h\cdot(b+m\cdot h)}{b+2\sqrt{1+m^2}}$
- trough-shaped (segmental)	$\frac{2}{3}\cdot b\cdot h$	$b\cdot\left(1+\frac{2}{3}a^2-\frac{2}{5}a^4\right)$	$\frac{2h}{3\cdot\left(1+\frac{2}{3}a^2-\frac{2}{5}a^4\right.}$

Designations indicated in the table above signify the following:

h - ditch or channel depth [m]

b - ditch or channel bottom width [m]

m - slope inclination (1 : m) [-]

a = 2h/b

3.1.4.1.2 Channels with Kerbs, Pointed Channels

The water flow quantity in channels with kerbs and pointed channels can be assessed by the simplified Manning-Strickler equation as follows:

$$Q = k_{st}\cdot h^{8/3}\cdot J^{1/2}\cdot\frac{0{,}315}{q} \qquad [m^3/s]$$

where:

h - water flow depth at kerb [m]

J - channel/ditch vertical alignment fall [m/m]

q - channel/ditch cross-fall [m/m]

3.1.4.1.3 Curved Channels

The water flow quantity in curved (trough-shaped, segmental) ditches can be assessed by the following equation:

$$Q = k_{st}\cdot h^{8/3}\cdot J^{1/2}\cdot\frac{b}{2h} \qquad [m^3/s]$$

where:

h - water depth in the middle of the trough (depression) [m]

b - trough (depression) width [m]

A simplification of water flow calculation is feasible by introducing available tables and diagrams.

3.1.4.2 Pipelines

For dimensioning of pipelines/sewage system the water flow quantity can be assessed by the equation by Prandtl and Colebrook taking account of the retaining capacity of a (filled-up) pipe:

$$Q = \frac{\Pi\cdot d^2}{4}\cdot\left[-2\lg\left(\frac{2{,}51\cdot v}{d\cdot\sqrt{2g\cdot J\cdot d}}+\frac{k_b}{3{,}71\cdot d}\right)\right]\cdot\sqrt{2g\cdot J\cdot d}$$

where:

Q - water flow (discharge) [m^3/s]

d - internal pipe diameter [m]

J - pipe fall [m/m]

g - gravitational acceleration [m/s^2]

v - kinematical water viscosity [m^2/s]

k_b - roughness coefficient [mm]

The roughness coefficient k_b does not only depend on the losses due to the material present in a straight pipe, but also on the losses at joints, on fabrication and installation inaccuracies, as well as on losses due to shafts and connections. For cement concrete pipes the roughness coefficient generally amounts to k_b = 1.5, whilst for plastic pipes k_b = 0.4 mm.

3.1.5 Road draining devices

Surface drainage along roads is made feasible by the following devices:

- curved (trough-shaped) ditches,
- trapezoidal and triangular ditches, and
- channels and gutters.

The water ariving from the side of a hill to the slope of the road cut shall be drained away by means of intercepting channels above the slope of the cut.

3.1.5.1 Curved Road Ditches

In view of the road traffic safety, curved (trough-shaped) ditches are the best. The most appropriate curved ditches are grassed ones, if such stabilization method is tolerated by the hydraulic conditions.

Curved (trough-shaped) ditches shall collect the water being drained off from both stabilized and non-stabilized road surfaces. If this water does not sink into the ground water it shall be led forth into collecting ditches (main drains).

As a rule, curved ditches shall be executed next to the fill or cut toe, and shall form a transition to the surroundings.

Curved ditches shall be 1.0 – 2.5 m wide (b), and at least 0.2 m deep (h), however not deeper than b/5.

The longitudinal fall of the bottom of a curved road ditch (J) shall follow the ground inclination or the longitudinal fall of the carriageway edge. If such a fall is insufficient to evacuate the water, the hydraulic capacity of the ditch shall be improved by increasing the ditch bottom fall or the cross-section, by carrying out smooth bottom, or constructing sink shafts.

To ensure the water outflow and to protect the ditch from erosion, the curved (trough-shaped) road ditch shall be suitably stabilized. Informative modes of stabilizing surfaces of curved road ditches are indicated in Table 6 and shown in Figures 2 – 5.

Table 6: Informative modes of stabilizing curved road ditches in dependence on the bottom longitudinal fall

Longitudinal fall J of ditch bottom (%)	Mode of stabilizing ditch bottom
< 1	smooth stabilization, if required for hydraulic reasons (e.g. bottom panelling)
1 to 3	grassing
3 to 10	rough bottom
> 10	rough lining of trough (depression)

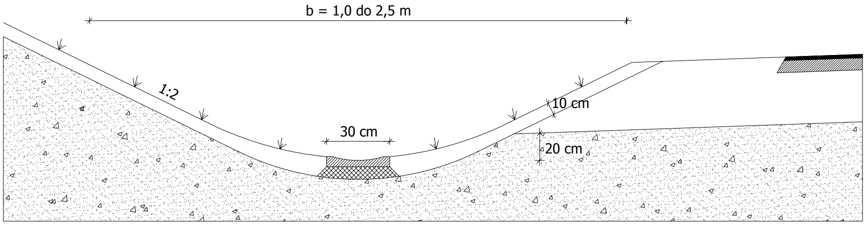


Fig 2: Curved (trough-shaped) road ditch with a smooth bottom for a longitudinal fall of J < 1 %

The smooth bottom shall be executed in a minimum width of 30 cm, however not exceeding a half of the curved ditch width (b/2). To stabilize the bottom, prefabricated cement concrete elements, bituminous mixtures, or paving stones can be used, which shall be laid onto an adequate substrate such as a sand layer of minimum 10 cm thickness.

The bottom of a curved road ditch shall be buried by at least 20 cm into the solid soil. It can also be constructed in such a way that its bottom is situated above the fill formation level. As in such a case the ditch only drains away the surface water from both roadway and slopes, suitable deep drainage (e.g. by means of a drainage below the curved ditch) shall be ensured to evacuate the water arriving from the fill formation.

The inclination of slopes of a curved (trough-shaped) road ditch shall not be steeper than 1:2. As a rule, the slopes shall be grassed, thus an approximately 10 cm thick topsoil layer is indispensable.

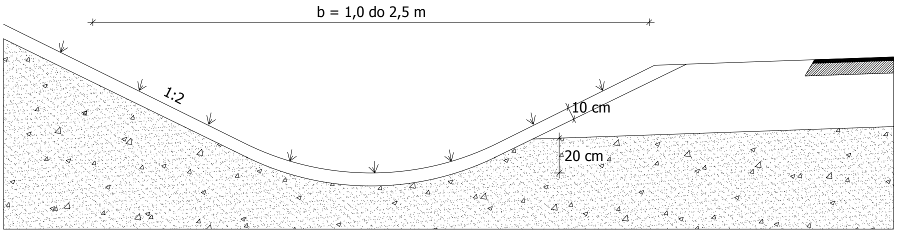


Fig. 3: Grassed curved road ditch for a longitudinal fall of J = 1 % to 3 %

If there is a danger of erosion, the ditch surface shall be stabilized with turf or grassed reed-mat.

Where major amounts of water have to disappear underground from a grassed curved road ditch, the latter can be designed as a sink depression.

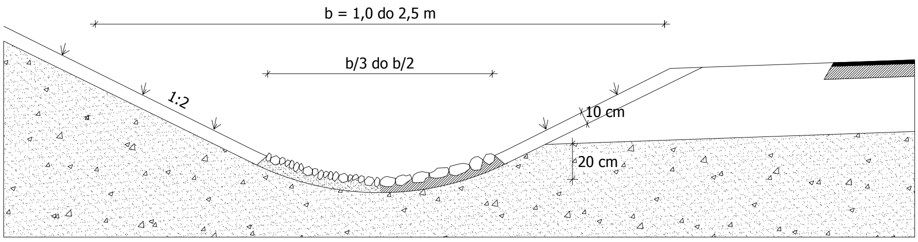


Fig. 4: Curved (trough-shaped) road ditch with a rough bottom for a longitudinal fall of J = 3 % to 10 %

In a curved (trough-shaped) road ditch with a rough bottom, the water flow velocity and the hazard of erosion shall be diminished.

For a longitudinal fall of the ditch of J = 3 % to 5 % it is appropriate to place rough crushed stone onto the ditch bottom in a width equal to a third of maximum to a half of the ditch width. The gravel or crushed stone underlay shall be approximately 10 cm thick.

For a longitudinal fall of the ditch of J = 5 % to 10 %, quarry s-stone, paving stones or turf pavers shall be placed to the ditch bottom onto an underlay of gravel or crushed stone of 10 cm thickness approximately. In exceptional cases cement concrete can also be applied as the underlay.

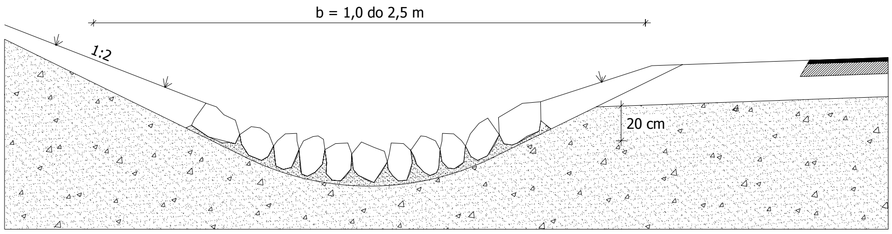


Fig. 5: Curved (trough-shaped) road ditch with a rough lining for a longitudinal fall of J > 10 %

The rough lining of a curved road ditch shall be so executed as to reduce the water flow energy to a harmless level.

A curved (trough-shaped) road ditch with a roughed lining can be constructed along the road at the slope toe or on the slope.

Pieces of quarry-stone measuring 20 to 30 cm shall be placed, tightly one to the other, to a gravel or crushed stone underlay, which thickness shall amount to approximately 15 cm. In case of non-cohesive soils they shall be laid onto the excavation formation as well. In certain cases, quarry-stones shall be placed onto a cement concrete layer.

Between the quarry-stone pieces crushed stone shall be placed up to a half a height.

For greater longitudinal falls of curved ditches with a rough lining wooden poles (ϕ 8 to 10 cm, l = 80 to 120 cm) or steel bars (ϕ 28 cm, l = 80 cm) rammed into the substrate might be necessary to ensure stability.

Erosion of the rough lining shall be prevented by placing larger pieces of quarry-stone at the ditch edges, and by additional fascines enabling vegetation.

To reduce the water flow velocity, sills or cascades can be introduced instead of quarry-stones.

3.1.5.2 Trapezoidal Road Ditches

Trapezoidal road ditches play the same role as curved (trough-shaped) ones, however their hydraulic capacity is generally higher.

The bottom width of trapezoidal road ditches shall amount to 50 cm. Only exceptionally it may be 40 cm.

The depth of a trapezoidal ditch at the roadway shall be such that the bottom of the ditch is situated at least 20 cm below the solid soil or fill formation level.

If directed by the hydraulic conditions, dimensions and shaping of trapezoidal road ditches can be different as well.

Suitable inclination of slopes of trapezoidal road ditches is 1 : 1.5. If permitted by ground conditions, they can even be steeper. The slopes shall be made green, and their upper edges shall be rounded off.

Longitudinal fall J of trapezoidal road ditches shall not be less than 0.3%. For smaller longitudinal falls the water outflow can be improved by a smooth stabilization of the ditch bottom, e.g. with precast cement concrete elements, or by paving with quarry-stones or paving stones.

When designing trapezoidal road ditches it shall be verified whether the bottom of the slopes of the ditch shall be protected from erosion, taking account of the soil type, longitudinal fall of the ditch, and the water amount to be evacuated.

To protect the cross-section of a trapezoidal ditch (bottom and/or slopes), natural stone, precast cement concrete elements, wooden wattles, and similar material can be used. On sandy ground, a layer of filter material or geotextiles might be necessary.

Examples of stabilization of trapezoidal road ditches are schematically presented in Figures 6 – 10.

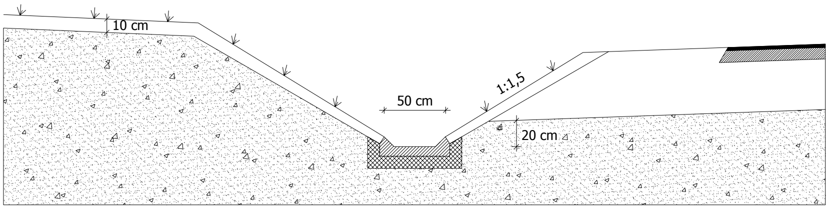


Fig. 6: Trapezoidal ditch with a smooth bottom (precast concrete element) for longitudinal falls of J< 0.3%

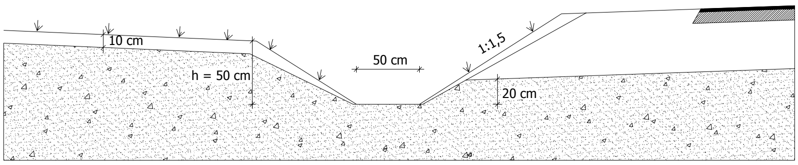


Fig. 7: Trapezoidal ditch without stabilized bottom, but with grassed slope (turf, if required) for longitudinal falls of J = 0.3 % to 3 %

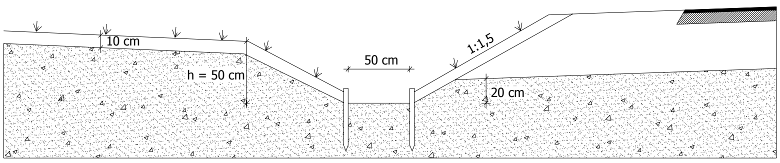


Fig. 8: Protection of cross-section of trapezoidal ditch with wooden poles and longitudinal wooden wattle

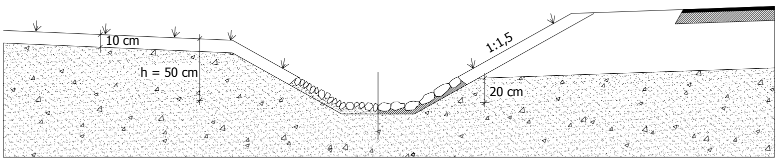


Fig. 9: Protection of cross-section of trapezoidal ditch with turf paving or quarry-stones (on a layer of sand or cement concrete blinding) for longitudinal slopes of J = 3 % to 10 %

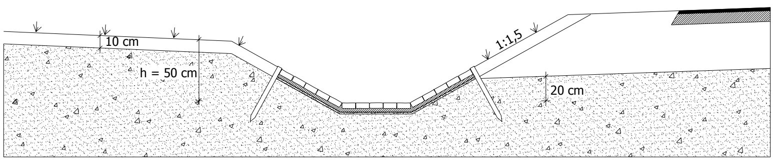


Fig 9.10: Protection of cross-section of trapezoidal ditch with paving stones placed onto layers of cement mortar and sand, and with poles for longitudinal falls J > 10 %

3.1.5.3 Intercepting Ditches

A trapezoidal or curved (trough-shaped) cross-sectional shape can also be provided for intercepting ditches on slopes above cuts. They are intended for capturing and draining away of slope water into collecting ditches.

The width of the intercepting ditch bottom shall amount to at least 0.3 m, whereas it depth shall be 0.2 to 0.5 m.

If the water seeping from the intercepting ditch endangers the slope stability, the ditch shall be sealed at least up to the half of its height, e.g. with a 20 cm thick layer of cohesive soil or with a waterproofing layer of synthetic material covered with topsoil (Fig. 11).

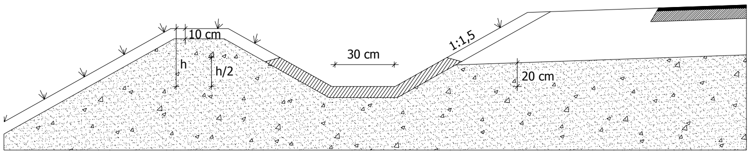


Fig. 11: Intercepting ditch on the slope above the cut, sealed with cohesive soil

3.1.5.4 Triangular Road Ditches

Triangular road ditches are only suitable to evacuate smaller water amounts, and, in view of their construction, to a limited space.

The inclination of slopes of triangular road ditches must not be greater than 1:3. The ditch bottom shall be buried by at least 0.4 m deep in the solid soil or 20 cm below the pavement substructure formation (Fig. 12). On principle, triangular road ditches shall be grassed.

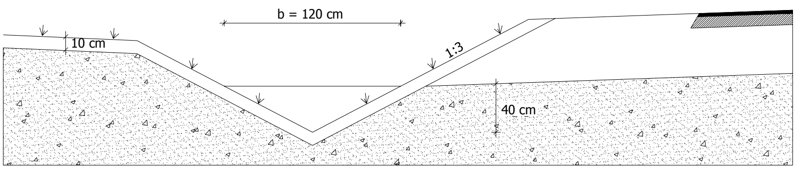


Fig. 9.12: Grassed triangular road ditch

2.1.9.1.1 Channels and Gutters

Channels and gutters drain off the water arriving from both sides or from one side only, along the road, to suitably designed outlets or directly into collecting drain ditches.

Channels shall be 0.5 to 0.9 m wide.

Longitudinal fall of channels and gutters shall be, as a rule, similar to the longitudinal fall of the edge of the drained adjacent carriageway, however it shall amount to at least 0.5 %. If the longitudinal fall of the carriageway edge is less than 0.5 %, the required longitudinal fall of the channel bottom shall be ensured by alternative varying the channel bottom.

The cross-fall of the channel surface towards the kerb shall amount to 10 % - 15 %. In case of varying bottom, it shall amount to 5 % to 15 %, however not less than the value of the cross-fall of the nearby carriageway (Fig. 13).

The channel top layer made of cement concrete, bituminous mixture or paving stones shall be laid onto suitable bearing underlay, e.g. of non-stabilized mixture of mineral grains).

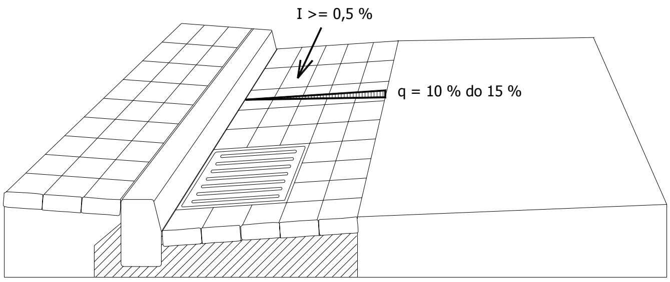


Fig. 13: Channel at the carriageway and footway edge

Gutters for draining away the surface water can either be trough-shaped or made of suitably formed precast elements covered with a grid, or only with a notch to allow water inflow.

A curved (trough-shaped) gutter shall enable passage of vehicles, therefore it must not be deeper than 1/15 of its width, however not less than 3 cm. The width of a curved gutter may amount to 0.5 to 1.0 m. The top layer can be paved or made of bituminous mix; the bearing substrate shall be adjusted to the structure of the boundary traffic surfaces (Fig. 14).

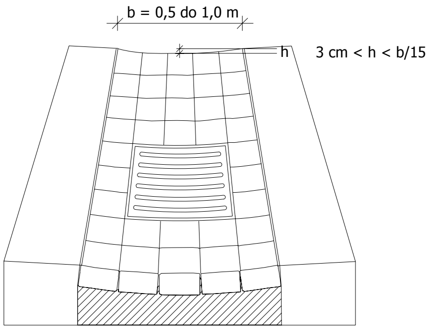


Fig 14: Paved curved (trough-shaped) gutter allowing vehicles to pass

A gutter covered with a steel grid shall allow vehicles to pass as well; therefore it shall be built-in onto a substrate of suitable bearing capacity.

To allow evacuation of water, prefabricated elements shall have a clear cross-section of b/h ≥ 10 cm/6 cm (Fig. 9.15). They shall be made of cement concrete with admixtures, or of synthetic materials.

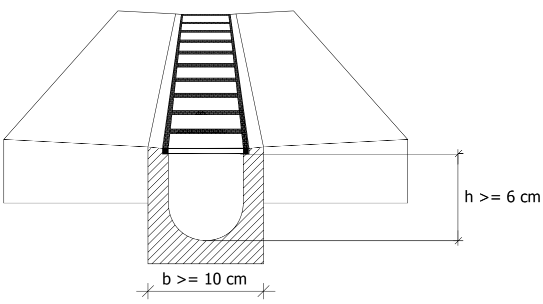


Fig. 15: Prefabricated gutter with a steel grid

Gutters with a notch to allow water inflow into a circularly shaped middle of a precast element enable a continuous drainage of traffic surfaces.

The diameter of the circular opening in the element shall amount to at least 10 cm, whilst the notch width shall be 13 mm to 30 mm (Fig. 16).

Prefabricated elements shall be made of cement concrete with admixtures to comply with the foreseen static and dynamic loading.

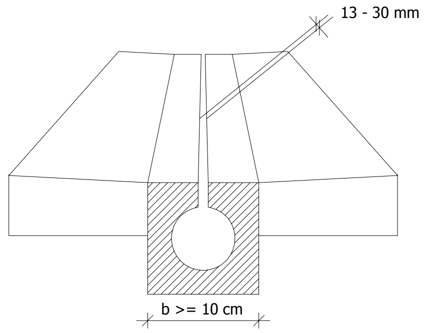


Fig. 16: Prefabricated gutter with a notch