Master Thesis

Natural Rock Walls as retaining structures for slope stabilization

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Statutory Declaration

I declare that I have authored this thesis independently, that I have not used other than the declared sources / resources and that I have explicitly marked all material which has been quoted either literally or by content from the used sources.

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Abstract

Natural Rock Walls have a widespread application. They are used especially for retaining slopes, terrain steps, cuts and fills because of their flexibility, aesthetic, natural like appearance and lower costs. These Natural Rock Walls are constructed by almost every construction company with experience as basis. Often no calculations or structural design are done for these constructions, what affect a higher damage rate.

The aim of this master thesis is to give information and recommendations for constructing high quality Natural Rock Walls. Material properties, construction types and calculation approaches of Natural Rock Walls are investigated. Furthermore pull out trials with up to 550 kg heavy blocks and different chinking materials are performed. Moreover different claims, their frequent causes of damage and their remediation are analysed.

Zusammenfassung

Steinschlichtungen haben eine große Anwendungsbandbreite und werden wegen ihrer Flexibilität, naturnahen Ansichtsfläche und vergleichsweise günstigen Kosten heutzutage vor allem zur Sicherung von Geländesprüngen, Böschungen, Einschnitten und Anschüttungen verwendet.

Sie werden von fast jeder Baufirma hergestellt, wobei der Herstellungsprozess von Steinschlichtungen nur auf Erfahrungswerten der jeweiligen Baufirmen beruht und ihm oftmals keine Berechnungen zu Grunde liegen, was zu einer verhältnismäßig hohen Schadensanzahl führt.

Ziel der Arbeit ist es Hinweise und Empfehlungen für die Herstellung von qualitativ hochwertigen und sicheren Steinschlichtungen zu geben. Dabei werden die Materialeigenschaften, Konstruktionsweisen und Berechnungsarten von Steinschlichtungen untersucht. Weiters wurden Auszugsversuche mit bis zu 550 kg schweren Blöcken und unterschiedlichen Auszwickungsmaterial durchgeführt. Ein weiterer Teil der Arbeit befasst sich mit der Analyse von Schadensfällen, den häufigsten Schadensursachen sowie ihrer Sanierung.

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Abbreviations

ARC	associated rockery contractors
BS	design situation
CC	consequence class
EQU	static equilibrium
FHWA	federal highway administration
GEO	total stability
GK	geotechnical category
HMB	heavy mass block
HYD	piping
LMB	light mass block

ÖBB		austrian federal railways
SLS		serviceability limit state
STR		structural stability
ULS		ultimate limit state

C_C	[-]	curvature coefficient
C_U	[-]	coefficient of uniformity

φ	[°]	friction angle
φ_A	[°]	alternative friction angle
φ_r	[°]	residual shear angle
α	[°]	response angle
μ	[-]	coefficient of friction
δ	[°]	wall friction angle
m	[°]	material value
c	[kN/m²]	cohesion
γ	[kN/m³]	unit weight
G	[kN]	weight
Z	[kN]	shear force
E_A	[kN/m²]	active earth pressure
E_P	[kN/m²]	passive earth pressure
E_0	[kN/m²]	earth pressure at rest
UCS	[kN/m²]	unconfined compressive strength

$\eta\ (bearing\ capacity)$	[-]	utilisation factor bearing capacity
$\eta\ (slope\ failure)$	[-]	utilisation factor slope failure
$\eta\ (sliding\ foot)$	[-]	utilisation factor sliding
$\eta\ (sliding\ blocks)$	[-]	utilisation factor sliding of single blocks
$\eta\ (EQU)$	[-]	utilisation factor static equilibrium

γ_F	[-]	partial factors of safety for the loads
γ_E	[-]	partial factors of safety for the demands
γ_M	[-]	partial factors of safety for the soil parameters
γ_R	[-]	partial factors of safety for the resistance of retaining structures, slope stability

1. Purpose of study

The aim of this master thesis is to give information and recommendations for constructing high quality Natural Rock Walls. The material properties, construction practices and calculations of Natural Rock Walls should be reviewed.
Another goal of research is the analysis of claims, avoiding of damages and the opportunity of reconstruction of Natural Rock Walls.

The ultimate goal is the creation of a guideline of the design and construction of Natural Rock Walls.

2. Introduction

Natural Rock Walls are widely-used especially in the function as retaining walls. One of the reasons why people are using this type of walls instead of reinforced concrete walls is that they are flexible, more aesthetic, naturally like and cheaper.

In Austria there are no specific standards and guidelines for design and construction of Natural Rock Walls.

Many construction companies are building Natural Rock Walls, but many of these constructions have no structural design as a basis. Most of the companies are building Natural Rock Walls only by their experience. That means: “The seemingly simple design of a Natural Rock Wall often results in an improperly construction” [8].

Fig. 1: Natural Rock Wall

2.1. Definition

There are several different definitions for Natural Rock Walls especially in the German language. To understand the definition used in this master thesis all definitions based on literature are explained in the following passages.

Rip Rap (Steinwurf)

„The unmachined, preferable cubic quarry stones should be dumped or thrown in a way that the single stones are interlocking. If necessary they have to be arranged."[g] Rip Raps are used for erosion control on costlines or on river shores.

Dry Stone Wall (Trockensteinmauer)

These walls are built by hand, using stones on site. The stones can be rounded as well as angular. There is a long tradition of building dry stone walls in England and Switzerland. Dry Stone Walls are used as boundary walls on land but also as retaining walls.


Fig. 2: Rip Rap (Steinwurf) [I] and Dry Stone Wall (Trockensteinmauer) [II]

Natural Rock Wall (Rockery, Steinschlichtung)

Two different definitions can be found in the literature,

„The raw, preferable cubic quarry stones are to deliver and should be stacked according to the profile in a stone bond with rough surface. The joints between the single blocks should be filled with coarse stone material or concrete." [g]

"Rockeries can be generally defined as rough rocks stacked in an "interlocking" pattern without concrete, mortar or steel reinforcement. Neither mechanical nor

physical connections are made between the individual rocks; interlocking is accomplished through proper rock layout, rock weight and frictional interaction.” [4]
These types of Natural Rock Wall are built by using an excavator with grab.

<u>Natural Rock Wall (Steinsatz)</u>

„The one sided flat quarry stones are set individually in a dense stone bond preferable with close joints. The residual voids must be smaller than 10% and should be filled with coarse stone material or concrete. For coarse stone material angular-regular material without fine grain fraction should be used. The horizontal joints should be normal to the inclined plain view. The vertical joints should be arranged shifted. The inclined plain view should be almost planar.“ [g]
Also this type of Natural Rock Wall is built using an excavator with grab.

Fig. 3: Natural Rock Wall (Steinschlichtung) and Natural Rock Wall (Steinsatz)

In this master thesis the definition of Natural Rock Wall is used as followed:

Raw, preferable cubic quarry stones should be stacked in a dense stone bond. The voids between the single blocks should be as small as possible and the joints as close as possible. Depending on the usage of the Natural Rock Wall the joints should be filled with coarse angular-regular stone material or concrete. Furthermore the horizontal joints should be normal to the inclined plain view and the vertical joints should be arranged shifted. The inclined plain view should be almost planar.

2.2. Areas of application

The application of Natural Rock Walls can be subdivided in two main parts:

- Erosion protection of slopes
- Retaining walls for terrain steps, cuts and fills

Other applications are for example drainage ribs and retaining of shorelines.

This master thesis deals only with Natural Rock Walls which are used for retaining terrain steps, cuts and fills. That means Natural Rock Walls are used as retaining walls/structures.

3. Theory

3.1. Material properties

One of the main components for construction of safe and high quality Natural Rock Walls are the properties of the different used materials. Most of the used materials e.g. blocks, material for chinking, drainage, backfill, foundation are naturally occurring. This means the material is not engineered like steal or concrete and most of them show anisotropic behaviour (different characteristics in different directions).

3.1.1 Foundation

Like all buildings Natural Rock Walls need a foundation. The foundation depends on the construction load, external load and the ground conditions. Depending on these factors there could be the possibility that for the foundation of a Natural Rock Wall a soil exchange, laying in mortar bed or even a strip foundation has to be done. Furthermore the foundation of the Natural Rock Wall must be in a freezing free depth, which depends on the climatic location of the construction. In Austria the average frost free foundation depth is in the order of about 0,8 m to 1,0 m.

Generally there are not so high requirements of settling sensitivity for Natural Rock Walls. Settlement rates through the whole construction in the range of a few centimetres can be accepted within Natural Rock Walls because of their flexibility. Higher settlement rates, especially within differential settling cannot be accepted and will lead to instability or even failure of the Natural Rock Wall.

To increase the stability against sliding failure the foundation can be constructed with an inclination into the slope.

Fig. 4: Foundations [6]

3.1.2 Blocks

The size, shape, rock type and rock properties of the single blocks are essential for the construction of Natural Rock Walls.

To make sure that the used blocks have the same properties only blocks from quarries should be used which fulfil the standard of ÖNORM EN 13383-1 Armour stone. Therefore this standard is the basis of the following sections.

<u>Size</u>

The requirements of the standard [d] are very useful for blocks in Natural Rock Walls. The blocks are classified on the basis of their weight in so called stone classes. For Natural Rock Walls the heavy stone class HMB 300-1000 and HMB 1000-3000 should be used. HMB 300-1000 means a class of blocks with a weight from 300 kg to 1000 kg and HMB 1000-3000 means a class of blocks with a weight from 1000 kg to 3000 kg.

To get an idea of the size of the single blocks with an average unit weight of 2600 kg/m³ and a cubic shape the edge length is listed in table 1.

weight [kg]	**approx. edge length [m]**
100	0,40
300	0,50
500	0,60
1000	0,75
1500	0,85
2000	0,95
2500	1,00
3000	1,05

Tab. 1: Weight, edge length of armour stones

Depending on the height of a Natural Rock Wall, blocks with a different size can be used. For high Natural Rock Walls big blocks should be used, because of the need of higher weights and to speed up the construction. However for low Natural Rock Walls smaller blocks can be used.

The cap block, which is the last block on the top of the wall, must have certain dimensions, depending on design and to have the security of being unliftable. *Marte* [2] recommended that the cap rock has a width greater than 0,8 m.

Fig. 5: Armour stones [6]

Shape

The shape of the single blocks depends on the texture of the rock with its planes of weakness like fractures, foliation and veins. Also the different mining methods play a role for the shape of a block.

The blocks should be roughly rectangular, tabular or cubic in shape. Blocks that are triangular in shape should not be used, because they are difficult to stack in a stable configuration. Rounded blocks and cobbles should also be avoided. The rounded nature of the blocks reduces the potential for interlocking and generally results in a less stable structure. [4]

There are different requirements in the standard [d] and the code of practice [h]. The standard says that the height to length ratio of a block should be smaller than 1:3. On the other hand the code of practice says that the length must be higher than 0,8 m and the height must be smaller than 2/3 of the length and higher than 1/5 of the length.

As a rule of thumb the used blocks should have a height to length ratio of higher than 1:2 and smaller than 1:5.

Rock type and Rock properties

The used blocks for Natural Rock Walls have to fulfil at least the physical and chemical requirements of the standard [d].

That means the used blocks must fulfil the physical and chemical requirements in the category groups.

Physical requirements	Chemical requirements
Rock density	Water absorption
Resistance against breakage	Freeze thaw durability
Resistance against loss	Resistance against salt crystallisation
	Sun burn

Tab. 2: Physical and chemical requirements [d]

Otherwise rock decomposition could lead to shifting, settlement, or loss of contact between the blocks [4].
Following rock types should not be used as blocks in Natural Rock Walls. They are not able to fulfil the requirements.

Rock types
Conglomerate
Breccia
Phyllite
Chalkstone
Marl
Clay shale
Rocks of cemented clay minerals

Tab. 3: Rock types which should be avoided in Natural Rock Walls [d]

Freeze-thaw durability

One of the most important things of the material properties is the freeze-thaw durability. This durability depends on the mineral composition of the used rock type.

For this material property the clay minerals play an essential role. Clay minerals form by weathering, sedimentation and hydrothermal activity. The type of clay mineral which will be formed is depending on the parent material and the climate.
The term “clay” has two different definitions. It is a grain size definition, depending on the used classification of clay, which can be smaller than 20 µm or smaller than

2 µm. On the other hand it is a mineralogical definition, which means clay is a composition of different clay minerals. A clay mineral is defined as a phyllosilicate with $(Si_2O_5)^{2-}$ as base unit. Furthermore not only one single clay mineral occur, there also can be a composition of different types of clay minerals, the so called mixed-layer clay minerals.

The phyllosilicates can be divided in two groups (table 4). The 1:1 type minerals (two layer) and the 2:1 type minerals (three layer). Phyllosilicates consist of tetrahedral- and octahedral sheets. A 1:1 type mineral means one tetrahedral sheet and one octahedral sheet, 2:1 type minerals means two tetrahedral sheets and one octahedral sheet. Furthermore the octahedral sheet has three sites which can be occupied by cations. It is called trioctahedral if all three sites are occupied by a divalent cation (e.g. Mg^{2+}) and dioctahedral if two of the three sites are occupied by a trivalent cation (e.g. Al^{3+}).

1:1 type minerals (two layer)	**2:1 type minerals (three layer)**
Serpentine	Talc and Pyrophyllite
Kaolinite	Micas
Halloysite	Vermiculite
	Smectite

Tab. 4: Typical Phyllosilicates

The most interesting clay minerals due to the freeze-thaw durability are Vermiculite and Smectite, which have a layer charge, a high cation exchange capacity and a high specific surface. In contrast to these clay minerals the 1:1 type minerals have no layer charge. The layer charge results from substitution of cations in the tetrahedral or octahedral layer. Between the layer packages (tetrahedral-octahedral-tetrahedral) of Vermiculite and Smectite is an interlayer in which, depending on the layer charge, different cations and H_2O molecules can be built in. Only within these minerals the intra-crystalline swell ability is possible, because of this layer charge. The interlayer water content is reversible and depends on the cation. The higher the specific surface and the higher the cation exchange capacity the more water can be adsorbed and the volume of the clay minerals increases.

Smectite and Vermiculite have the highest specific surface and cation exchange capacity and are therefore the most swell able clay minerals.
In summary, a high content of clay minerals, especially swell able clay minerals like Smectite and Vermiculite result in very low freeze-thaw durability. During the wetting and drying of the rock the swell able clay minerals extend and shrink. If the swelling pressure is high enough it destroys the texture of the rock and the rock will disintegrate.
To guarantee high freeze-thaw durability the used blocks in a Natural Rock Wall must fulfil the requirements of the standard [d]. This means if the blocks have a water absorption less than 0,5 % and no fractures are formed during the water absorption test the blocks fulfil the requirements. But if the water absorption of the rock is higher than 0,5 % further tests should be taken.

3.1.3 Chinking

Chinking is called the filling between the single blocks. There are different possibilities of materials which are used for chinking.

- block above block, no filling
- fine grained material
- coarse grained, angular material
- mortar

Fig. 6: Different types of chinking a.) no chinking, b.) fine grained material, c.) angular, coarse grained material, d.) mortar

<u>Block above block</u>

Blocks are stacked above blocks without filling between the single blocks. This kind of chinking is possible when the blocks are very planar. Such blocks are for example machined quarry stones. But in most of the cases the natural non machined blocks are not planar. If the non-planar blocks are stacked it comes to a point bearing. There is hardly a contact between the different blocks. They just touch each other through specific points.

Fine grained material

If it is used material with a high fine grain fraction for chinking the bearing will be extensive but the friction between the blocks will also be low. Another problem is that the fine grain fraction will easily be washed out from water and the Natural Rock Wall will become instable.

Coarse grained, angular material

Using this material will cause an extensive bearing between the blocks and a higher friction. The material also cannot be washed out easily. Another benefit of using coarse grained, angular material for chinking is that this material provides good drainage of incoming water behind the Natural Rock Wall.

Mortar

Mortar as chinking will also cause an extensive bearing between the blocks and a high friction and cohesion. The material cannot be washed out. In this case drainage drillings are needed to remove the water behind the Natural Rock Wall, respectively to prevent water pressure.

The best choice of material for chinking for not regular blocks should be coarse grained, angular material and mortar because of the extensive bearing between the blocks, higher friction and in the case of mortar also cohesion. For very regular blocks (machined blocks) no chinking should be used.

To confirm these theoretical considerations and to verify the amount of friction between the blocks with different materials pull out trials in the geotechnical laboratory of the TU Graz, Austria has been done.

3.1.4 Pull out Trials

The pull out trials were performed in the laboratory of the institute of soil mechanics and foundation engineering, TU Graz, Austria. With these trials the amount of friction in the joint between the blocks with different chinking material should be determined.

3.1.4.1 Material properties

To define the conditions of the pull out trials the properties of the used materials have to be known. Therefore the blocks and the chinking material is analysed mineralogically and geomechanically.

<u>*Blocks*</u>

Five blocks with a mass between 280 kg and 550 kg were provided for the pull out trials from the "Kanzel Steinbruch" in Gratkorn, Austria.
Samples were taken from the blocks, two thin sections of the rock were created and an x-ray powder diffractogram has been done. Therefore the rock can be described in petrographic detail. First of all the blocks are described macroscopic and then the thin sections are described microscopically. Furthermore a core sample of one block was taken and an unconfined compressive strength test (UCS) was done.

Fig. 7: Blocks from "Kanzel-Steinbruch"

Macroscopic Description

The rock is homogeneously with a grey to dark grey colour. The grain size of the rock is uniform and very fine to microcrystalline. With a 3 % salt acid the rock blusters weak. So the rock type can be classified as carbonate.

Microscopic Description

First of all the minerals which are the main components of this rock are listed and then the structure and texture of the rock is classified.

Minerals: the minerals in this thin section are carbonate minerals (dolomite, calcite) but without other methods it is not possible to differ clearly between these two minerals.

Structure: -Crystallinity: holocrystallin
-Granularity: microcrystalline, fine grained, uniform grained
-Crystal shape: hypidiomorph

Texture -Arrangement: hypidiomorph-grained, isometric
-Grain binding: direct

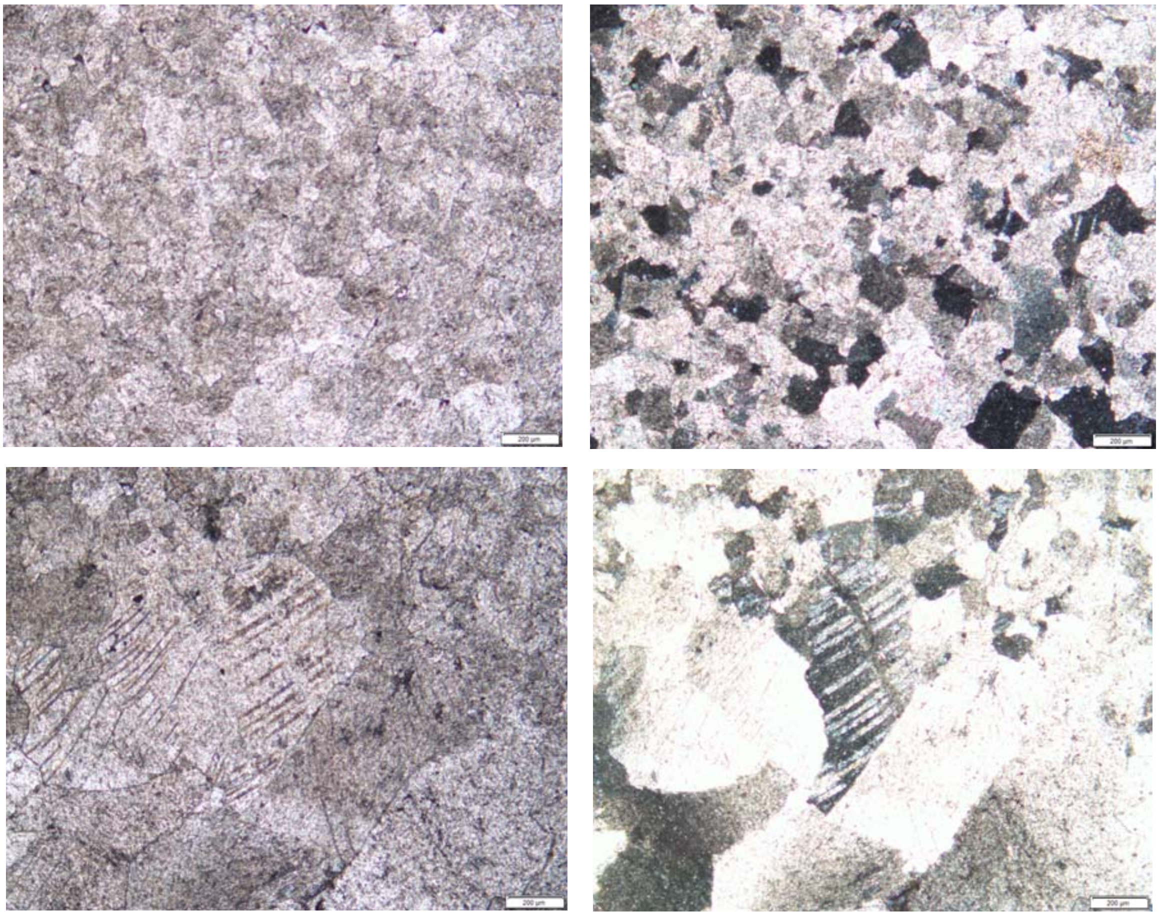

Fig. 8: Thin section blocks

X-ray powder diffractogram

To determine the rock type more accurate also an x-ray powder diffractogram has been done. According to this analysis the macroscopic and microscopic description can be confirmed. The rock type of the used blocks is carbonate with a composition of dolomite and calcite (Appendix A.1). No phyllosilicate can be found in the diffractogram analysis.

Unconfined compressive strength test

The UCS test (Appendix A.2) shows that the used blocks have an unconfined compressive strength in the order of $89400\ \frac{kN}{m^2}$.

<u>Coarse grained material</u>

The used coarse grained material is a 8/16 (Ø 8 mm to Ø 16 mm) mainly subrounded grain, which was already analysed in detail during a completed master project [19]. Within this master project [19] the grain size, grain shape, repose angle were analysed and also a shear test was done. In the following section the results of this investigation is summarised. The following figure and table show the grain size distribution, the coefficient of uniformity C_u and the curvature coefficient C_c.

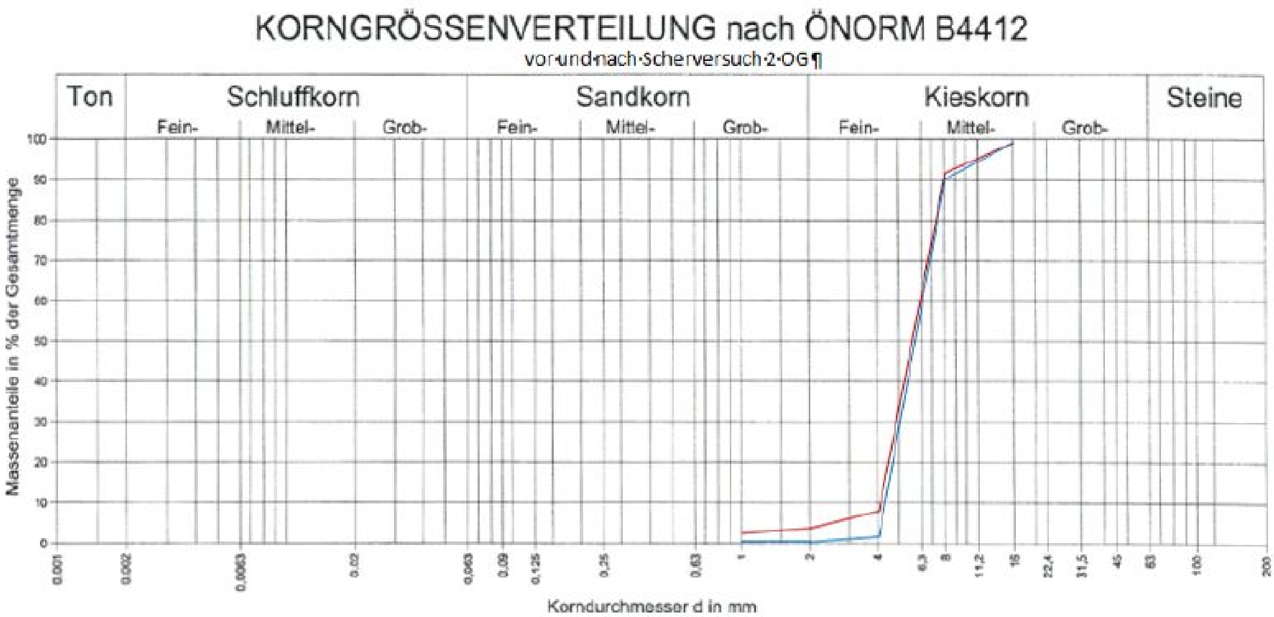

Fig. 9: Grain size distribution, coarse grained material [19]

size [mm]	Mass [%]
16	12,3
8	83,0
4	3,2
smaller	1,4
sieve losses	0,2

C_U	1,51
C_C	0,89

Tab. 5: Grain size distribution, coefficients, coarse grained material [19]

The coefficient of uniformity $C_u = 1{,}51$ and the curvature coefficient $C_c = 0{,}89$ classifies the coarse grained material as uniform soil with a poorly-graded grain size distribution.

Furthermore *Havinga [19]* used 150 grains to determine the shape of the grains according to the comparative table of *Rittenhouse* (Appendix A.3). The following table shows the classification of the grain shape.

class	grains
a. rounded	0
b. angular	26
c. subrounded	71
d. subangular	53

Tab. 6: Grain shape, coarse grained material [19]

The response angle of the coarse grained material is between $\alpha = 36° - 43°$ and the shear test shows a friction angle between $\varphi = 40{,}6° - 41{,}9°$ [19].

To complete the analysis of the coarse grained material a macroscopic description of the different rock types was done. In this material four different types of rocks can be found: gneiss, quartzite, greenstones and carbonate.

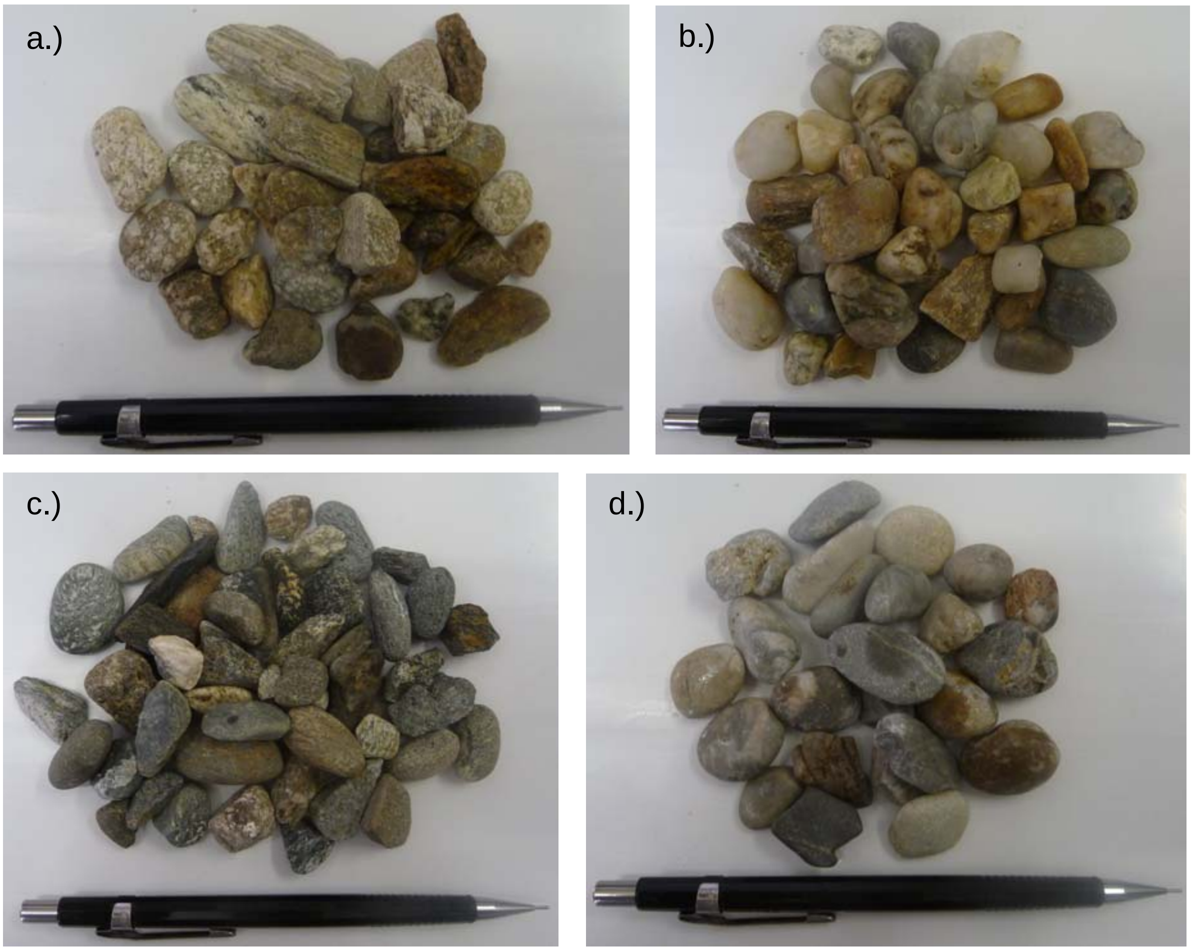

Fig. 10: a.) Gneiss, b.) Quartzite, c.) Greenstones, d.) Carbonate

Fine grained material

However to describe the used fine grained material a grain size analysis, a macroscopic description of the grain size higher than 4 mm, a x-ray powder diffractogram of the grain size smaller than 0,125 mm and a shear test was done.

The following figure and table show the grain size distribution, the coefficient of uniformity C_u and the curvature coefficient C_c.

Fig. 11: Fine grained material

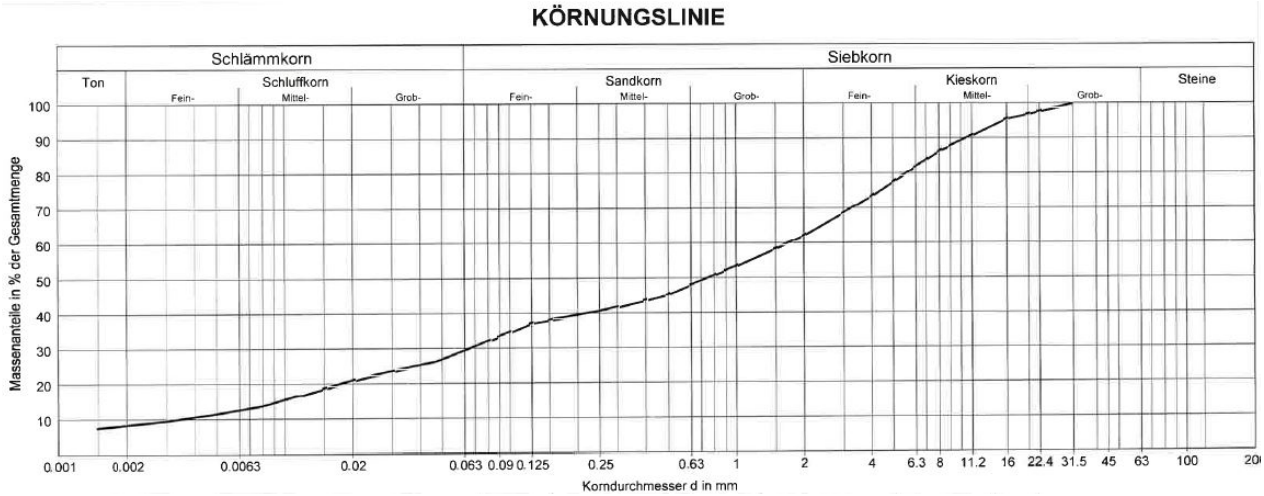

Fig. 12: Grain size distribution, fine grained material

size [mm]	mass [%]
31,5	4,1
16	9,3
8	12,9
4	11,4
2	8,6
1	8,1
0,5	4,8
0,25	3,2
0,125	7,9
0,063	8,5
0,02	12,7
0,002	8,5

C_U	501
C_C	0,79

Tab. 7: Grain size distribution, coefficients, fine grained material

The coefficient of uniformity $C_u = 501$ and the curvature coefficient $C_c = 0{,}79$ classifies the fine grained material as non-uniform soil with a well-graded grain size distribution. Depending on the grain size classification of clay there is a mass of 21,2 % (< 20µm) or 8,5 % (< 2µm) of clay in this fine grained material. The detailed results of the grain size analysis are shown in Appendix A.4.

Macroscopic Description

A description of the grains with a size higher than 4 mm was done. The colour of the rock is dark grey to brown with little shiny flakes and shows a foliation. Therefore the rock can be classified as mica-shist.

Fig. 13: Macroscopic description, fine grained material > 4 mm

X-ray powder diffractogram

The material with a grain size smaller than 0,125 mm was analysed with the X-ray powder diffractometry. The analysis shows that this material is a mixture of quartz, albite, muscovite and chlorite minerals, also pyllosilicates were found in this sample. To determine the different clay minerals exactly further tests are needed (Appendix A.5).

Shear test

For this test the fine grained material was sieved and only the material with a grain size smaller than 4 mm was used for the shear test. Otherwise it would not be possible to perform the shear test in a shear box with the dimensions of 10 cm x 10 cm x 2 cm (l x w x h).

The friction angle of this material determined with the shear test is $\varphi = 28{,}9°$ and the cohesion $c = 27{,}8\,\frac{kN}{m^2}$. Furthermore a residual shear angle $\varphi_r = 27{,}9°$ was determined. Moreover the detailed results are shown in Appendix A.6

3.1.4.2 Trials

Pull out trials with blocks above blocks, coarse grained material between the blocks and fine grained material between the blocks were carried out. The used blocks have approximately dimensions of 0,7 m x 0,6 m x 0,5 m (l x w x h) and a mass of 280 kg - 550 kg.

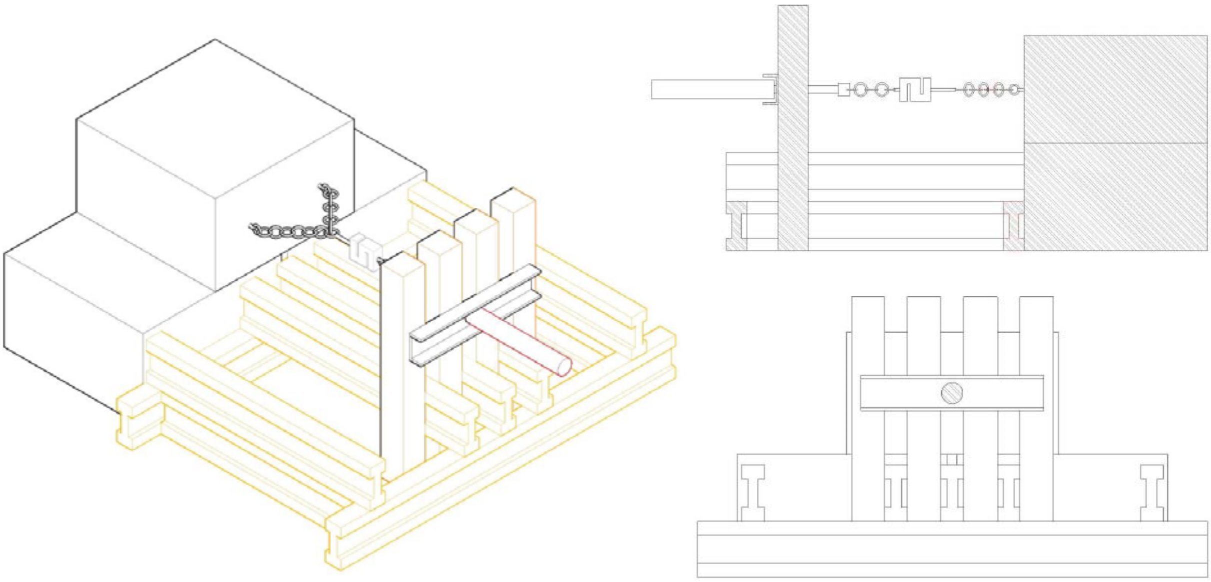

Fig. 14: Schematic sketch of pull out trial (*Havinga M.*)

The weight force G of all the five blocks was measured with a load cell. From the weight the mass of each block was calculated. Two blocks with the most planar surface were used as base. In front of them a pulling tool with a hydraulic jack was installed. Three different blocks A,B, C with different weight and shape were used for the trials. On each of the three blocks anchors with towing eyes were mounted. Each one of the three blocks is placed on the base and pull out trials with block above block, with block above coarse grained material above block and pull out trials with block above fine grained material above block were performed. Overall 33 pull out trials with the different blocks and different material were carried out. Between the towing eyes of the tested block and the pulling tool a load cell was mounted to measure the maximum shear force Z. The next pictures show the layout of the pull out trial and the different pull out trials.

Fig. 15: Layout of pull out trial

Fig. 16: Pull out trial, block above block, coarse grained material, fine grained material

Block A has a mass of 550 kg, a platy-cuboid regular shape and the used joint for the trials is planar. Block B has a mass of 549 kg, an irregular shape and the used joint for the trials is not planar. Block C is the lightest one with a mass of 340 kg, its shape is cuboid regular and the used joint for the trials is planar.
The two Blocks D and E are used as base, D has a mass of 283 kg and E has a mass of 338 kg, the shape of these blocks is regular and the used joint has a flat surface. Block A, B, C and E can be classified as HMB 300-1000 and block D as LMB 60-300.

Fig. 17: Shape of tested blocks A, B and C

To determine the material value m which means the relation between shear force Z and weight force G ("friction") in the joint between the single blocks equation 1 is needed. To keep it simple the cohesion is neglected.

equ. 1: $$Z = G * \tan m$$

In which G is the weight force and Z is the maximum shear force. G was measured during the weighing of the blocks and is the mass of blocks above the observed joint multiplied by gravity. Z is applied through pulling out the block above the observed joint with the pulling tool and measured with a load cell. Now the material value m in the joint between the tested blocks and the base (consists of two blocks which are lying side by side) can be calculated from equation 2.

equ. 2: $$m = \tan^{-1} \frac{Z}{G}$$

In the next three tables the results of the 27 pull out trials from the three test blocks A, B and C carried out on 25.03.2013 are presented.

		block above block		coarse grained, material		fine grained material	
Nb.	**G** [kN]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]
1	5,4	3,3	31,3	2,5	24,7	2,0	20,5
2	5,4	2,8	27,4	1,5	16,0	1,8	18,1
3	5,4	3,0	29,4	1,4	14,4	1,6	17,0
mean		3,0	**29,4**	1,8	**18,4**	1,8	**18,5**

Tab. 8: Result pull out trials of block A, 25.03.2013

		block above block		coarse grained, material		fine grained material	
Nb.	**G** [kN]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]
1	5,4	1,3	13,4	1,8	18,2	0,4	4,1
2	5,4	1,2	12,8	1,5	15,5	0,7	7,6
3	5,4	1,7	17,4	1,4	14,7	0,7	7,8
mean		1,4	**14,5**	1,6	**16,1**	0,6	**6,5**

Tab. 9: Result pull out trials of block B, 25.03.2013

		block above block		coarse grained, material		fine grained material	
Nb.	**G** [kN]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]
1	3,3	1,1	18,5	0,9	15,8	0,7	12,3
2	3,3	1,1	18,4	1,1	18,3	0,9	15,5
3	3,3	1,2	19,5	0,9	15,4	1,1	17,8
mean		1,1	**18,8**	1,0	**16,5**	0,9	**15,2**

Tab. 10: Result pull out trials of block C, 25.03.2013

The trials with block A, 25.03.2013 show a mean material value *m = 29,4°* for block above block, *m = 18,4°* for the coarse grained material between the blocks and *m = 18,5°* for the fine grained material between the blocks. Moreover the trials with block B, 25.03.2013 show a material value *m = 14,5°* for block above block, *m = 16,1°* for the coarse grained material between the blocks and *m = 6,5°* for the fine grained material between the blocks. Finally the trials with block C 25.03.2013 show a material value *m = 18,8°* for block above block, *m = 16,5°* for the coarse grained material between the blocks and *m = 15,2°* for the fine grained material between the blocks.

On the 10.04.2013 six more pull out trials with block C were performed. The results of these trials are presented in the next table.

		block above block		**fine grained, material**	
Nb.	**G** [kN]	**Z** [kN]	**m** [°]	**Z** [kN]	**m** [°]
1	3,3	1,7	26,4	1,0	17,1
2	3,3	1,5	24,4	1,0	16,5
3	3,3	1,5	24,7	1,0	16,7
mean		1,6	**25,1**	1,0	**16,8**

Tab. 11: Result pull out trials of block C, 10.04.2013

The trials with block C, 10.04.2013 show a mean material value *m = 25,1°* for block above block and *m = 16,8°* for the fine grained material between the blocks.

3.1.4.3 Interpretation and Discussion

The determined material value m in these tests is the relation between shear force Z and weight force G acting in the joint between the single blocks which has no fill (block above block) or is filled up with coarse grained or fine grained material. The whole material values determined with these trials are in general very low. Before the trials higher material values were expected.

The mean material value for block above block (A *m = 29,4°*, B *m = 14,5°*, C *m = 18,8°*) on 25.03.2013 and the mean material value (C *m = 25,1°*) on 10.04.2013 show that the placing of the blocks changes the material value. Furthermore if the tested block is placed a little bit backward on the blocks D and E the material value is higher. In the trials on 10.04.2013 block C is placed about 0,1 m backward and now the material value is higher than for block C on 25.03.2013.

The mean material value for coarse grained material between the blocks (A *m = 18,4°*, B *m = 16,1°*, C *m = 16,5°*, 25.03.2013) is much lower than the friction angle estimated for the coarse grain material itself ($\varphi = 40{,}6° - 41{,}9°$). A possible explanation for this phenomenon is that the grains are rotating and not sliding. To confirm this hypothesis pull out trials with very angular and coarse grained material should be done.

The mean material value for fine grained material between the blocks (A *m = 18,5°*, B *m = 6,5°*, C *m = 15,2°*, 25.03.2013) is also lower than the friction angle estimated for the material itself ($\varphi = 28{,}9°, c = 27{,}8\,\frac{kN}{m^2}$). A possibility is that the wet fine grained material was loaded to quickly and pore water pressure develop, which leads to this small material value. At the pull out trials on 10.04.2013 this possibility was disproved because the same fine grained material which has nearly 14 day’s time to dry was built in one day before the trials and the material value is not much higher (block C *m = 16,8°*, 10.04.2013). The very low value of block B can be explained that this block was not sliding. Because of its irregular shape he topples down and caused this low material value.

The variation of the values among themselves is maybe a result that the block is not placed on the same location every time. It is also possible that the material values for

all types of material are caused by the velocity (average 150 mm pro minute) of the pull out trials. Furthermore the velocity is not constant because of the use of a hydraulic jack which is pressure controlled.
For additional pull out trials the block should be placed every time on the same place (controlled via e.g. laser pointer), the velocity should be slower and also a path controlled pulling tool should be used.

With these pull out trials it is not possible to derive a friction angle for a whole Natural Rock Wall because in reality the toothing between the single blocks plays an essential role. However a single block in a Natural Rock Wall is fixed and has not the possibility to move in any direction. Furthermore no continuous sliding joint will occur.

In general qualitative it can be said that that the material value for block above block is the highest if the used blocks have a regular shape and the joints are very planar. This is most common in Natural Rock Walls (Steinsatz) were the blocks are machined. For irregular blocks with non-planar joints like block B the material value for block above block is lower than the material value for the coarse grained material between the blocks. That means the coarse grained material between the blocks leads to an extensive bearing and to a higher material value. The fine grained material between the blocks has the lowest material value. Anyway it should be avoided to use fine grained material in Natural Rock Walls.

Furthermore the pull out trials show that the placing of the blocks and the inclination of the wall plays an essential role for the stability of a Natural Rock Wall.

The pull out trials confirm that the shape of the used blocks, the chinking material and the placing of the blocks are the essential factors for the stability of a Natural Rock Wall. The trials show that if the blocks are very regular with planar joints no chinking should be used, but if the blocks are very irregular (like in most of the cases) then angular coarse grained material should be used for chinking.

3.1.5 Backdrain/Backfill

Behind the Natural Rock Wall a backdrain has to be installed. The backdrain has the function as drainage for the slope water. It assures that no pore water pressure can build up behind the Natural Rock Wall and also helps to reduce the overall soil pressure [4].

The width of the backdrain should be at least 0,3 m and freeze-thaw durable, coarse grained, angular, crushed rock material without fine grain fraction should be used. On the lowest point behind the Natural Rock Wall, a perforated drain pipe (e.g. DN 150) should be situated over the whole length to drain the collected water off. In the case of mortared Natural Rock Walls additionally drainage drillings through the wall are needed to ensure no water pressure will build up behind the wall.

Another important issue is that the backdrain must be separated from the surrounding backfill or native soil by a geotextile. The main function of the geotextile is to protect the surrounding backfill, soil against piping. Piping is a process through which fine grained soil particles are transported from the soil medium into the voids of the backdrain by water flow. The result of piping can be loss of ground, ground surface settlement or ground instability [4]. Also the backdrain will lose the function of draining water and a water pressure will be built up behind the Natural Rock Wall.

The cover layer of the backdrain should be impermeable, to prevent additional surface water (rain, surface run off) from getting behind the Natural Rock Wall into the backdrain.

Backfill is the area between the back cut (soil) and the backdrain. Any material (free of trash, organics) can be used for the backfill. In most of the cases the excavated material is used for this application. But if a material with a very low friction angle is used the dimension of the wall must be bigger to resist the driving forces. Be careful if the backfill is compacted after the construction of the Natural Rock Wall, then a compaction earth pressure will occur additionally. In fill conditions the engineered fill is typically placed and compacted before the Natural Rock Wall construction [4].

3.2. Construction

In this chapter the different construction types with their disadvantages and advantages are discussed. Furthermore the usual geometries of Natural Rock Walls are pointed out. Moreover a short summary of a construction cycle for Natural Rock Walls and some examples for standard sections are given.

3.2.1 Construction Types

In general four construction types of Natural Rock Walls can be differentiated. They can be distinguished by their number of rows (single row or multi row) and if they are dry stacked or mortared.

The different construction types have different applications, not every type can be used in every situation.

<u>Single row / multi row</u>

A single row Natural Rock Wall means that only one block is stacked above another block. The width of such wall is limited by the size of the used blocks. Single row Natural Rock Walls can only be used for protection of erosion and Natural Rock Walls with a very low height. In the case that a Natural Rock Wall is used for protection of erosion the native soil respectively the fill behind the wall must be stable even without the wall. No additional loads can be carried by such a Natural Rock Wall, only the single blocks are carrying themselves.

In contrast to a single row Natural Rock Wall, the multiple row Natural Rock Wall consists of at least two blocks per row. This means the width of such walls is not restricted. Because of the unlimited width and the hence resulting in a much higher weight of the wall, these Natural Rock Walls are used in the function as retaining walls. They are able to carry additional loads, such as, native soils, fills, traffic loads, etc.

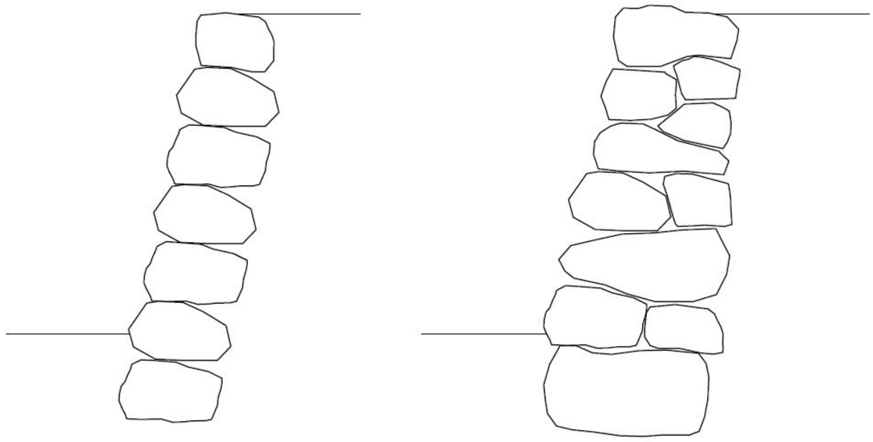

Fig. 18: Schematic sketch of single row and multi row, Natural Rock Wall

Fig. 19: Multi row, Natural Rock Wall [6]

Dry stacked / mortared

The blocks are stacked above each other and the joints between the blocks of a dry stacked Natural Rock Wall are filled with coarse grained, angular material. In such type of wall the shape of the blocks plays a more essential role. The combination of coarse grained, angular material and the ideal shape of the blocks lead to an extensive bearing. This type of wall also provides a good drainage of water from behind the wall.

The friction between the blocks depends on the used coarse grained, angular material for chinking or if no chinking is used it depends on the shape of the single blocks. The friction angle of a Natural Rock Wall has an order of $\varphi = 40° - 45°$ this means a $\mu = 0{,}8 - 1{,}0$ plus the “toothing friction”. In this case cohesion should not be considered. The unit weight of a dry stacked Natural Rock Wall has a magnitude of $\gamma = 20\frac{kN}{m^3} - 22\frac{kN}{m^3}$ if the assumed voids are 20 %.

As distinguished from dry stacked Natural Rock Walls the joints of mortared Natural Rock Walls are filled with mortar. In this case the shape of the blocks doesn't matter, because nearly all voids are filled with mortar, which leads to an extensive bearing. In this case drainage drillings are needed to get the water behind the Natural Rock Wall away.

The friction between the blocks is much higher than within dry stacked walls. Beneath the friction there is also cohesion acting. The range of friction and cohesion depends on the used type of mortar and has for mortared Natural Rock Walls general a magnitude of $\varphi = 40° - 45°$ and $c = 100\frac{kN}{m^2} - 500\frac{kN}{m^2}$. If the coefficient of friction is needed for calculation the alternative friction angle φ_A must be determined.

equu. 3: $$\mu_A = \tan \varphi_A$$

The alternative friction angle is calculated from the effective φ, c based on the current stress condition. The unit weight of a mortared Natural Rock Wall has a magnitude of $\gamma = 23\frac{kN}{m^3} - 25\frac{kN}{m^3}$ if the assumed voids are smaller than 10 %.

Fig. 20: Stacked and mortared, Natural Rock Wall

Also a combination of mortared and dry stacked Natural Rock Wall is used. In such case the lower part, to the point of a certain height, is mortared and the rest is dry

stacked. The mortared height depends on the horizontal distance of the loads (e.g. traffic loads) to the Natural Rock Wall.
Depending on the loads and height of Natural Rock Walls they will be constructed dry stacked, mortared or a combination of both. Higher loads or bigger heights always lead to a mortared construction.

To sum up it can be said that a Natural Rock Wall, which is used as retaining wall, must have multiple blocks per row and can be constructed dry stacked, mortared or a combination of both.
According to *Marte* [2] “A single row Natural Rock Wall is not a Natural Rock Wall”. Only Natural Rock Walls with a very low height can be constructed as single row Natural Rock Wall.

3.2.2 Geometry

In the literature there are different suggestions about the maximum inclination of the face and back of Natural Rock Walls. In some cases also the height of the Natural Rock Wall is limited. To have a quick overview about the different suggestions, they are summarized in table 8. All this data relates to dry stacked Natural Rock Walls.

	max. height [m]	**max. face inclination V:H; [°]**	**max. back inclination V:H; [°]**
Gates & Fisher [1]	3,5	4:1; 76°	4:1; 76°
Marte [2]	-	3,8:1; 75°	12:1; 85°
Gray & Sotir [4]	3,0	3:1; 72°	-
Gifford & Kirkland [4]	4,6	4:1; 76°	-
FHWA [4]	-	4:1; 76°	-
ARC [5]	-	4:1 - 6:1; 76° - 81°	-
ÖBB [15, 16, 17]	8	2,5:1; 68°	5:1; 79°

Tab. 12: Geometries of dry stacked Natural Rock Walls

The maximum inclination of the face should be 4:1 (76°) and less than vertical (90°) for the back of a dry stacked Natural Rock Wall. For a mortared Natural Rock Wall the maximum inclination of the face and back should be less than vertical (90°). The thickness of the cross section of the wall depends on its height, the retained soil and the additional loads.

Natural Rock Walls are theoretically not limited in their height, but at a certain height the width of the cross section will be too thick for economic construction. It is a task of the calculations to fit the smallest possible width and the largest possible height out.

3.2.3 Construction Cycle

The following passage gives a short summary about the construction cycle of a Natural Rock Wall. Because of the dimension of the blocks and the faster construction an excavator is needed.

- First of all the site, where a Natural Rock Wall is decided to be build, must be investigated. At least some trial pits are needed to know the properties of the retained soil and the properties of the underground which is essential for the design and calculation of a Natural Rock Wall.
- After the calculation and design the construction can begin. Now the cut for the wall is done. The back cut angle is depending on the soil parameters. In some cases the cut must be done in sections (also the construction) or in the case of bad ground condition a temporary pit supporting system (e.g. soil nailing) is needed.
- After the cut the different types of foundation, depending on the soil properties, construction loads and additional loads are constructed.
- Now the first row of blocks is placed. Behind the blocks the geotextile with the drainage pipe and the backdrain is situated. Also the backfill is fitted between the geotextile and the back cut. The trench in front of the first blocks is filled and compacted until the designed surface.
- The chinking between the blocks (coarse grained, angular material or mortar depending on the design) is installed. The next row of blocks is placed and the geotextile with the backdrain and the backfill are situated behind the blocks.
- The same construction cycle will be repeated as often as the design height of the Natural Rock Wall is reached.
- Behind the top of the Natural Rock Wall a layer of impermeable soil is built in.
- In the case of a mortared Natural Rock Wall drainage drillings are installed at the end of the construction or drainage pipes are installed within the wall during the construction.

3.2.4 Standard Sections

For the construction of a Natural Rock Wall after the design a standard section should be provided. It will be very useful for the contractor and also for the owner of the building to control the work of the contractor.

In Appendix B the standard section of this master thesis, the ÖBB [16, 17] and the FHWA [4] are shown.

The geometry of the Natural Rock Wall in the standard section of the ÖBB [16, 17] is fixed and not adjustable. Also the quality of the used blocks is specified. Beneath the geometry there are values for the calculation of a Natural Rock Wall provided.

The standard section of the FHWA [4] is adjustable, depending on the calculation result different geometries can be filled out. Furthermore some examples of improper construction are pictured. This standard section shows only single row Natural Rock Walls.

The standard sections of the ÖBB and FHWA illustrate dry stacked Natural Rock Walls, mortared Natural Rock Walls are not considered.

In the designed standard section of this master thesis everything is adjustable. Beginning from the geometry, over the type of backdrain, until the type of foundation everything is customizable. Furthermore the type of the Natural Rock Wall (dry stacked, mortared or a combination of both) is considered. As a result of the site conditions and the calculations the most economical design can be used and pictured in the standard section.

3.3. Calculation

In this chapter the failure mechanisms, verification of stability, models, software, comparative calculations and the significance of calculations of Natural Rock Walls are discussed.

Natural Rock Walls which are used as retaining walls, in generally are considered as gravity walls [h]. That means the high mass of the wall is resisting the driving forces. The resulting force of all driving forces (e.g. earth pressure, self-weight, loads, etc.) will be applied through the base of the wall into the underground [3]. Furthermore the resulting force must be within the first core-width [7].

3.3.1 Failure Mechanisms

For the calculation of Natural Rock Walls the different possible failure mechanisms should be discussed in first place. In general failure mechanisms can be distinguished in external and internal failures. This classification of external and internal failure mechanism originates from the reinforced concrete walls.

External failure means that the failure occurs from outside. In the case of a reinforced concrete wall the structure (wall) will not be destroyed but in the case of a Natural Rock Wall the external failure leads to an internal failure and the whole structure (wall) will be destroyed.

Internal failure means that the failure occurs within the wall. In both cases, reinforced concrete wall or Natural Rock Wall the whole structure (wall) will be destroyed.

In the following table the external and internal failure mechanisms are summarised.

External failure	Internal failure
Bearing Capacity	Slope failure through rock wall
Slope Failure	Toppling of single blocks
Toppling	Sliding of single blocks
Sliding	Material failure
Differential settlements (ground)	Bulging
	Differential settlements (wall)

Tab. 13: External and internal failure mechanisms

<u>External failure</u>

The external failure mechanisms like bearing capacity, slope failure, toppling, sliding and differential settlements (ground) are based on failures outside the Natural Rock Wall. In figure 21, 22 and 23 these failure mechanisms are illustrated.

Bearing capacity occurs if the shear strength of the soil is exceeded. The soil on the base of the Natural Rock Wall is displaced and the wall can subside or tilt.

If the loads at the slope are too high, then the shear strength of the soil is exceeded and a slope failure will occur.

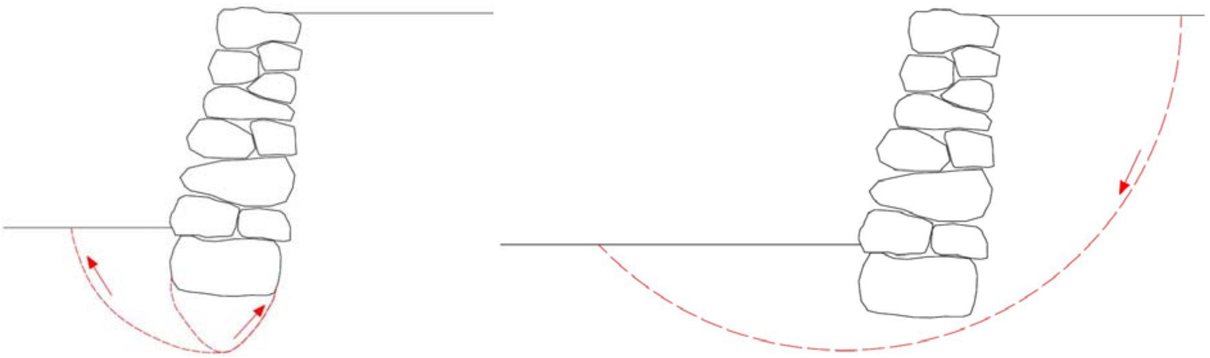

Fig. 21: Bearing capacity, slope failure

Toppling will occur if the driving moments at the base of the Natural Rock Wall are higher than the resisting moments.

Also sliding will occur if the driving forces at the base of the wall are higher than the resisting forces (e.g. friction, self-weight, etc.).

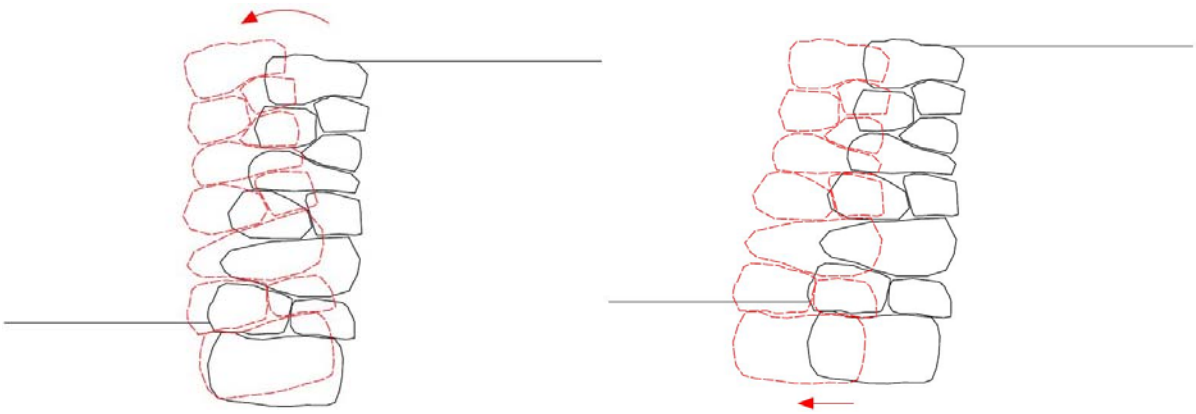

Fig. 22: Toppling, sliding

Differential settlements (ground) can occur if the ground is not stable over the whole length of the wall.

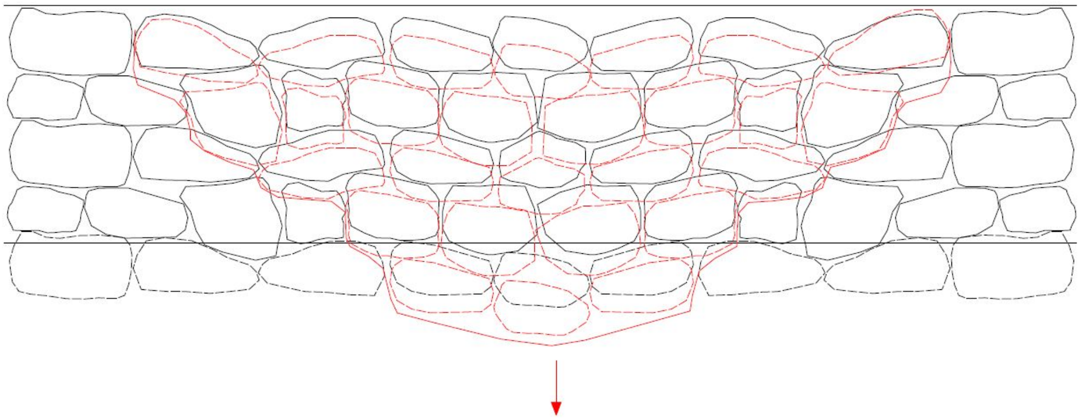

Fig. 23: Differential settlements (ground), front view

<u>Internal failure</u>

The internal failure mechanisms like slope failure through rock wall, toppling of single blocks, sliding of single blocks, material failure, bulging and differential settlements (wall) are based on failures within the Natural Rock Wall. In figure 24, 25, 26 and 27 these failure mechanisms are illustrated.

A slope failure through the Natural Rock Wall will occur if the loads behind the wall are too high. The failure will go through the weakest point (joint between the single blocks) and displace the part of the wall. Toppling of single blocks will occur if the driving moments at the front of a joint are higher than the resisting moments. The part of the wall above this junction point will topple.

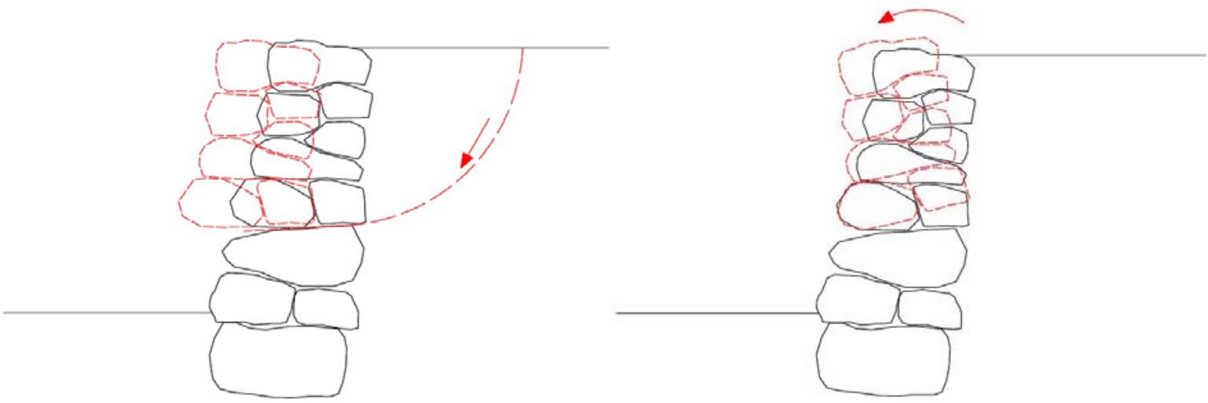

Fig. 24: Slope failure through rock wall, toppling of single blocks

Sliding of single blocks will occur if the driving forces in a joint are higher than the resisting forces. One block or even the whole part of the wall above the joint will slide. A material failure can occur if the blocks are composed of weak rock. The blocks will be destroyed due to the self-weight of the above lying blocks or because of the weathering.

Fig. 25: Sliding of single blocks, material failure

Bulging will occur if the loads behind the wall are too high. First the wall will bulge and after a certain time toppling is possible.

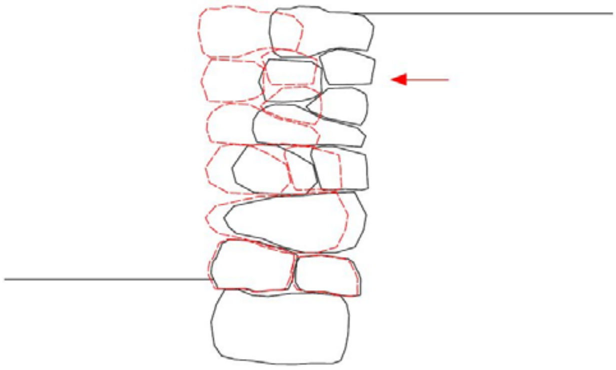

Fig. 26: Bulging

Differential settlements (wall) can occur if the blocks are not stacked very narrow (too much voids) or the ground is not stable over the whole length of the wall.

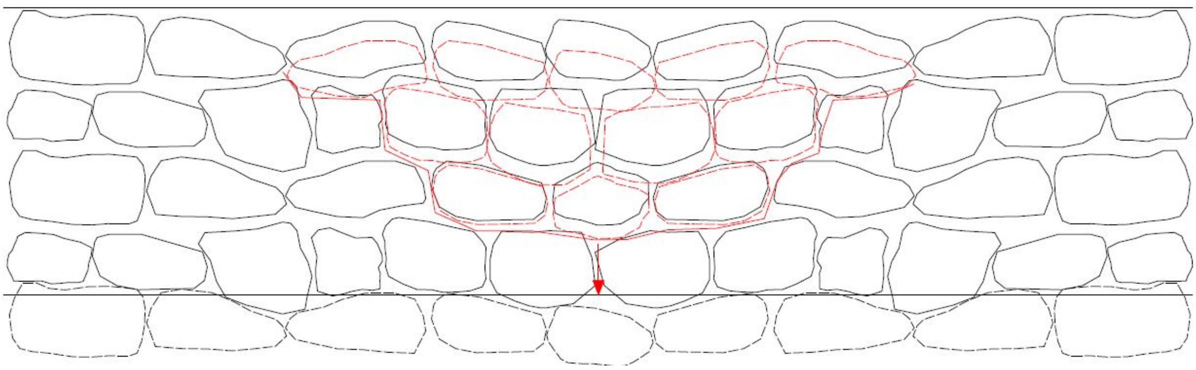

Fig. 27: Differential settlements (wall), front view

3.3.2 Verification of Stability

Natural Rock Walls are according to Eurocode 7 [a] retaining structures. Before designing, each structure has to be classified to a geotechnical category (GK) from 1 to 3.

GK 1: small and simple structures with negligible risk and comparable local experience. Design of structures after routinely procedure is possible [a]. Verification of structural stability without calculations [18].
GK 2: conventional structures without special risk or complicated site- and load-conditions. E.g. raft foundations, pile foundations, excavation pit, retaining structures, bridge pillar and abutment, embankment, etc. Geotechnical parameters and calculations are needed. Furthermore routine in-situ- and laboratory-tests for the design and construction are needed [a].
GK 3: all structures that are not belonging to GK 1 and 2. E.g. big and unusual structures, structures with extraordinary risk and special site- and load-conditions, structures at seismic sites, etc. [a]. For this category Eurocode 7 gives only a few recommendations [18].

Retaining structures with a height smaller than 2 m can be assigned GK 1, every retaining structure higher than 2°m must be assigned GK 2 [18]. That means a Natural Rock Wall with a height of more than 2 m is classified as GK 2.

Furthermore for design and geotechnical calculation the structure has to be assigned to a design situation and consequence class [b]. The design situation and consequence class is subdivided in three different situations/classes (BS 1 to BS 3 and CC 1 to CC 3).

Design situation BS 1: permanent design situation, situations for common terms of use like permanent loads, constant net loads and traffic loads, snow, groundwater, wind [b].
Design situation BS 2: temporary design situation during construction or repair, not regular traffic loads, freeze pressure [b].
Design situation BS 3: extra ordinary design situation like earthquakes, fire, explosion, impact, extreme groundwater or flood water [b].

Consequence class CC 1: no danger for human lives, low economic consequences e.g. circumstantial buildings, slope stabilisations at circumstantial traffic ways [b].
Consequence class CC 2: danger for human lives, economic consequences e.g. slope stabilisation at traffic ways, flood detention reservoir [b].
Consequence class CC 3: danger for many human lives, series economic consequences e.g. dam, public infrastructure buildings with paramount importance [b].
Depending on the design situation and consequence class different partial safety factors are used for geotechnical calculation. In figure 27 situations of CC 1 (no danger for human lives, low economic consequences) and CC 2 (danger for human lives, economic consequences) for Natural Rock Walls are shown.

Fig. 28: Example for consequence class CC 1 and CC 2 [6]

In general the structural stability (STR), total stability (GEO), uplift (UPL), piping (HYD) and static equilibrium (EQU) of the structure have to be proven for the design and calculation. For the design and calculation of Natural Rock Walls the verification of structural stability (STR), total stability (GEO) and static equilibrium (EQU) are the most important ones.
Moreover for all types of stability the ultimate limit state (ULS) and the serviceability limit state (SLS) have to be verified. The verification of ultimate limit state are for example bearing capacity, slope failure, sliding and toppling, the verification of serviceability limit state (SLS) are for example bulging, settlement and displacement.

<u>Structural stability (STR)</u>

This stability consist the verification of bearing capacity, sliding of the whole structure, sliding of single blocks and demands on the single blocks

To verify this stability verification procedure 2 has to be used. That means the partial factors of safety are applied on the demands (stress, force) as result of the loads and resistance of the ground [a]. In Appendix C (Calculation) the partial factors of safety for the demands (γ_E), soil parameters (γ_M) and the resistance of retaining structures (γ_R) for the ultimate limit state (ULS) are summarised.

For calculation of the serviceability limit state (SLS) all partial factors of safety are 1,00.

<u>Total Stability (GEO)</u>

The total stability consists the verification of slope failure and settlement analysis.

To verify this stability verification procedure 3 has to be used. That means the partial factors of safety are applied on the loads and the soil parameters [a]. In Appendix C (Calculation) the partial factors of safety for the loads (γ_F), soil parameters (γ_M) and the resistance of slope stability verification (γ_R) for the ultimate limit state (ULS) are summarised.

For the verification of the serviceability limit state (SLS) numerical analysis, method of observation or limiting of mobilised shear strength are recommended [b].

<u>Static equilibrium (EQU)</u>

The static equilibrium consists the verification of toppling of the whole structure and toppling of single blocks.

In Appendix C (Calculation) the partial factors of safety for the loads (γ_F) and soil parameters (γ_M) are summarised.

Furthermore the code of practice [h] is dividing the stability in external stability and internal stability. To external stability belongs the verification of toppling, sliding, bearing capacity, slope failure and settlements. However to internal stability belongs the verification of toppling of single blocks, sliding of single blocks and demands on the single blocks.

In general Natural Rock Walls are very flexible and have not so high requirements for settlements than other structures. Settlement analysis should be done if the construction ground is very sensitive for higher settlements.
In most cases the verification of demands on single blocks may be waived if blocks according to the standard of armour stones are used, because the uniaxial compressions strength of the blocks is much higher than the forces, stresses which can appear.

To sum up the whole section verification of stability, before designing a Natural Rock Wall it has to be classified to a geotechnical category (GK 1 to GK 3). Depending on the situation, for the calculation, the structure has to be assigned to a certain design situation (BS 1 to BS 3) and consequence class (CC 1 to CC 3). After this procedure the structural stability (STR), total stability (GEO) and static equilibrium (EQU) of the structure have to be proven. Moreover for all types of stability the ultimate limit state (ULS) and the serviceability limit state (SLS) have to be verified.

3.3.3 Models

Natural Rock Walls can be abstracted through three different models. These three models are following the principle from simple to complex. In figure 28 the different models are illustrated.

<u>Single-body-model</u>

The whole Natural Rock Wall is considered as a rigid single body like a reinforced concrete wall. The single-body-model can be used for the calculation of external failures like bearing capacity, slope failure, toppling and sliding. It cannot be used for the calculation of internal failures.

<u>Idealised multiple-body-model</u>

The Natural Rock Wall is separated in idealised blocks with a defined length and width. This model can be used for the calculation of all kinds of external and internal failures.

<u>Multiple-body-model</u>

At this model the Natural Rock Wall is also separated in blocks, but each single block has a different geometry. This model is closest to reality and can also be used for the calculation of external and internal failures.

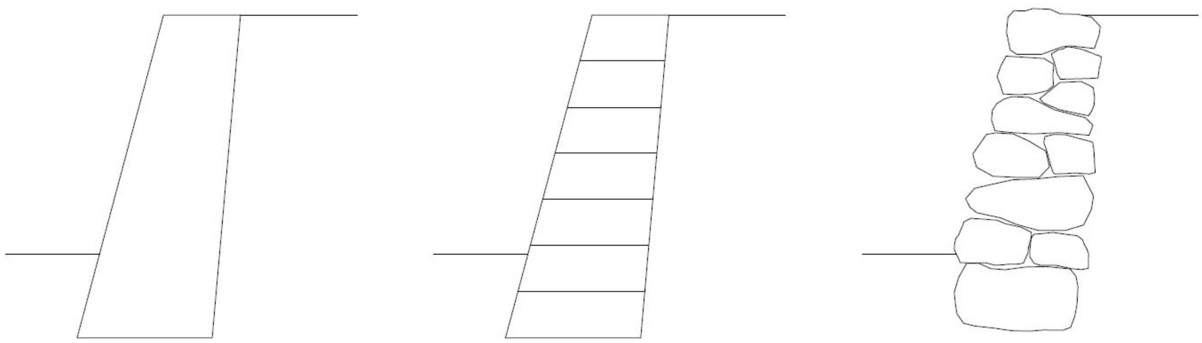

Fig. 29: Single-body-model, idealised multiple-body-model, multiple-body-model

In practice only the single-body-model and the idealised multiple-body-model are used. The multiple-body-model is not often used because it is too complicated and time consuming.

3.3.4 Software

Nowadays most of the stability analyses are carried out with computer software. In general there are two different types of analysis, the limit equilibrium analysis and the numerical analysis.

<u>Limit equilibrium analysis</u>

All limit equilibrium analyses are restricted to predefined failure modes and assume that failure occurs along the failure surface. Furthermore the limit equilibrium analysis is a comparison of forces (limit equilibrium between resisting forces and driving forces is calculated) for a particular failure mode [11].

<u>Numerical analysis</u>

For the numerical analysis all material properties and the behaviour of the materials (constitutive laws) have to be known. On the basis of these properties the failure mechanism can be modelled. Numerical analyses are particularly applied for complex failure mechanism, which are composed of more than two failure mechanisms. Furthermore the numerical analysis can be distinguished in continuum modelling, discontinuum modelling and hybrid/coupled modelling.

In practice limit equilibrium analyses are more often used for stability considerations of Natural Rock Walls than numerical analyses. Although for more complicated problems, limit equilibrium analyses are not appropriate. The analysis of the stability of a Natural Rock Wall is often difficult because geometries of the blocks and parameters are hardly known [11].
In the case of numerical analysis for Natural Rock Walls the material behaviour of the backfill can be described as continuum and the single blocks can be described as discontinuum [11].

In this master thesis the main focus lies on the limit equilibrium analysis and for this application the software GGU Gabion and GGU Stability (Civilserve GmbH) is used. Further software for the calculation of Natural Rock Walls is RIBTEC RTgabion (RIB Software AG) and DC Gabion (DC Software Doster & Christmann GmbH). Currently available software only allows too create idealised blocks. It is not possible to create blocks in any shape, because the main application of such software is the calculation of gabions.

3.3.5 Calculation Approaches

Three different calculation methods form Germany, the United States and Spain based on different approaches are reviewed and compared.

<u>Code of practice ”Stützkonstruktionen aus Betonelementen, Blockschlichtungen und Gabionen” [h]</u>

The calculations in the code of practice from the year 2003 are based on global safety factors. All calculations are carried out with the program GGU Gabion and GGU Stability. With the newest version of this software calculations according to the Eurocode 7 (partial safety factors) are available.

The size of the blocks is variable adjustable but the shape of the blocks is like the idealised multiple-body-model. Different modes of active E_A and passive E_P earth pressures are adjustable. The external stability (toppling, sliding, bearing capacity, slope failure, settlements) and the internal stability (toppling of single blocks, sliding of single blocks, demands on the single blocks) are analysed. Furthermore each single block is analysed.

<u>FHWA “Rockery Design and Construction Guidline” [4]</u>

The calculations are based on the factor of safety. The active earth pressure E_A is calculated after coulombs method and the passive earth pressure E_P is calculated after rankines method. The external stability (toppling, sliding, bearing capacity, slope failure) and the internal stability (toppling of single blocks, sliding of single blocks) are analysed. Furthermore a seismic analysis is done. For analysis the structure is separated in simple geometries, not every single block is analysed.

<u>Alejano “Stability of granite drystone masonry retaining walls” [13,14]</u>

Also this approach is based on the factor of safety. Moreover the approach is based on granite drystone masonry retaining walls from Spain where each block is machined and has a nearly cubic shape. The active earth pressure E_A is calculated after coulombs method and the passive earth pressure E_P is calculated after the by Berry & Reid modified Caquot & Kerisel method. It is possible to define the factor of safety and then for each row the block width is calculated, also the opposite way around where the block width for each row is defined and the factor of safety is calculated is possible. The external stability (toppling, sliding) and the internal stability (toppling of single blocks, sliding of single blocks) are analysed.

According to the valid standards in Austria and the state of the art the method after the code of practice [h] is the most adjustable and practical one. Furthermore all different verifications of stability can be calculated. A minor disadvantage is that only idealised shaped blocks can be created.

3.3.6 Comparative Calculations

The best model respectively the best calculation approach does not make sense if the used parameters for the calculation are not correct. To get an idea how parameters are effecting the calculation result (utilisation factors) comparative calculations with different parameters for one type of Natural Rock Wall were performed.

For the calculations of the structural stability, total stability (settlements) and static equilibrium the program GGU Gabion (Version 5.13, 21.01.2013) and for the calculation of the total stability (slope failure) the program GGU Stability (Version 10.40, 01.02.2013) based on the limit equilibrium analysis is used.

However for the comparative calculation the partial safety factors according to Eucocode 7 [b] for design situation BS 1 and consequence class CC 1 are used. Furthermore the earth pressures are calculated after DIN 5085:2011.

Comparative calculations among three different conditions were performed. In the comparative calculation A the active E_A and passive earth pressure E_P is applied, in comparative calculation B full active earth pressure E_A and passive earth pressure E_P limited to 0,001kN/m² (software doesn't allow $E_P = 0$) and in comparative calculations C full earth pressure at rest E_0 and passive earth pressure E_P limited to 0,001kN/m² is applied. For all comparative calculation the Natural Rock Wall has the following geometry (figure 22), five blocks with a dimension of 0,5 m x 1,0 m, a height of 2 m, a depth of 0,5 m and an inclination of 75°. The terrain above and below the Natural

Rock Wall is horizontal. No additional loads are applied and the ground consists of sand with the following soil parameters $\gamma = 20\frac{kN}{m^3}$, $\varphi = 30°$, $c = 0\frac{kN}{m^3}$.

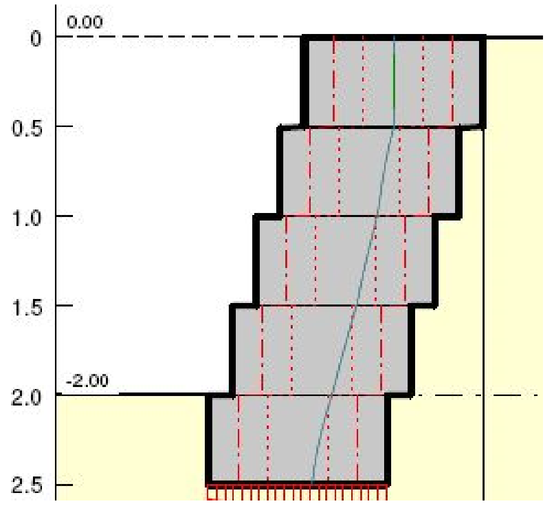

Fig. 30: Geometry of Natural Rock Wall for comparative calculations (GGU Gabion)

The essential parameters for the calculation of Natural Rock Walls can be classified in soil parameters and block parameters. Soil parameters are the unit weight γ, friction angle φ and the cohesion c of the soil. The block parameters are the unit weight γ of the blocks including the voids and joints, block size, joint thickness, the coefficient of friction μ between the blocks and the wall friction angle δ. All these parameters are adjustable in the program GGU Gabion. The results of the calculations are the different utilisation factors $\eta\ (sliding\ foot)$, $\eta\ (sliding\ blocks)$, $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$.

The following results show how different block parameters effect the different utilisation factors within the comparative calculation A, B and C.

<u>Results of comparative calculation A</u>

- A higher coefficient of friction μ leads to a lower utilisation factor of $\eta\ (sliding\ blocks)$. All other utilisation factors remain constant.
- A higher value for the joint thickness leads to a higher utilisation factor of $\eta\ (sliding\ foot)$ and to a lower utilisation factor of $\eta\ (sliding\ blocks)$, $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$.

- A higher wall friction angle δ leads to a higher utilisation factor of $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$. Furthermore it leads to a lower utilisation factor of $\eta\ (sliding\ foot)$ and $\eta\ (sliding\ blocks)$.
- A higher unit weight γ of the blocks leads to a higher utilisation factor of $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$. It also leads to a lower utilisation factor of $\eta\ (sliding\ foot)$ and $\eta\ (sliding\ blocks)$.

Results of comparative calculation B

- A higher coefficient of friction μ leads to a lower utilisation factor of $\eta\ (sliding\ blocks)$. All other utilisation factors remain constant.
- A higher value for the joint thickness leads to a higher utilisation factor of $\eta\ (sliding\ foot)$ and $\eta\ (sliding\ blocks)$. It also leads to a lower utilisation factor of $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$.
- A higher wall friction angle δ leads to a higher utilisation factor of $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$. Furthermore it leads to a lower utilisation factor of $\eta\ (sliding\ foot)$ and $\eta\ (sliding\ blocks)$.
- A higher unit weight γ of the blocks leads to a higher utilisation factor of $\eta\ (bearing\ capacity)$ and $\eta\ (EQU)$. However it also leads to a lower utilisation factor of $\eta\ (sliding\ foot)$ and $\eta\ (sliding\ blocks)$.

Results of comparative calculation C

- A higher coefficient of friction μ leads to a lower utilisation factor of $\eta\ (sliding\ blocks)$. All other utilisation factors remain constant.
- A higher value for the joint thickness leads to a higher utilisation factor of $\eta\ (sliding\ foot)$, $\eta\ (sliding\ blocks)$ and $\eta\ (bearing\ capacity)$. It also leads to a lower utilisation factor of $\eta\ (EQU)$.
- A higher wall friction angle δ leads to a higher utilisation factor of $\eta\ (EQU)$ and to a lower utilisation factor of $\eta\ (sliding\ foot)$, $\eta\ (sliding\ blocks)$ and $\eta\ (bearing\ capacity)$.
- A higher unit weight γ of the blocks leads to a higher utilisation factor of $\eta\ (EQU)$ and to a lower utilisation factor of $\eta\ (sliding\ foot)$, $\eta\ (sliding\ blocks)$ and $\eta\ (bearing\ capacity)$.

Also the values of the different utilisation factors are increasing from comparative calculation A to C. This is a result of the different types of earth pressures which are used in the three diffetent comparative calculations.

The complete values, results of the three comparative calculations are listed in tables in appendix C.2 and C.3.

3.3.7 Significance of Calculation Models

In this master thesis the limit equilibrium analysis programs GGU Gabion and GGU Stability are used. The program GGU Gabion is used for the calculation of structural stability, total stability (settlements) and static equilibrium. Moreover the program GGU Stability is used for the calculation of the total stability (slope failure).

In the program GGU Gabion a section of the Natural Rock Wall is calculated. In this calculation the toothing between the single blocks at the sides are not considered. Furthermore only idealised shaped blocks can be created which is not in accordance with the reality.

In the program GGU Stability the whole Natural Rock Wall is described as a homogenous soil layer with fictive soil parameters. Moreover continuous shear joints are developing, but in reality a continuous shear joint will never occur.

Only with feasible and improved calculation parameters the calculation models have a sense. However it is not simple to find appropriate parameters for a Natural Rock Wall because it is really hard to determine them.

In spite of the disadvantages of the calculation models, programs and the difficult estimation of the calculation parameters limit equilibrium analysis is state of the art. Nevertheless these models are a strong simplification of the reality.

4. Claims

Different cases of damaged Natural Rock Walls from geotechnical consultants are collected and analysed to get an idea which causes of damage can occur and which are the most frequent ones. Furthermore the possible remedial action of a damaged Natural Rock Wall is discussed.

4.1. Analysis

Different Natural Rock Walls are analysed in respect to their construction and failure mechanism. In some of these cases a failure respectively damage already occurred. In all of the cases in first place the construction and the used material is described. After this description the possible failure mechanism and its damage is interpreted. Moreover possible counter measures are explained. In the worst case the complete Natural Rock Wall has to be rebuilt. In most of the cases it is really hard to say something about the backdrain, drainage and the width of the cross section, which are essential factors for the stability of Natural Rock Walls. Only such things which can be apparently seen are described and interpreted.

<u>Case 1</u>

The height of this Natural Rock Wall is approximately 3,0 m and the used blocks can be classified as light mass blocks LMB 60-300. The blocks used in this wall are consisting of two different rock types. The white blocks are carbonate rock and the brown ones are conglomerates. The shape of the blocks is polyhedral and round. For chinking between the blocks fine grained material was used. Between the single blocks bigger voids are occurring. Furthermore most of the vertical joints are continuous. The horizontal joints are not normal to the inclined plain and are dipping out of the wall. Apparently no backdrain is carried out behind this Natural Rock Wall.

Fig. 31: Case 1 [6]

First of all the used blocks in this Natural Rock Wall are to small, as minimum blocks of the category HMB 300-1000 should be used. The conglomerate blocks are not fullfilling the standard of armour stones [d], first evidence of weathering of the blocks can be seen, they are decomposing in their single components. As chinking fine grained material was used, this should be avoided. Instead of this material angular coarse grained material should be used. The blocks should be stacked in a dense stone bond and they should be arranged shifted, that means in such a way that no continous vertical joints through the wall are occuring. The horizontal joints should dip inside the wall. Furthermore water pressure will be build up behind the wall because no backdrain is installed.
This Natural Rock Wall can be classified in the failure category as material failure. Furthermore the construction of wall is unsufficient. The only way to restorate such a Natural Rock Wall is a complete reconstruction after the state of the art.

<u>Case 2</u>
The Natural Rock Wall in this case has a height of approximately 3,0 m. Blocks in the class of HMB 300-1000 have been used. The rock type of this blocks is carbonate and their shape is platy-cuboid. In some cases cracks at the blocks can be seen. There are big voids between the single blocks. Moreover some vertical joints are continous through the Natural Rock Wall.

Fig. 32: Case 2 [6]

It seems that the rock not only consists of carbonate, maybe there are also some clay minerals. Due to the freeze-thaw cycles the clay minerals swell and generate cracks in the rock which will disintegrate the blocks. The blocks should be investigated in

respect to their freeze and thaw durability. Moreover in this case no material for chinking or only very fine grained material which is already washed out were used. This leads to large voids between the single blocks which causes instability of the wall because the blocks touch each other only in certain points.
The failure mechanism in this Natural Rock Wall is material failure. Furthermore the used material for chinking schould be avoided. If the freeze-thaw durability show that the blocks are durable enough then the damaged blocks should be replaced. To close the big voids the whole chinking should be carried out with mortar (fill joints and voids). If the blocks have a insufficient frezze-thaw durability the whole Natural Rock Wall should be reconstructed.

Case 3

This Natural Rock Wall has a height of approximately 5,5 m. The used blocks are in the armour stone class of HMB 300-1000 and their rock type is gneiss. The shape of the used blocks is platy-cuboid. Between the single blocks there are partially smaller and also bigger voids. Fine grained material is used for chinking. In some zones also continous vertical joints are going through the Natural Rock Wall.

Fig. 33: Case 3 [6]

The fine grained material which was used for chinking is washed out and bigger voids are formed. In some places the single blocks are only through specific points in contact, not over the whole block surface. This setting can cause instabilities and it is possible that single blocks fall out.
Chinking with such a material should be avoided. The whole chinking should be carried out with mortar (fill joints and voids) to restorate this Natural Rock Wall.

Case 4

The Natural Rock Wall in this case has a height of approximately 4,0 m. The used blocks can be classified as HMB 1000-3000. The rocktype of the blocks is carbonate. The shape of the blocks is polyhedral and round. As chinking stones and fine grained material were used. Continous vertical joints through the Natural Rock Wall are occuring.

Fig. 34: Case 4 [6]

Because of the shape of these blocks bigger voids are occuring and the horizontal joints are not inclined perpendicular to the front view. However horizontal joints are dipping slightly outside the wall. The fine grained material which was used for chinking is washed out from erosion. This process formed voids between the single blocks. Due to this instabilities are possible, stones and in the worst case also bigger blocks can fall out.

The material used for chinking and also in some parts the stacking is not appropriate. In general such shape of blocks should be avoided in Natural Rock Walls without mortar as chinking. To stop erosion and make the wall safe the whole chinking should be carried out with mortar (fill joints and voids).

Case 5

The height of this Natural Rock Wall is approximately 6,5 m. The used blocks are in the category of HMB 300-1000 and their rocktype is carbonate. Moreover the shape of this blocks is platy to cubic. As chinking fine grained material was used. Also continous vertical joints through the whole Natural Rock Wall are occuring.

Fig. 35: Case 5 [6]

The fine grained material which was used for chinking is washed out by erosion and leads to big voids between the single blocks. The blocks are only through some certain points in contact with each other. Moreover the blocks are not arranged shifted an therefore continous vertical joints are occuring. Sometimes the horizontal joints are not perpendicular to the inclined plain and so they are dipping out of the wall. This setting can cause instabilities of this Natural Rock Wall.
Furthermore the used material for chinking and in some parts also the stacking of the wall is not correct. The easiest way to restorate this Natural Rock Wall is the use of mortar for chinking (fill joints and voids). Also drainage drillings should be carried out.

<u>Case 6</u>

The Natural Rock Wall in this case has a height of approximately 4,5 m. Most of the blocks are in the armour stone category of HMB 300-1000 and some single blocks are also in the category HMB 1000-3000. The rock type of all of these blocks is gneiss and their shape is platy. For the chinking fine grained material was used. In some parts the horizontal joints are not orientiated normal to the inclined plain, so they are dipping out of the wall.

Fig. 36: Case 6 [6]

The single blocks of this wall are stacked in a good way but the used fine grained material for chinking is partly washed out and forms bigger voids between the blocks. The bigger voids between the blocks causes that the single blocks are only through certain points in contact with each other and not over the whole area of the blocks. It is possible that some smaller blocks can fall out of the wall. Furthermore differential settlements over the whole length of the Natural Rock Wall could occur because of the erosion of the chinking. Usually that small settlements are not a problem but in that case they are a problem because on the top of the Natural Rock Wall a construction with a continous concrete beam is built. The differential settlements of the Natural Rock Wall will lead to cracks in the concrete beam and in the worst case to a tilting of the construction.
The used chinking in this Natural Rock Wall has to be avoided. The simplest way to stop erosion and as consequence the differential settlemtents (wall) of this Natural Rock Wall is the use of mortar for chinking. That means all joints and voids should be filled up with mortar. Futhermore to avoid water pressurre behind the Natural Rock Wall drainage drillings should be carried out.

Case 7

The Natural Rock Wall in this case has a height of approximately 4,5 m. Most of the used blocks can be classifed as HMB 300-1000 and some single blocks also as HMB 1000-3000. The rock type of the used blocks is carbonate and their shape is polyhedral and round. For chinking fine grained material and some stones were used. However through the whole wall continous vertical joints are occuring and the horizontal joints are dipping outside the wall. No backdrain behind the wall is installed.

Fig. 37: Case 7 [6]

All blocks are only placed on the slope, they are not stacked in a dense bond. The fine grained material which was used for chinking is washed out and big voids are resulting. The single blocks are only through specific points in contact with each other and have a point bearing. This Natural Rock Wall is very instable bigger blocks can fall out very easily. Furthermore the native soil behind the wall and the chinking material could be washed out easily because no backdrain is installed.

The construction of this Natural Rock Wall is complete incorrect. The only way to restorate such a Natural Rock Wall is a complete reconstruction after the state of the art.

Case 8

The Natural Rock Wall in this case has a height of approximately 4,0 m. The used blocks can be classified as HMB 300-1000. Moreover the rock type of these blocks is carbonate. Their shape is polyhedral and round. Fine grained material was used for chinking between the single blocks. Some of the vertical joints are continous through the whole wall. In the back of the Natural Rock Wall no backdrain was installed. On the top of the Natural Rock Wall an access road to a familiy home is situated. In the zone of the wall the access road has a vertical displacement of approximately 1,0 m.

Fig. 38: Case 8 [6]

Apparently the blocks of the Natural Rock Wall are only placed on the slope. The wall is not embedded into the soil and no foundation was built. Furthermore the blocks are not stacked in a dense bond and the fine grained material for chinking is washed out. Due to the missing backdrain it is also possible that native soil behind the wall and more chinking material could be washed out. A combination of all these factors leads to the failure of this Natural Rock Wall. A differential settling with a vertical displacement of nearly 1,0 m occurred.

The failure mechanism of this wall is differntial settlement (wall and ground). Furthermore it can be said that the construction is complete incorrect. This Natural Rock Wall has to be reconstructed completely according to the state of the art.

Case 9

The height of this Natural Rock Wall is approximately 3,0 m. Blocks of the category HMB 300-1000 were used and their rock type is greenstone. The shape of the used blocks is platy to cuboid. Fine grained material was used for the chinking between the single blocks. Continuous vertical joints are occurring through the whole wall. For this Natural Rock Wall no backdrain is installed. On the top of the wall an additional fill secured with masonry blocks with an approximately height of 1,4 m is carried out. Furthermore two little ponds are situated behind the whole construction.

Fig. 39:Case 9 [6]

Apparently the sealing foil of the little ponds was not carried out correct and has some cracks. So the water from the ponds seeped continuously into to the soil behind the masonry blocks and the Natural Rock Wall. During the time because of the missing backdrain native soil and chinking material gets washed out. A combination of changed load conditions due to the construction (masonry blocks, fill) above the wall, the fine grained material for chinking and the not in a dense bond stacked blocks led to this collapse of the Natural Rock Wall.

The failure mechanismn of this wall is toppling. This Natural Rock Wall has to be reconstructed completly according to the state of the art. However if additonal loads (masonry blocks, fill) are used again the Natural Rock Wall has to be completly new designed.

Case 10

The Natural Rock Wall in this case has a height of approximately 4,0 m. The used blocks are in the armour stone class of HMB 300-1000 and the shape of this blocks is platy to cuboid. For the chinking between the single blocks mortar was used. That means the whole Natural Rock Wall is mortared. Furthermore in front of the wall a road is situated.

Fig. 40: Case 10 [6]

Apparently no backdrain is provided behind the Natural Rock Wall. Furthermore a little slide has occurred in the upper section behind the wall. Due to the missing backdrain and the missing drainage drillings it is also possible that a water pressure was build up behind the mortared wall. However the slide behind the wall led to much higher earth pressures which were acting on the wall. A combination of the higher earth pressures and the possible water pressure induced the failure. The resisting forces (e.g. self-weight) of the wall were not able to resist the driving forces (earth pressure, water pressure) and the wall collapsed.

The big cracks and the lifting up of the asphalt in front of the Natural Rock Wall are leading to the conclusion that the first failure mechanism is sliding of the whole wall which was followed from the second failure mechanism toppling. In this case the complete Natural Rock Wall must be redesigned (for such additional loads) and reconstructed.

4.2. Frequent causes of Damage

The analysed cases of damage show the different mistakes which are made within the construction of a Natural Rock Wall. These mistakes are leading to failures and stability problems of the Natural Rock Walls.

One of the common mistake in the construction of Natural Rock Walls is the usage of non proper material. That means blocks with no durability against weathering, too small blocks, bad shape of the blocks and the wrong chinking between the single blocks causes problems.

Furthermore in most of the cases the blocks are stacked not adequate that means continous vertical joints and also horizontal joints which are dipping outside the wall are occuring and are leading to stability problems. This also means the whole wall is not stacked in a dense bond.

Another common faliure is the absent of a backdrain behind the Natural Rock Wall which alows to wash out native soil and chinking material very easily. In the case of a mortared Natural Rock Wall a missing backdrain alows to bulilt up water pressure behind the wall. Both factors can lead to stability problems.

Moreover in some of the cases no foundation was provided and the wall is not embedded into to soil. This situation leads to differential settling or even to a failure of the complete Natural Rock Wall.

However many of the Natural Rock Walls are built without a proper design and stability caluclations. This leads in many cases to a failure especially if the load conditions are changed or difficult ground conditions are occuring.

To sum up the frequent causes of damage are insufficient material, false stacking, no backdrain, no foundation or embedding and non proper design of the whole Natural Rock Wall. One of these factors can lead to small stability problems, but a combination of more then one of these factors could lead to a collapse of the Natural Rock Wall.

4.3. Restoration Examples

In some cases it is possible to restorate damaged or slightly wrong constructed Natural Rock Walls. Two different renovation examples are pointed out.

<u>Example A</u>

The height of the Natural Rock Wall is approximately 5,5 m. The used blocks are in the armour stone class of HMB 300-1000 and their rock type is gneiss. The shape of the used blocks is platy-cuboid. The fine grained material which was used for chinking is washed out and bigger voids are formed. In some places the single blocks are only through specific points in contact, not over the whole block surface. This setting cause instabilities and it is possible that single blocks fall out.

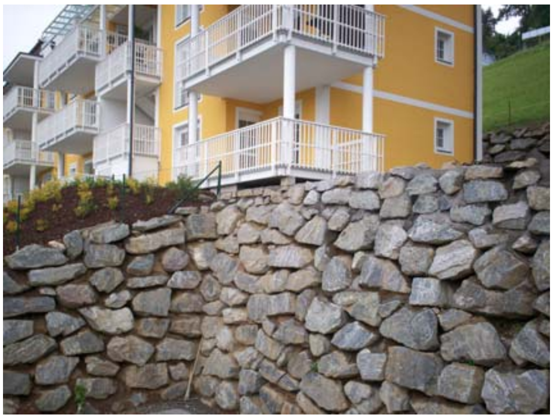

Fig. 41: Renovation Example A [6]

To close the voids between the single blocks and to increase the stability of the whole wall an appropriate action is needed. A simple and cheap thing was done to protect the wall. The complete chinking and all the voids are filled up with mortar, which also increased the stability. Furthermore drainage drillings should be provided that water pressure cannot build up behind the wall.

A possible alternative to protect little blocks from falling out of the wall is to attach a guard net over the whole Natural Rock Wall. This measure only protects blocks from falling out but it does not increase the stability of the whole wall.

Example B

This Natural Rock Wall has a height of approximately 3,0 m. The rock type of the used blocks is gneiss and they can be classified in the category of the armour stone class HMB 300-1000. For chinking between the single blocks fine grained material was used. On the top of the wall two rows masonry blocks are placed and above them a steel fence is mounted. Behind the Natural Rock Wall a road is situated.

Fig. 42: Renovation Example B [6]

The load conditions behind the Natural Rock Wall have changed which can lead to a possible collapse of the wall. Moreover the Natural Rock Wall was not designed for higher loads because the section width of the wall is too small. The resisting forces (self-weight) of the wall are not able to resist the additional driving forces (loads) and it is possible that the wall collapse. Countermeasures have to be done to increase the resisting forces and guarantee the stability of the wall.

In this case a designed nailing wall was placed in front of the Natural Rock Wall. As a possible alternative the Natural Rock Wall has to be designed new and reconstructed with a bigger section width. This alternative is only possible if enough free space for the new construction with bigger dimensions is available.

Moreover smaller problems like falling out of blocks or chinking mistakes are easy to restorate with mortar the whole wall or attaching a guard net. Bigger problems like design failures with a possible collapse of the wall are not so easy to restorate. An alternative to a complete new designed and reconstructed Natural Rock Wall is the placing of a nailing wall in front of the Natural Rock Wall.

5. Notes and Recommendations

The essential recommendations for the construction of high quality Natural Rock Walls are discussed. Furthermore the limits, advantages and disadvantages of Natural Rock Walls are analysed.

5.1 Construction of high quality and safe Natural Rock Walls

To construct a high quality and safe Natural Rock Wall without damage some essentials recommendations should be noticed.

Recommendations

- investigation of the building site
- proper design and calculation at the state of the art
- excavation according to design and soil properties
- foundation according to the ground condition, construction load and additional load
- usage of blocks in the armour stone class HMB 300 - 1000 and HMB 1000 – 3000 fulfilling the requirements of the standard [d], cubic shape of the blocks is preferable
- usage of the right chinking material (angular coarse grained material or mortar)
- construct always a multiple row Natural Rock Wall (in section at least two blocks per row)
- stacking in a dense bond with joints as small as possible
- horizontal joints should be normal to the inclined plain view
- vertical joints should be arranged shifted
- the inclined plain view of the wall should be almost planar
- the soil in front of the Natural Rock Wall should be compacted as soon as possible
- a backdrain behind the wall should be installed and separated through a geotextile from the surrounding backfill or native soil, the cover layer of the backdrain should be impermeable
- drainage drillings or drainage pipes for mortared walls are needed

Furthermore the skills and experience of the excavator operator play also a very essential role for the construction of high quality Natural Rock Walls. The operator has to fit the single blocks to a complete Natural Rock Wall like a puzzle with different big blocks. The greater the experience of the operator the faster is the construction and the higher is the quality of the wall.

5.2 Limits

In general Natural Rock Walls have an average height between 2 m and 6 m. But also walls with a height of more than 8 m have been built. In the following section advantages and disadvantages of Natural Rock Walls are discussed.

Advantages of Natural Rock Walls

- in general cheap construction
- aesthetic and natural like view
- flexible adaptable at each terrain
- not high requirements of settling sensitivity

Disadvantages of Natural Rock Walls

- the quality of the wall depends on the used material (blocks, chinking)
- in all of the cases also a big part of the quality of the wall depends on the experience of the operator, each Natural Rock Wall is unique and no one of the blocks looks like the other
- for high walls or bigger loads a bigger section width is needed, that means also bigger excavation and more space for the whole construction
- construction always from the bottom to the top, that means the back cut must be stable otherwise a temporary pit supporting system is needed
- rigid structural elements on the top of the Natural Rock Wall will show settlement effects

To sum up for small heights (<6 m), not so big loads and low settling sensitivity Natural Rock Walls are a good alternative to other retaining structures like reinforced concrete walls.

6. Outlook

The issue “Natural Rock Walls as retaining structures for slope stabilisation” offers also in the future several researches especially in the areas calculation parameters for Natural Rock Walls and calculation approaches.

Calculation parameters

The actual layout of the pull out trials should be reviewed and a new layout with the scope to determine the real friction angle within a complete Natural Rock Wall should be invented. With these new layout pull out trials with block above block, angular coarse grained material between the blocks, mortar between the blocks and also fine grained material between the blocks should be performed and so the real friction angle within the Natural Rock Wall should be determined for different chinking material.

Calculation approaches

Furthermore the actual limit equilibrium analysis of Natural Rock Walls should be reviewed in more detail and critical analysed. If more parameters of the material and also the behaviour of Natural Rock Walls are known in more detail numerical analysis of a Natural Rock Wall System could be a further step of research item.

References

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[15] ÖBB Infrasruktur: *Gestaltung und Dimensionierung von Mauern* – ÖBB Diensbehelf DB 740 Teil 5, 26.10.2009

[16] ÖBB Infrasruktur: *Steinsatz in Verwendung als Stützmauer* – ÖBB Diensbehelf DB 740 Teil 5, Regelzeichnung UM 1, 25.09.2009

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Standards

[a] ÖNORM EN 1997-1 *Eurocode 7: Geotechnical design – Part 1: General rules (consolidated version)*, 15.05.2009

[b] ÖNORM B 1997-1-1 *Eurocode 7: Geotechnical design – Part 1: General rules National specifications concerning ÖNORM EN 1997-1* and national supplements, 15.03.2010

[c] ÖNORM B 4434 *Erd- und Grundbau Erddruckberechnungen*, 01.01.1993

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Appendix

A. Pull out trials
  1. X-ray powder diffractogram blocks
  2. UCS blocks
  3. Comparative table grain shape [19]
  4. Grain size analysis fine grained material
  5. X-ray powder diffractogram fine grained material
  6. Shear test fine grained material

B. Standard Sections
  1. Standard Section Master Thesis
  2. Standard Section ÖBB [16, 17]
  3. Standard Section FHWA [4]

C. Calculation
  1. Partial factors of safety (according to Eurocode 7)
  2. Comparative calculations
  3. Sample calculations

A. Pull out trials

1. X-ray powder diffractogram blocks

2. UCS blocks

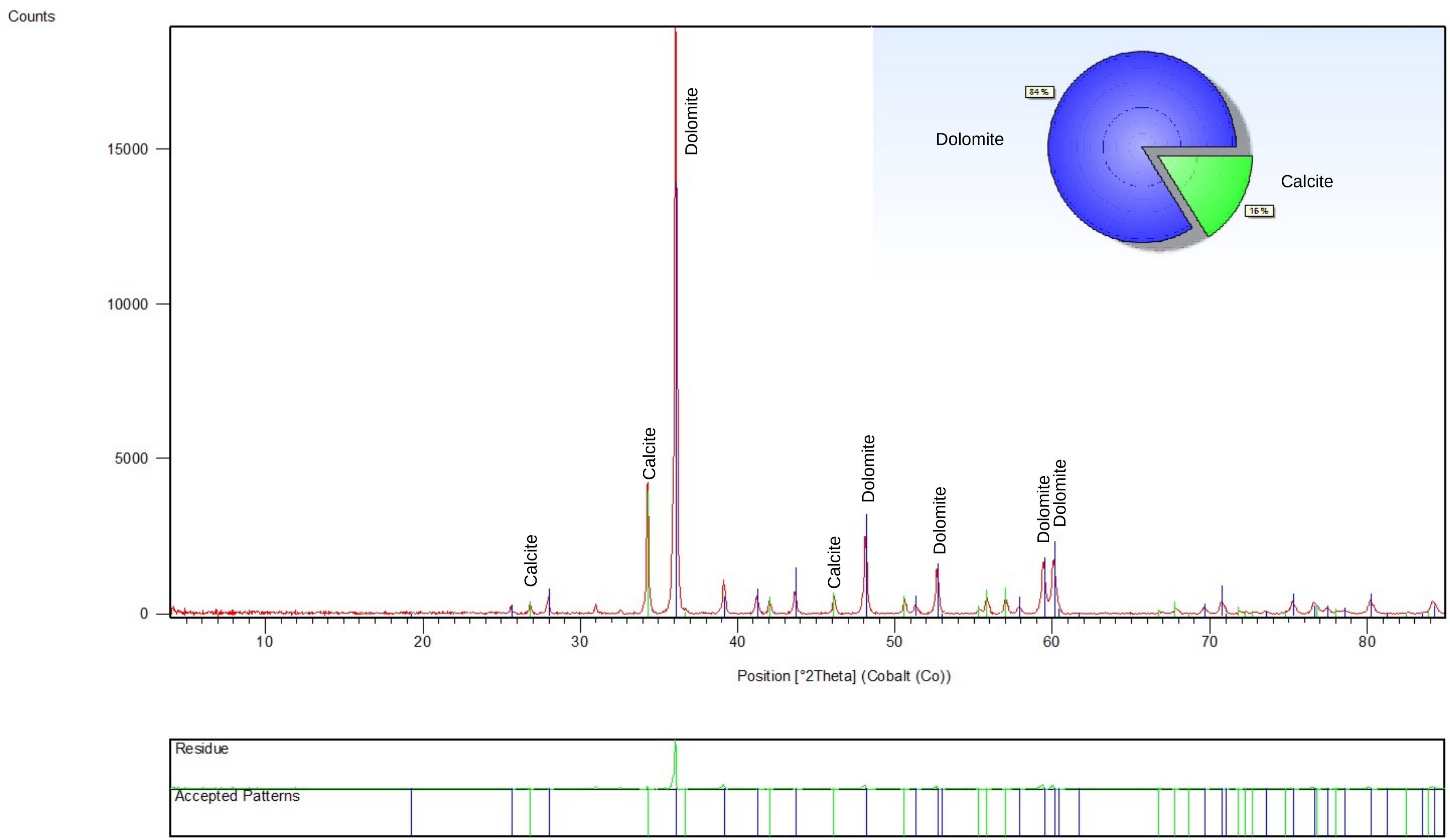
3. Comparative table grain shape [19]

4. Grain size analysis fine grained material

5. X-ray powder diffractogram fine grained material

6. Shear test fine grained material

A.1 X-ray powder diffractogram blocks

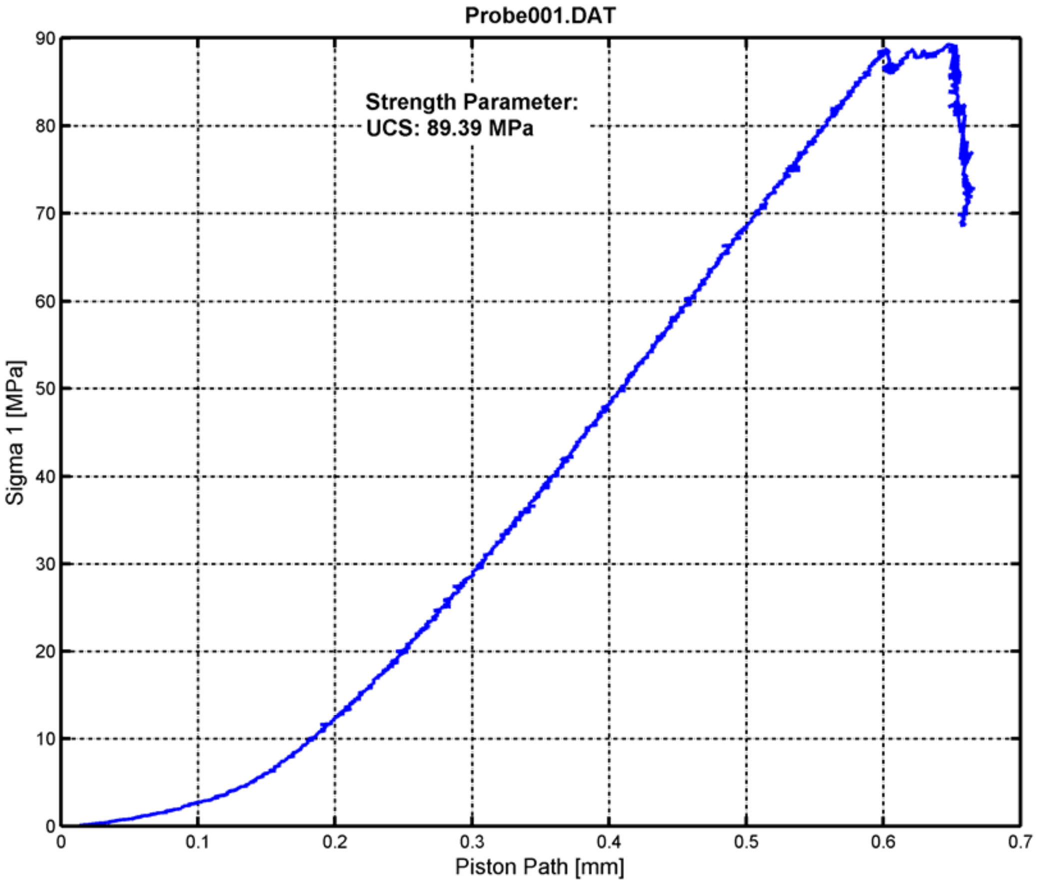

Counts
15000
10000
5000
0
10
20
30
40
50
60
70
80
Position [°2Theta] (Cobalt (Co))
Calcite
Calcite
Dolomite
Calcite
Dolomite
Dolomite
Dolomite
Dolomite
Dolomite
84 %
Calcite
16 %
Residue
Accepted Patterns

A.2 UCS blocks

Unconfined Compressive Strength Test

Sample properties		
Diameter	73,85	[mm]
Length	143,23	[mm]
Mass	1660,90	[g]
Current time	25,80	[µs]

UCS: 89400 kN/m²



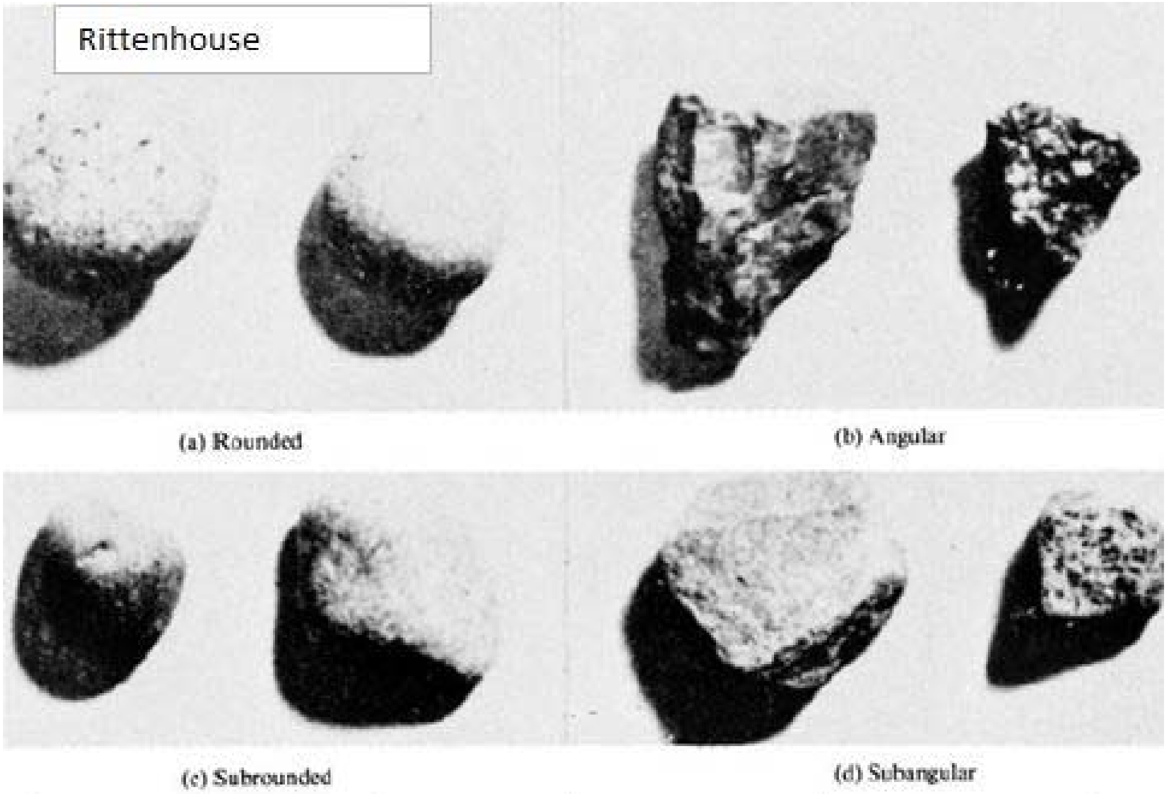
Sample before test



Sample after test




A.3 Comparative table grain shape [19]



A.4 Grain size analysis fine grained material

TUG
TU-GRAZ

INSTITUT FÜR BODENMECHANIK UND GRUNDBAU
GEOTECHNISCHES LABOR

A - 8010 GRAZ, RECHBAUERSTRASSE 12, AUSTRIA
Tel.: +43(0)316/873 6237 Fax: +43(0)316 / 873 6238

VORSTAND: O. Univ. Prof. Dipl. Ing. Dr. techn. STEPHAN SEMPRICH

IBG

KORNGRÖSSENVERTEILUNG

ANGABEN ZUR PROBE

Projekt: Masterarbeit Florian Steiner	**Labornummer: 18220**
Auftraggeber:	Bearb.: / Datum: 09.04.2013
Bezeichnung: feinkörniges Material	Höhe:
Lage:	bezogen auf
Herkunft:	[ ] eingebaut / [ ] nicht eingebaut
Notiz:	[x] gestört / [ ] ungestört

ZUSAMMENSTELLUNG DER SIEBDURCHGÄNGE

Siebanalyse

Ø	%	%
200	-	
120	-	
100	-	
63	-	
45	-	
31,5	100,0	
22,4	-	
16	95,9	
11,2	-	
8	86,6	
4	73,7	
2	62,3	
1	53,7	
0,5	45,6	
0,25	40,8	
0,125	37,6	
0,09	-	
0,063	-	

Schlämmanalyse

Ø	%	Ø	%
0,0642	29,9		
0,0472	26,5		
0,0340	24,7		
0,0221	22,0		
0,0133	18,0		
0,0079	13,9		
0,0050	11,6		
0,0029	9,5		
0,0015	7,7		

rechnerisches Größtkorn		
0.063	29,7 %	
0.02	21,2 %	
0.002	8,5 %	
U=D60/D10	501	

Sachbearbeiter/Datum:

TUG
TU-GRAZ

INSTITUT FÜR BODENMECHANIK UND GRUNDBAU
GEOTECHNISCHES LABOR

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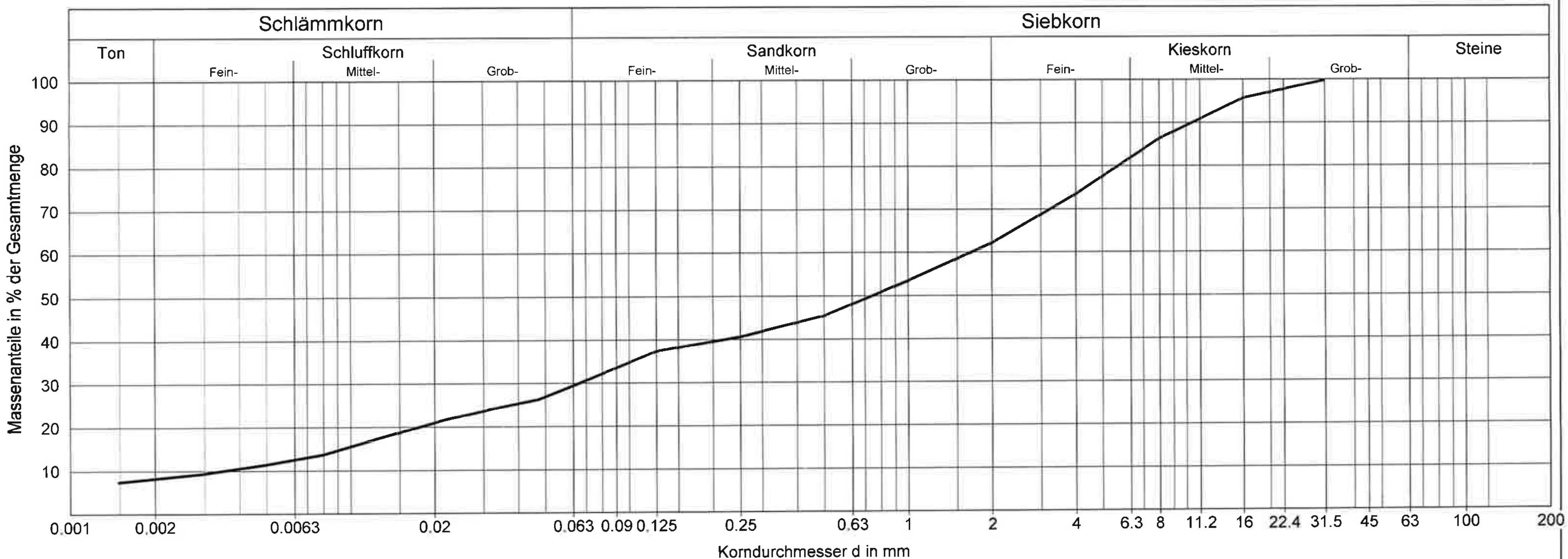
VORSTAND: O. Univ. Prof. Dipl. Ing. Dr. techn. STEPHAN SEMPRICH

IBG

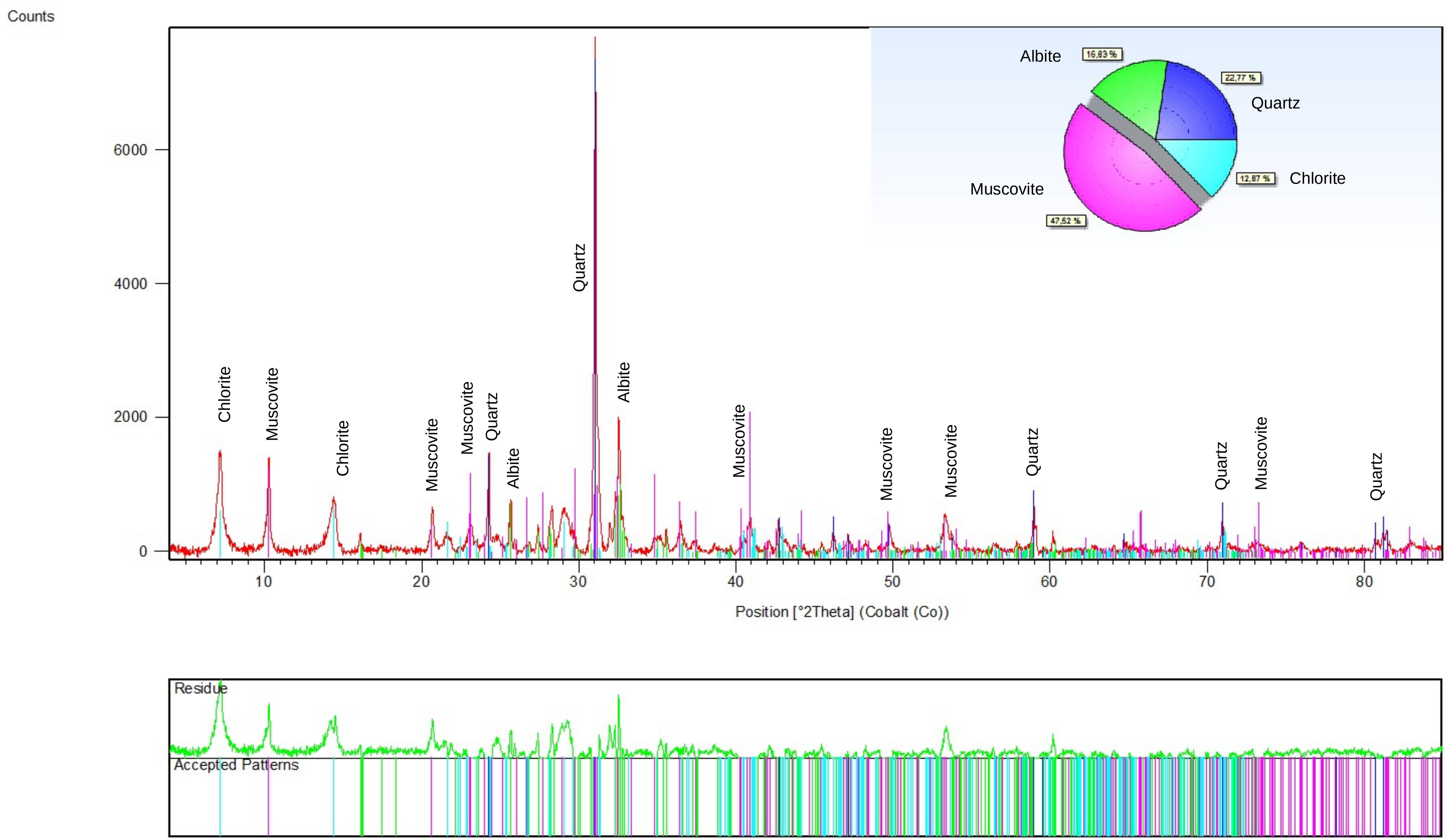
Auftraggeber	Projekt	Projekt Nr	Bezeichnung	Bearbeiter	Datum
	Masterarbeit Florian Steiner		feinkörniges Material Kantkorn		09.04.2013

KÖRNUNGSLINIE



Labor Nr.	Signatur	Anteile kleiner Ø	Ton %	Schluff %	Sand %	Kies %	Steine %	Anteile größer Ø	Bodenart	d10
18220	———	0,002: 8,5%	8,5	21,2	32,6	37,7			cl' si Sa Gr	0,0033

A.5 X-ray powder diffractogram fine grained material


Counts
6000
4000
2000
0
Chlorite
Muscovite
Chlorite
Muscovite
Muscovite
Quartz
Albite
Quartz
Albite
Muscovite
Muscovite
Muscovite
Quartz
Quartz
Muscovite
Quartz
10
20
30
40
50
60
70
80
Position [°2Theta] (Cobalt (Co))
Albite
16,83 %
22,77 %
Quartz
12,87 %
Chlorite
Muscovite
47,52 %
Residue
Accepted Patterns

A.6 Shear test fine grained material

TU Graz

INSTITUT FÜR BODENMECHANIK UND GRUNDBAU
GEOTECHNISCHES LABOR
A-8010 GRAZ, RECHBAUERSTRASSE 12, AUSTRIA
Tel.: +43 (0) 316 873-6237   Fax.: +43 (0) 316 873-6238

IBG

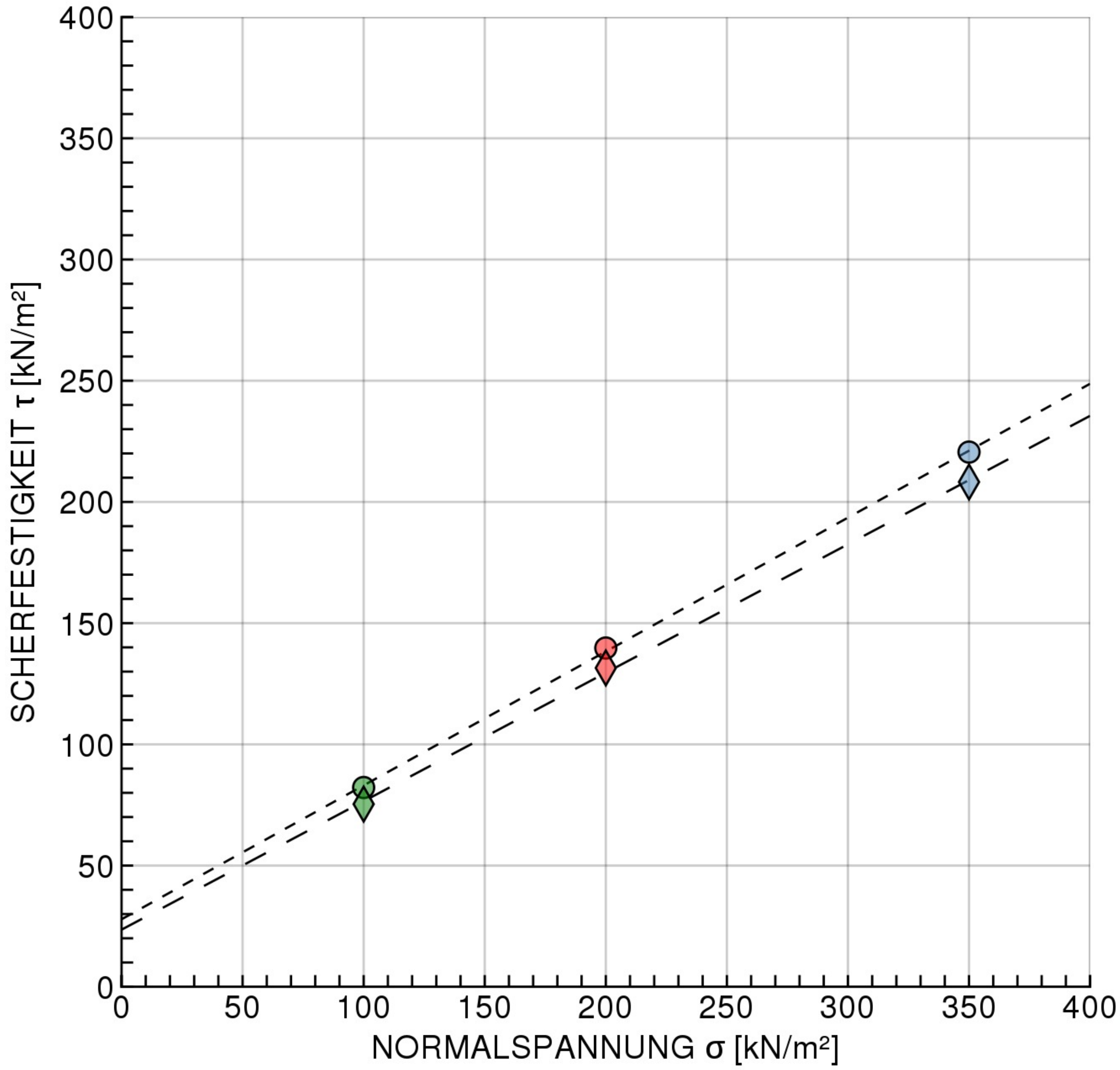
AUFTRAGGEBER: **Institut für Bodenmechanik und Grundbau**	BODENART: **cl' si Sa Gr**	LABORNUMMER: **18222**
PROJEKT: **Diplomarbeit**	TIEFE:	AUFTRAGSNR: **2622**
BEZEICHNUNG: **MA Steiner**	BEARBEITER: **OM**	DATUM: **10.04.13 - 18.04.13**

RAHMENSCHERVERSUCH NACH ÖNORM . 4416

Büchsengröße: 100 x 100 x 20 mm

GRÖSSTKORN: < 4 mm			**Versuch 1**	**Versuch 2**	**Versuch 3**
KONSOLIDIERUNGSDRUCK	σ_c	kN/m²	550	550	550
KONSOLIDIERUNGSZEIT	t_c	h	72	72	72
NORMALSPANNUNG	σ	kN/m²	100	200	350
SCHERFESTIGKEIT	τ_f	kN/m²	82,2	139,7	220,6
SCHERWEG	s_1	mm	16,6	16,6	12,5
RESTSCHERFESTIGKEIT	τ_r	kN/m²	75,3	131,5	208,2
RESTSCHERWEG	s_2	mm	270,8	268,5	254,5
WASSERGEHALT nach dem Versuch	w	%	19,1	18,6	16,6

REIBUNGSWINKEL (ϕ')	28,9	°	PROBENZUSTAND	gestört
KOHÄSION (c')	27,8	kN/m²	SCHERGESCHWINDIGK9Ḽ¢	лΣлло ϒϒkmin
RESTSCHERWINKEL (ϕ)	27,9	°	RESTSCHERGESCHWINDIGϒ9Ḽ¢	нΣону mm/min



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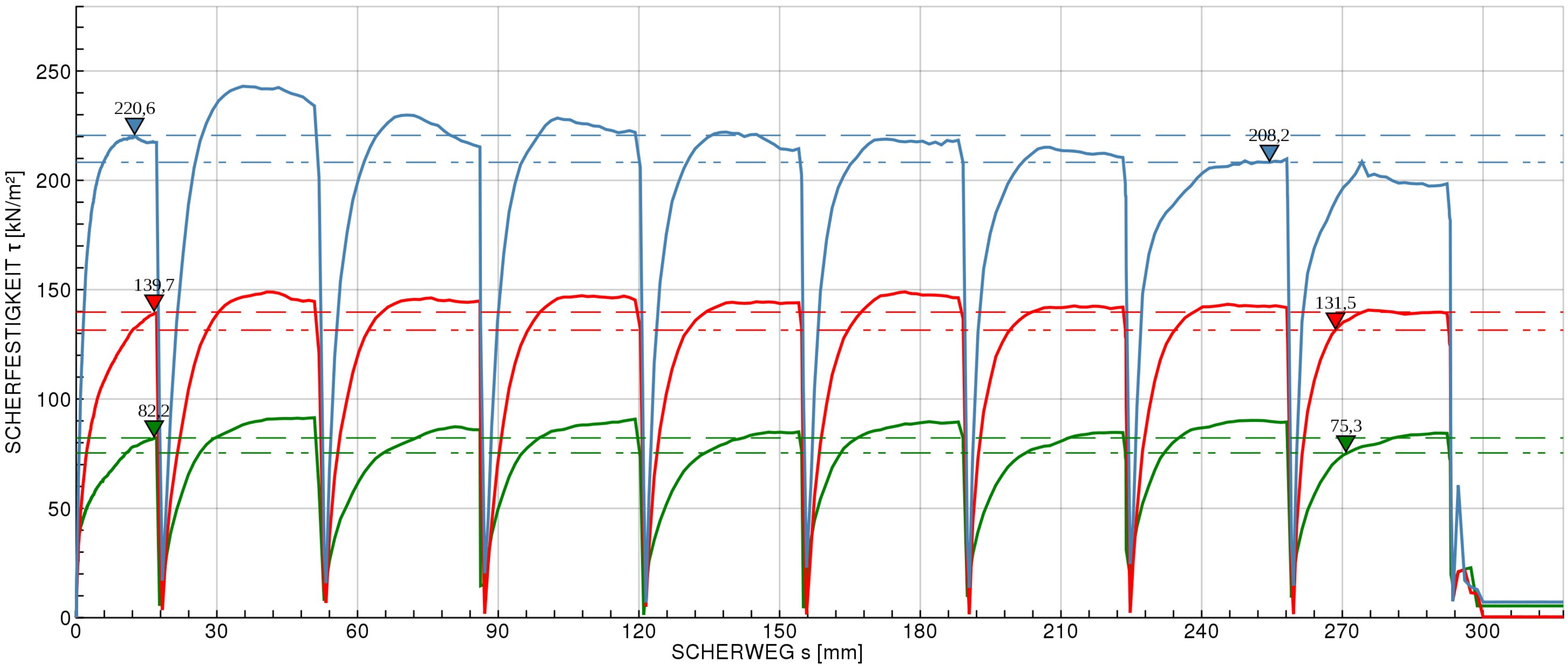
IBG

σ = 100 kN/m², t = 72 h, σc = 550 kN/m², gestört	
σ = 200 kN/m², t = 72 h, σc = 550 kN/m², gestört	
σ = 350 kN/m², t = 72 h, σc = 550 kN/m², gestört	

Labornummer	**18222**
Versuchsdatum	**10.04.13 - 18.04.13**
Schergeschwindigkeit	**0,003 mm/min**
Größtkorn	**< 4 mm**



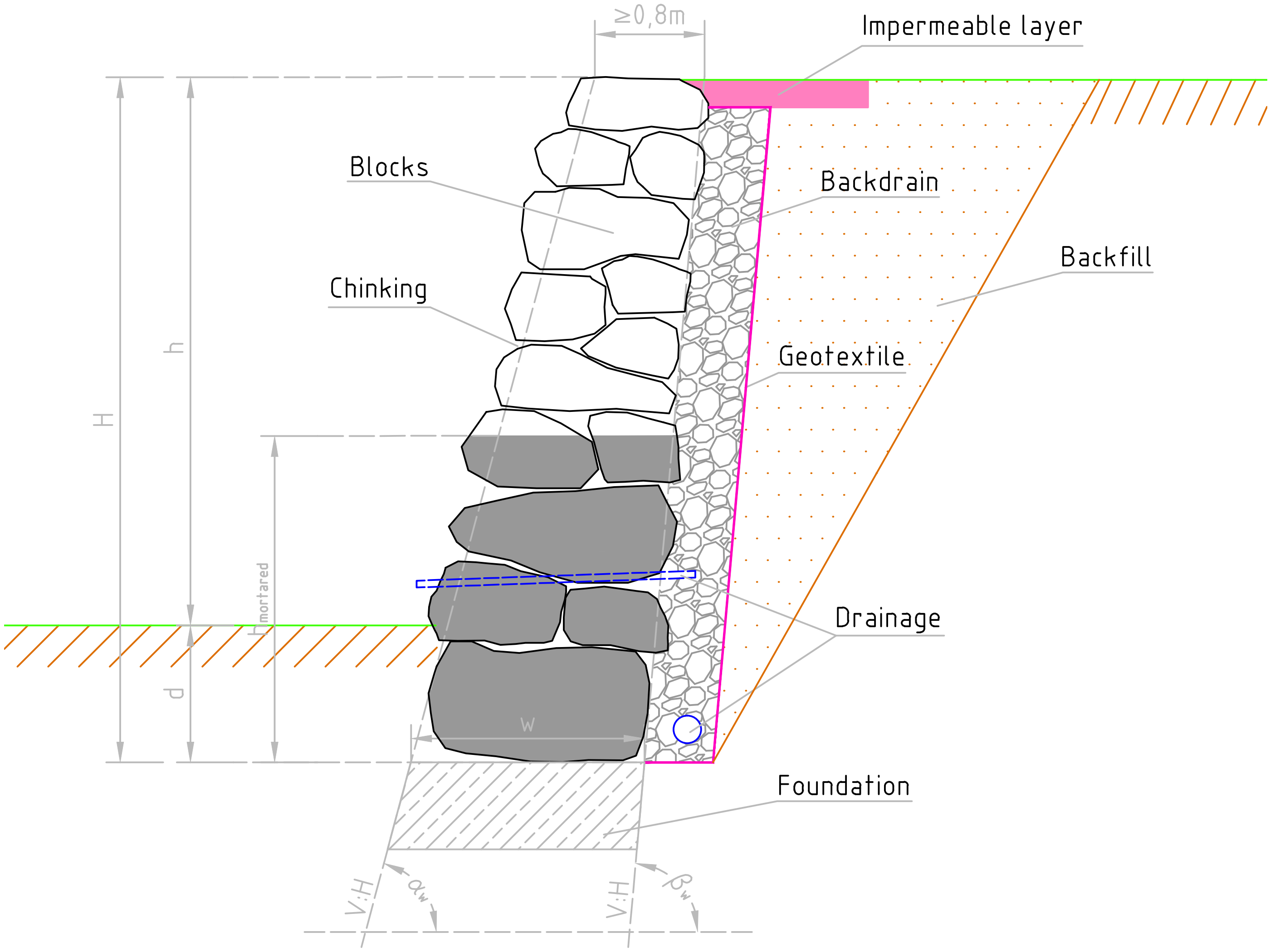
B. Standard Sections

1. Standard Section Master Thesis

2. Standard Section ÖBB [16, 17]

3. Standard Section FHWA [4]

B.1 Standard Section Master Thesis



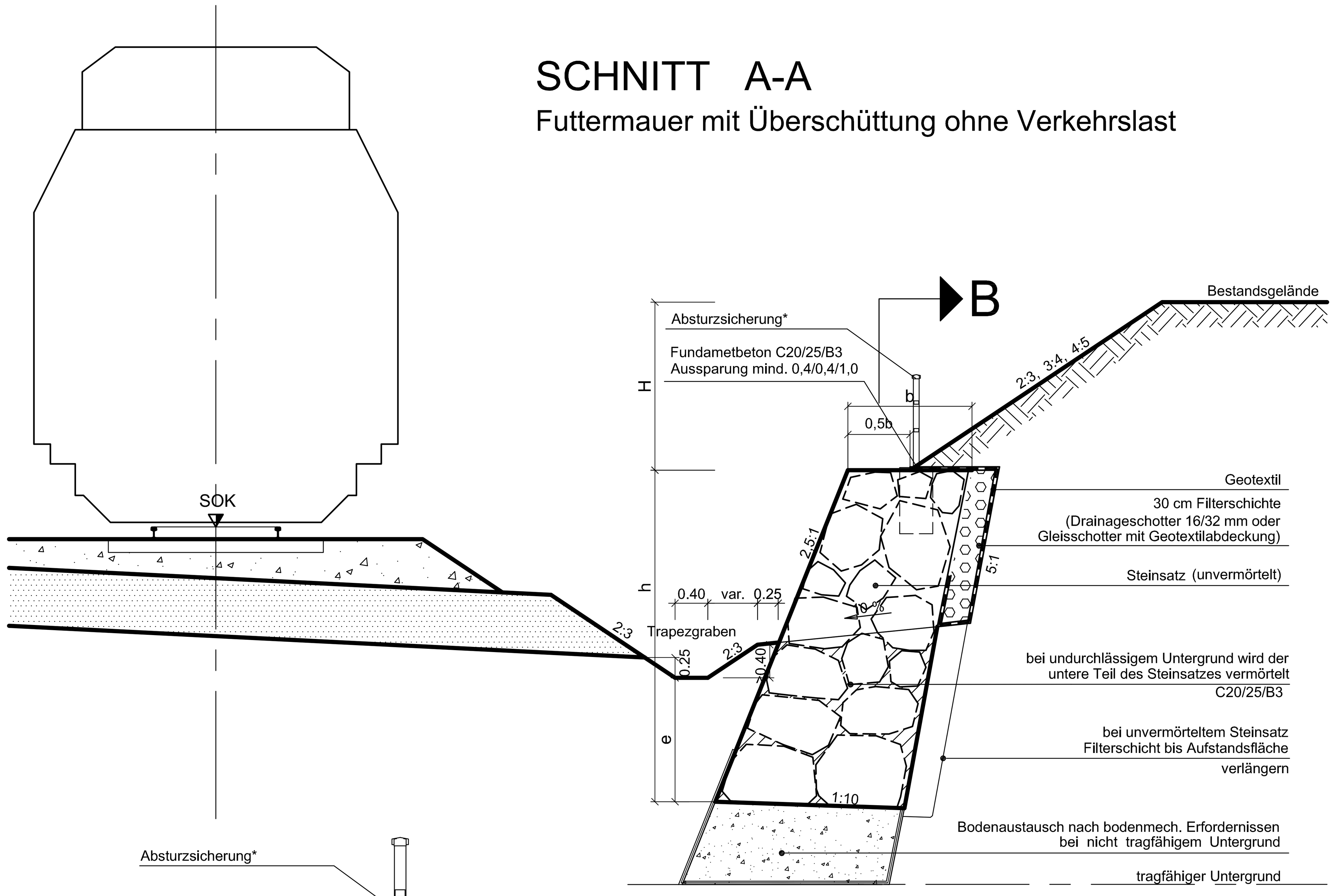
Geometry							
	H [m]	h [m]	d [m]	$h_{mortared}$[m]	w [m]	α_w [°], V:H	β_w [°], V:H
Wall							

Material		
	Type	Notes
Foundation		
Rock Category		
Chinking		
Geotextile		
Drainage		
Backdrain		
Backfill		
Impermeable Layer		

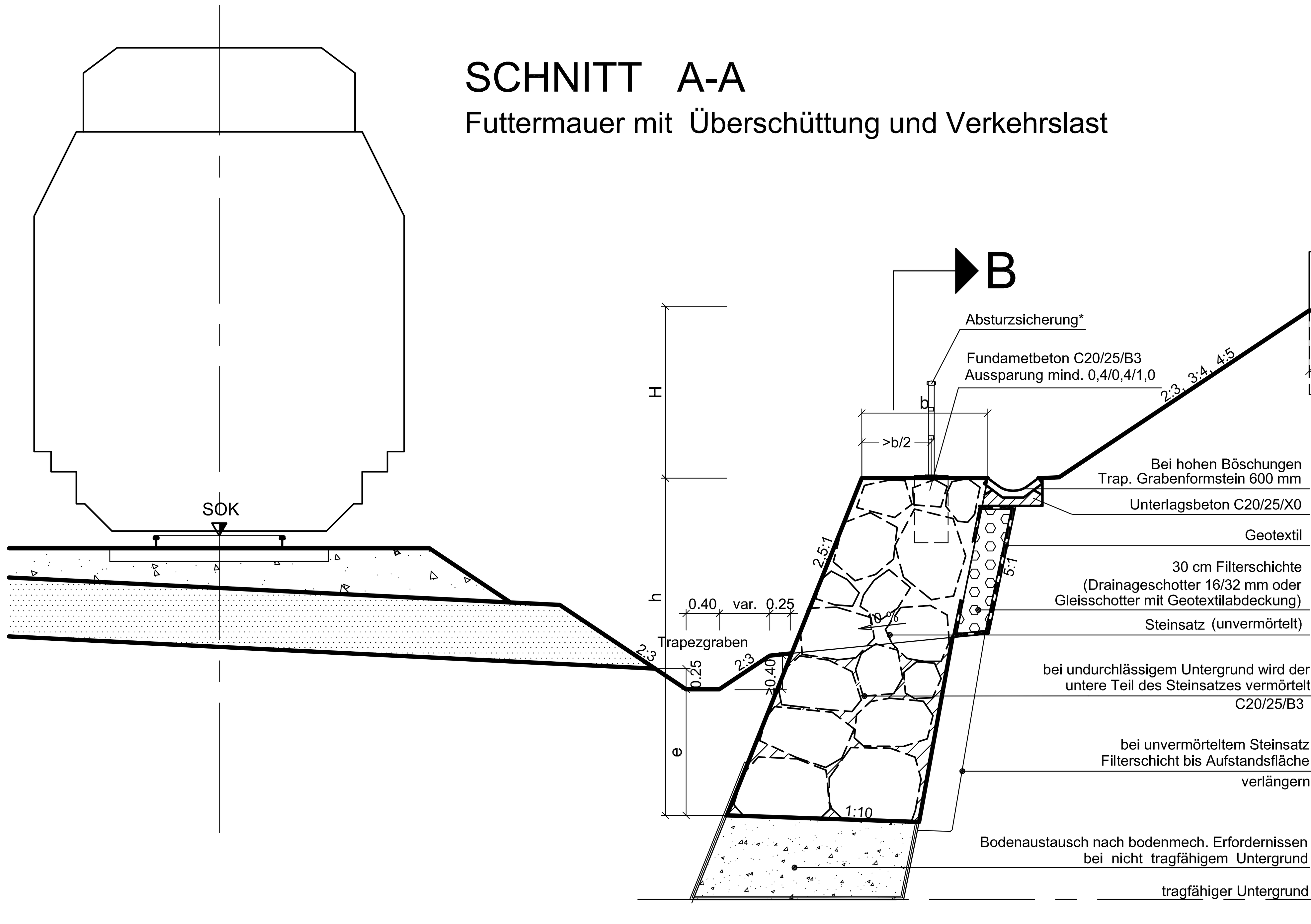
Content		
Natural Rock Wall Standard Section		
Number 001	Scale -	Designer Florian Steiner

B.2 Standard Section ÖBB [17, 18]

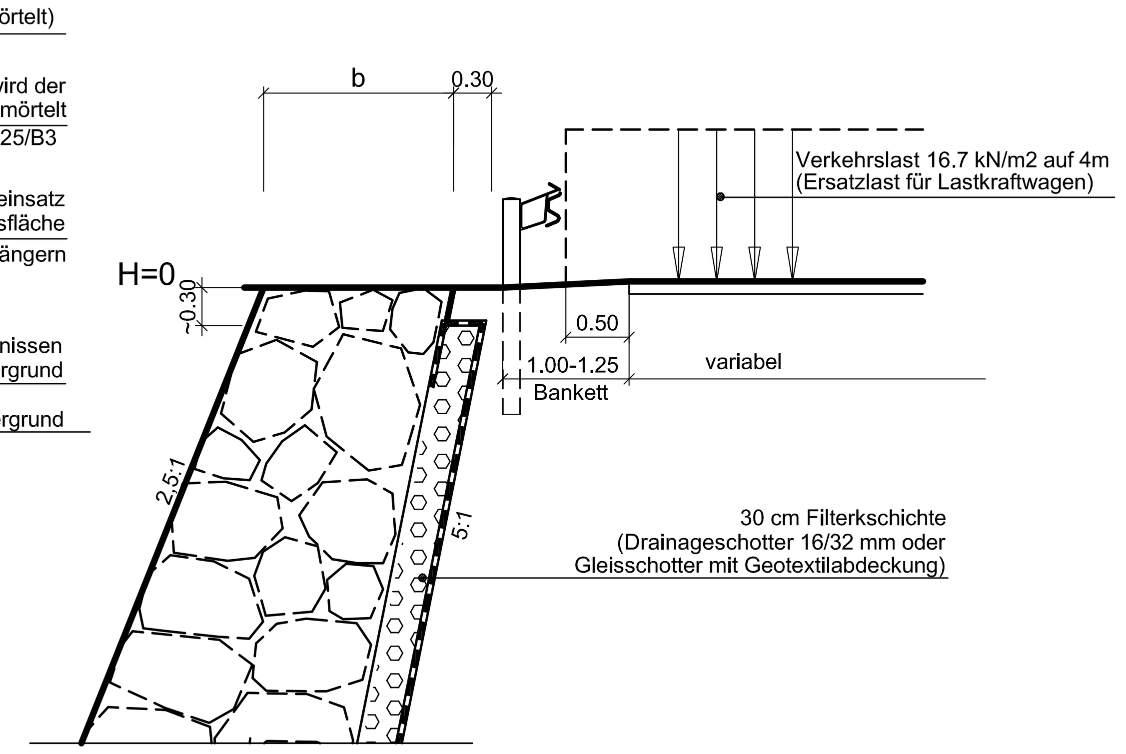
SCHNITT A-A
Futtermauer mit Überschüttung ohne Verkehrslast

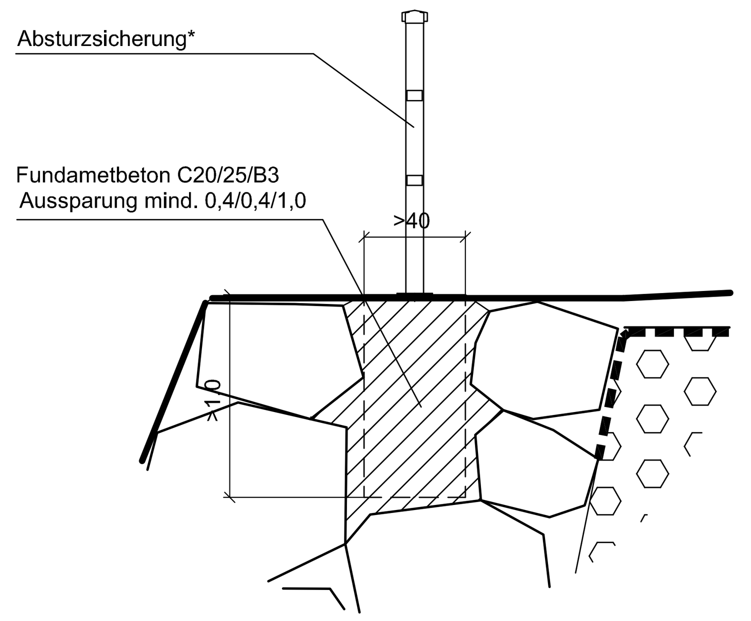


SCHNITT A-A
Futtermauer mit Überschüttung und Verkehrslast



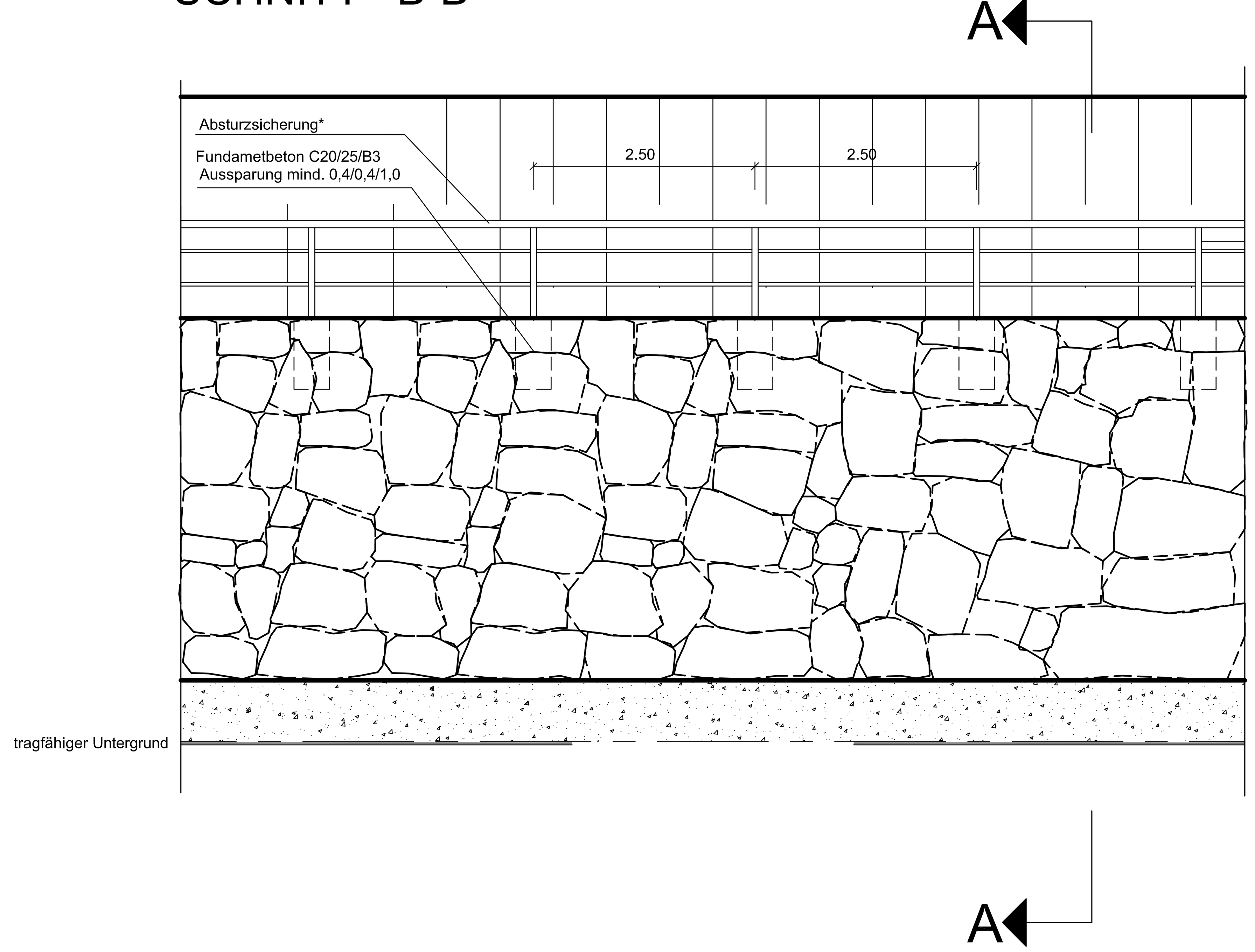
SCHNITT A-A
Futtermauer ohne Überschüttung und Verkehrslast





Fundamente Absturzsicherung

SCHNITT B-B



Einbindetiefe e:
Frosttiefe, jedoch mind. 1.0 m bis zu einer freien Standhöhe (h-e) von 2.5 m
Frostiefe, jedoch mind. 1.5 m ab einer freien Standhöhe (h-e) von 2.5 m

Kennwerte für Steinsatz:
Winkel der inneren Reibung = 40°
Kohäsion = 100kN/m2
Wichte = 22,5 kN/m3

Baustoffe:
Steine der Güte CS 130
h < 3,0 m HMB 300/1000
h ≥ 3,0 m HMB 1000/3000

Böschungsneigung Überschüttung:

Winkel der inneren Reibung	mögliche Böschungsneigung
35°	2 : 3
37,5°	3 : 4
40°	4 : 5

Kronenbreite b der Futtermauer [m] (ohne Verkehrslast)

Überschüttung H [m]	Mauerhöhe h [m] 2,0	3,0	4,0	5,0	6,0
H= 0,0	1,5	1,5	1,5	1,5	1,5
2,0			1,5	1,5	1,5
4,0			1,5	1,5	1,6
6,0			1,5	1,7	1,9
8,0			1,6	1,9	2,1
10,0			1,7	1,9	2,3
12,0			1,8	2,1	2,4
16,0			1,8	2,2	2,6
ab 20,0	1,5	1,5	1,8	2,2	2,7

Kronenbreite b der Futtermauer [m] (mit Verkehrslast)

Überschüttung H [m]	Mauerhöhe h [m] 2,0	3,0	4,0	5,0	6,0
H= 0,0	1,5	1,5	1,5	1,5	1,5
2,0			1,5	1,5	1,5
4,0			1,5	1,6	1,7
6,0			1,6	1,8	2,0
8,0			1,6	1,9	2,2
10,0			1,7	1,9	2,3
12,0			1,8	2,1	2,4
16,0			1,8	2,2	2,6
ab 20,0	1,5	1,5	1,8	2,2	2,7

*) Absturzsicherung:
Befindet sich die Krone einer Mauer in unmittelbaren Nahbereich von Verkehrsflächen (begehbarer Sicherheitsraum, Bedienungsraum, sonstige Wege), ist eine Absturzsicherung mit einer Mindesthöhe von 1,0 m erforderlich.

ÖBB INFRASTRUKTUR

DIENSTBEHELF DB 740

UNTERBAU
TEIL 5
GESTALTUNG UND DIMENSIONIERUNG VON MAUERN

REGELZEICHNUNG

Regelzeichnung Nr.	Rev.	Datum
UM 2	A	25.09.2009

Änderung	Datum	Name	Art der Änderung	Zustimmung
D				
C				
B				
A	25.09.2009	wal	Anpassung Nomenklatur;	

Bearbeitet / Geprüft / Freigegeben / File Name

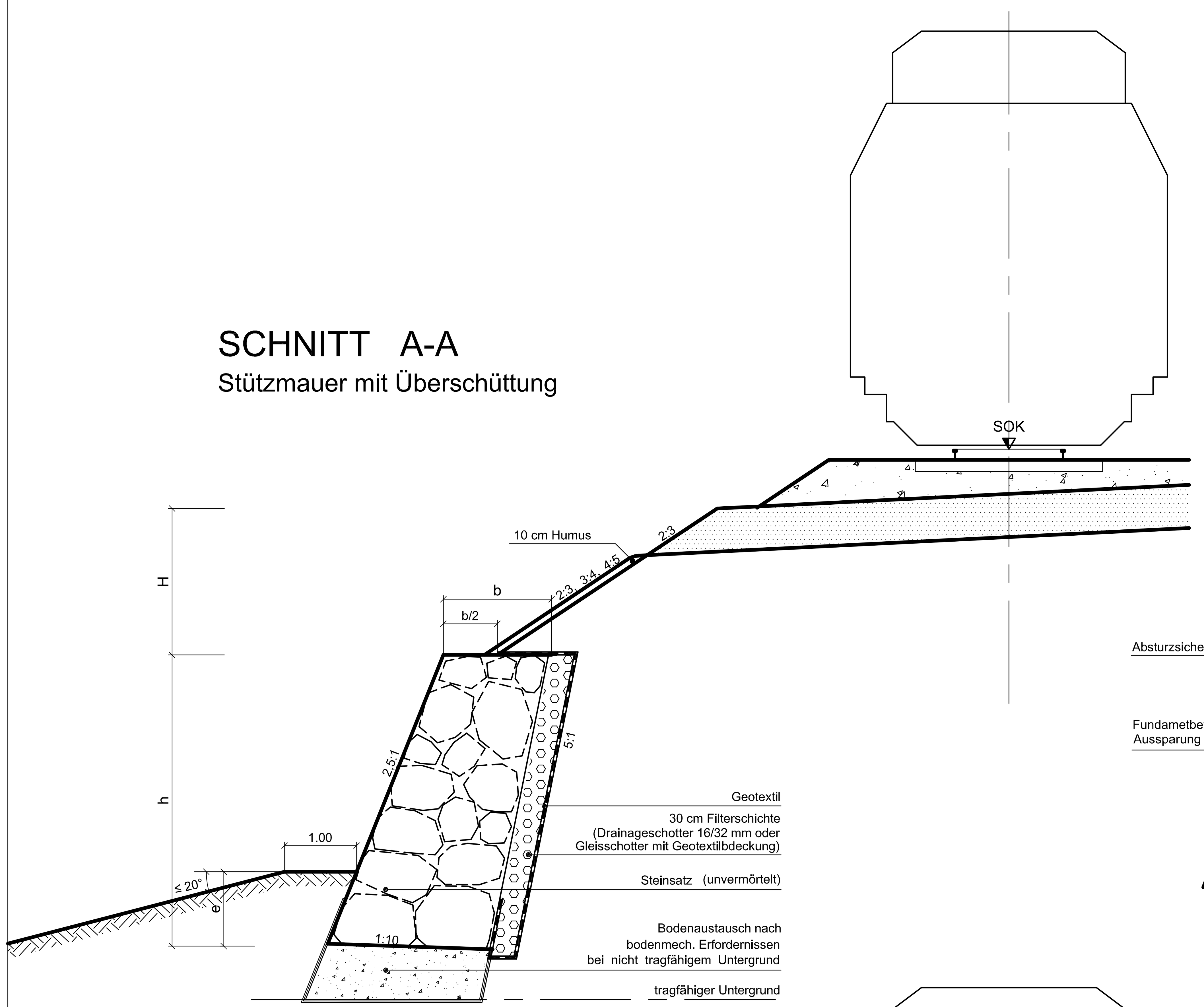
STEINSATZ IN VERWENDUNG ALS FUTTERMAUER

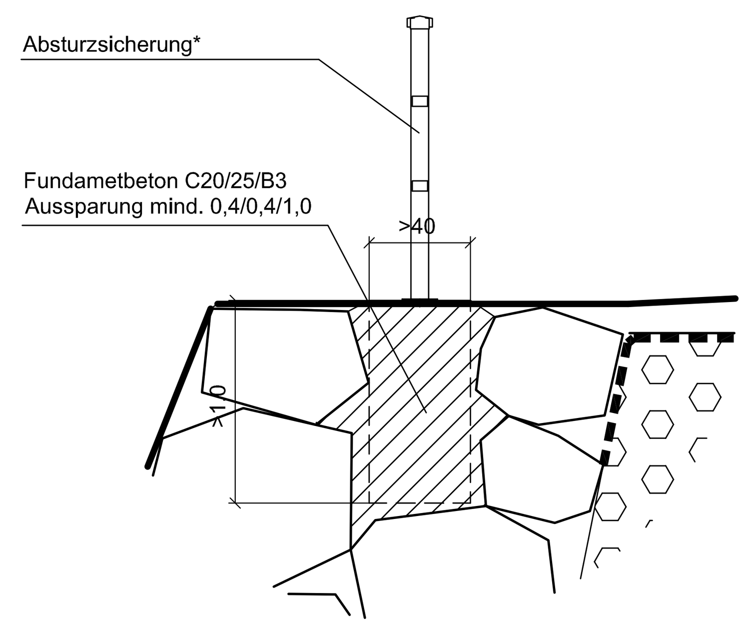
Maßstab	Plangröße	Ausgabe Datum
1:50 1:25	0,594 m x 0,841 m = 0,50 m²	25.09.2009

PLANUNG:

FACHREFERENT:
ÖBB Infrastruktur AG
Engineering Services
Tiefbau Geotechnik

SCHNITT A-A
Stützmauer mit Überschüttung





Fundamente Absturzsicherung

Einbindetiefe e:

Frosttiefe, jedoch mind. 1.0 m bis zu einer freien Standhöhe (h-e) von 2.5 m

Frosttiefe, jedoch mind. 1.5 m ab einer freien Standhöhe (h-e) von 2.5 m

Kennwerte für Steinsatz:

Winkel der inneren Reibung = 40°

Kohäsion = 100kN/m2

Wichte = 22,5 kN/m3

Baustoffe:

Steine der Güte CS 130

h < 3,0 m HMB 300/1000

h ≥ 3,0 m HMB 1000/3000

Böschungsneigung Überschüttung:

Winkel der inneren Reibung	mögliche Böschungsneigung
35°	2 : 3
37,5°	3 : 4
40°	4 : 5

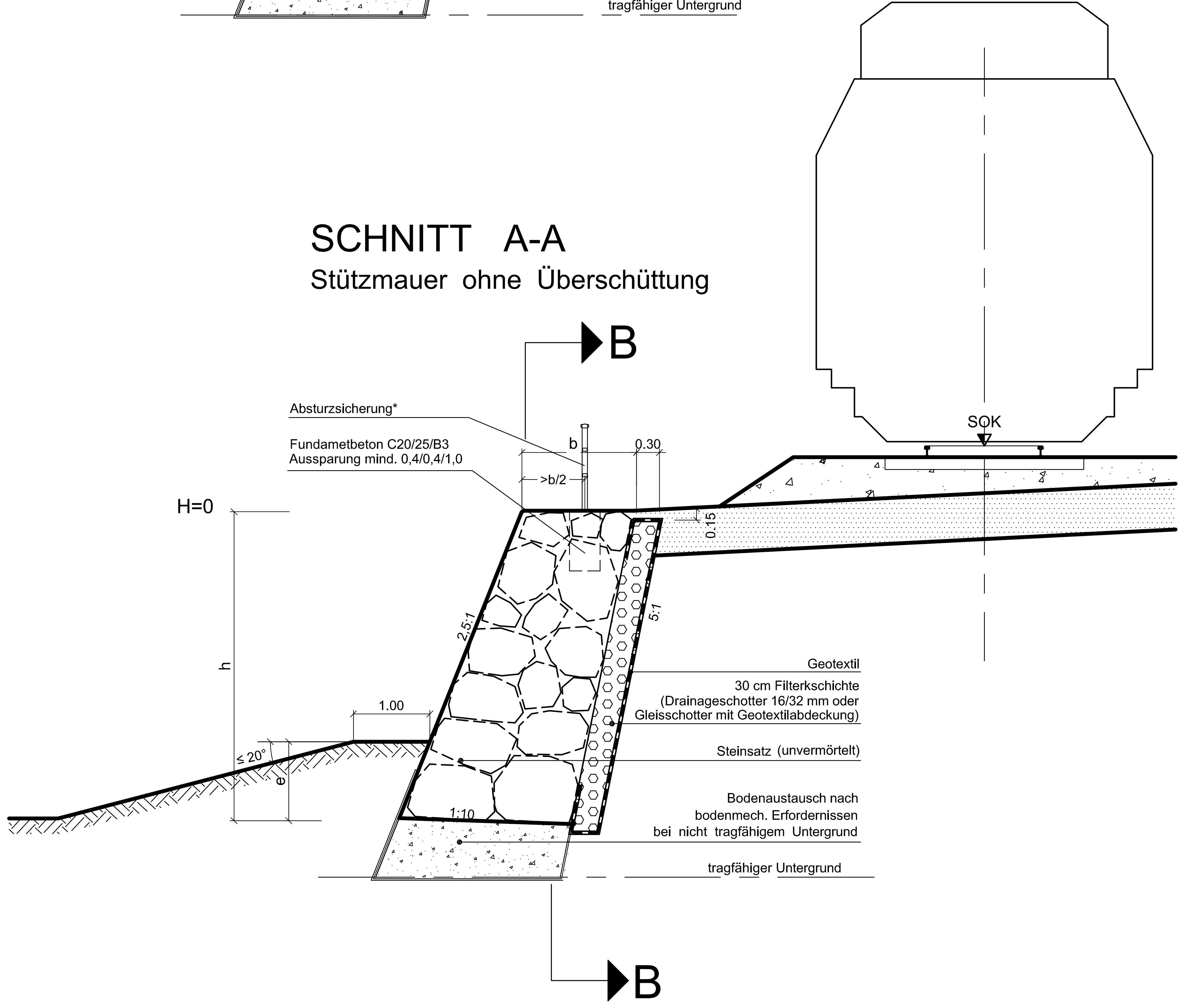
Kronenbreite b der Stützmauer [m]

Überschüttung	Mauerhöhe h [m]				
H [m]	2,0	3,0	4,0	5,0	6,0
H= 0,0	1,5	1,5	1,5	1,5	1,5
2,0	↓	↓	1,5	1,5	1,5
4,0			1,5	1,6	1,7
6,0			1,5	1,7	2,0
8,0			1,6	1,9	2,2
10,0			1,7	2,0	2,3
12,0			1,8	2,1	2,4
16,0			1,8	2,2	2,6
ab 20,0	1,5	1,5	1,8	2,2	2,7

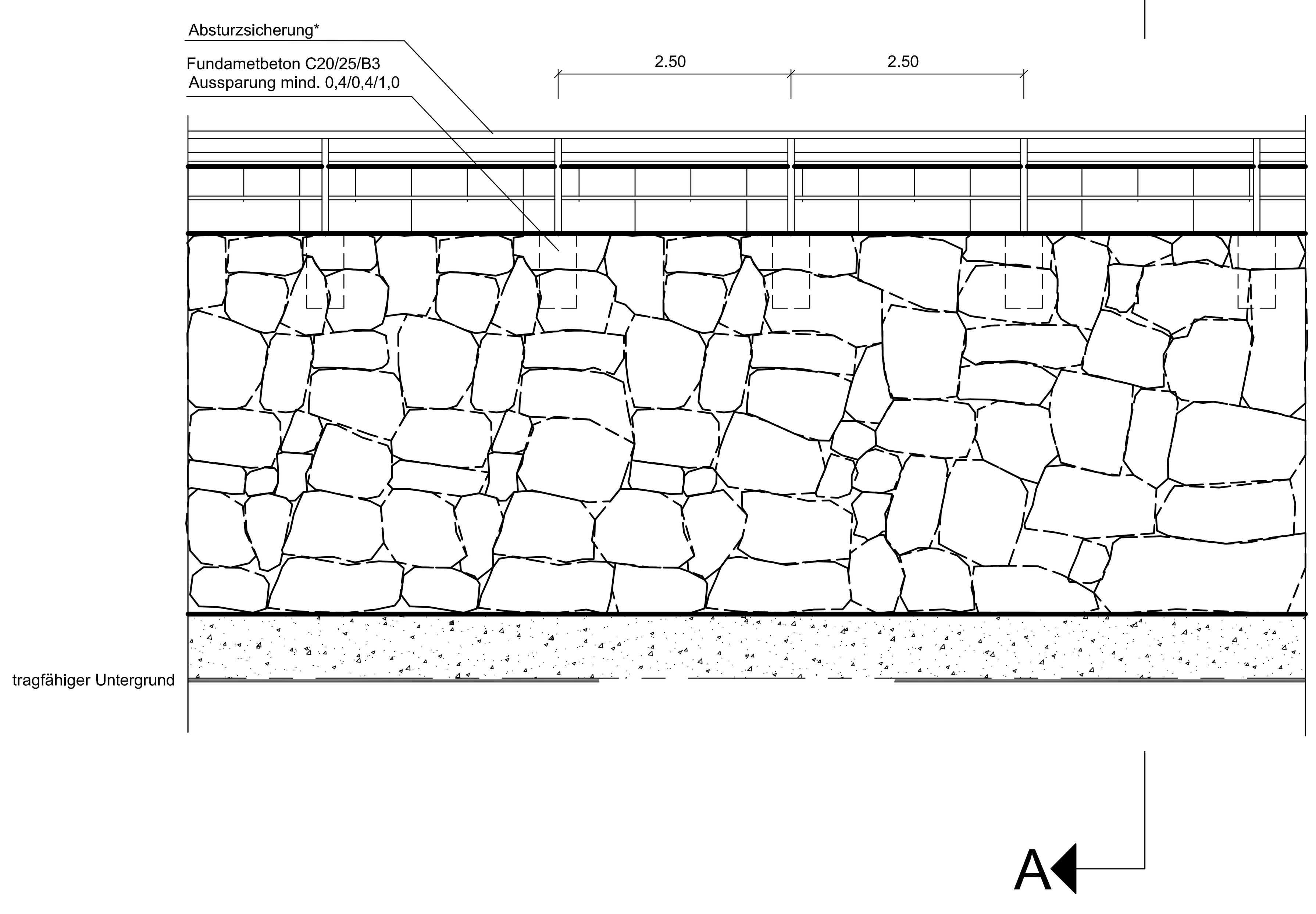
bei Geländeneigung >20° talseits der Stützkonstruktion ist für den Steinsatz eine gesonderte Dimensionierung erforderlich

*) Absturzsicherung:
Befindet sich die Krone einer Mauer in unmittelbaren Nahbereich von Verkehrsflächen (begehbarer Sicherheitsraum, Bedienungsraum, sonstige Wege), ist eine Absturzsicherung mit einer Mindesthöhe von 1,0 m erforderlich.

SCHNITT A-A
Stützmauer ohne Überschüttung



SCHNITT B-B



ÖBB INFRASTRUKTUR

DIENSTBEHELF DB 740

UNTERBAU

TEIL 5

GESTALTUNG UND DIMENSIONIERUNG VON MAUERN

REGELZEICHNUNG

Regelzeichnung Nr.	Rev.	Datum
UM 1	A	25.09.2009

D				
C				
B				
A	25.09.2009	wal	Anpassung Nomenklatur;	
Änderung	Datum	Name	Art der Änderung	Zustimmung

Bearbeitet / Geprüft / Freigegeben / File Name

STEINSATZ IN VERWENDUNG ALS STÜTZMAUER

Maßstab	Plangröße	Ausgabe Datum
1:50 1:25	0,594 m x 0,841 m = 0,50 m²	25.09.2009

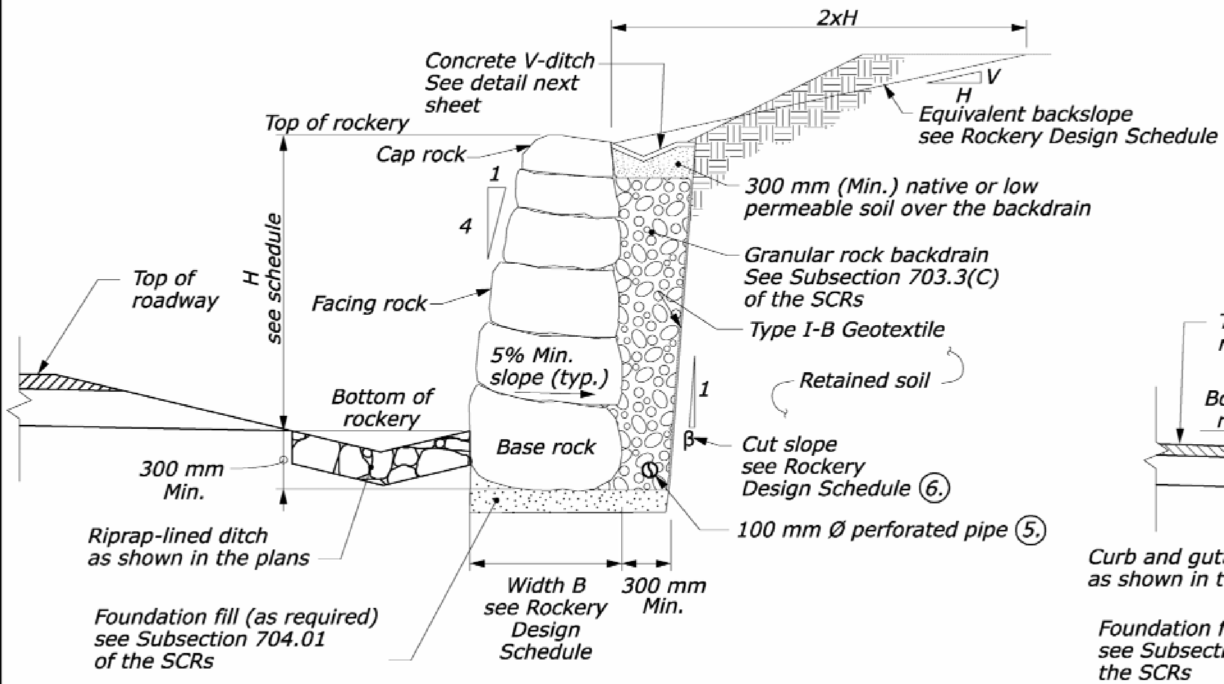
PLANUNG:

FACHREFERENT:

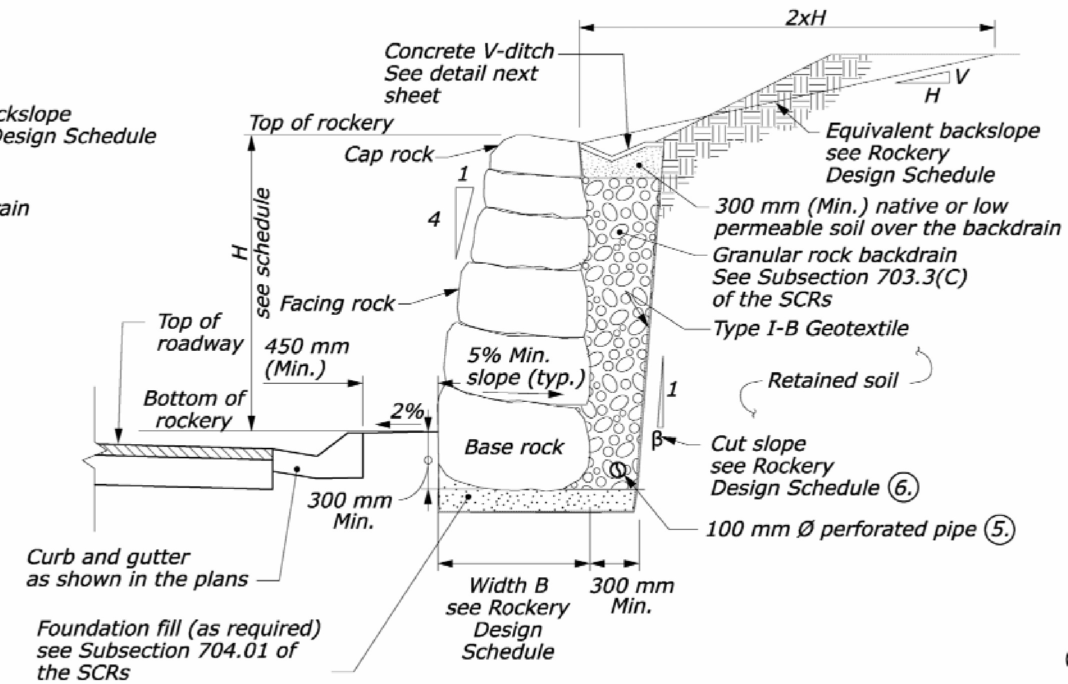
ÖBB Infrastruktur AG
Engineering Services
Tiefbau Geotechnik

B.3 Standard Section FHWA [4]

REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS



ROCKERY WITH RIPRAP-LINED DITCH TYPICAL SECTION



ROCKERY WITH CURB AND GUTTER TYPICAL SECTION

NOTE:

1. Construct rockery and place base, facing, and cap rocks according to Section 252 of the SCRs. Place each rock individually by equipment suitable for lifting, manipulating, and placing rocks of the size and shape specified. Ensure that each rock is firmly set and supported by underlying materials and adjacent rocks. Reposition or replace loose rocks.

2. A maximum tolerance of 150 mm may be applied toward the total base rock width. Use rock with minimum L of 1700 mm. When L exceeds 1700 mm, two approximately equal size base rocks may be used, provided rocks are in contact at two points or more. Do not consecutively place base rocks with widths less than B.

3. Place base, facing, and cap rocks so that their height dimension is not greater than their width. The longest dimension of the base, facing, and cap rocks is perpendicular to face of rockery.

4. Where loose, soft, or otherwise unsuitable foundation soil conditions are encountered, contact the CO for supplemental recommendations.

5. Surround the perforated pipe on all sides by at least 100 mm of permeable backfill according to Subsection 703.04.

   Discharge outlet pipes to a protected outlet or other permanent drainage structure at low points in the rockery and at 30 m (max.) spacing. Drain outlets should not empty into storm drains that are designed to back-up during heavy flows.

6. Stability of temporary cut slopes is the responsibility of the Contractor.

7. Do not construct rockeries or slopes exceeding the heights shown on the Rockery Design Schedule without prior written approval by the CO.

8. Construct rockeries parallel to curb grade unless otherwise noted.

ROCKERY DESIGN SCHEDULE										
STATION		LT/RT TIER	MAX. HEIGHT H (m)	MIN. BASE ROCK WIDTH B (m)	MIN. CUT A. SLOPE BATTER β V:H	MAX. EQUIVALENT BACKSLOPE V:H	MIN. ROCK WEIGHT (kg)		DITCH TYPE	SURCHARGE TYPE B.
BEGIN	END						CAP ROCK	BASE ROCK		

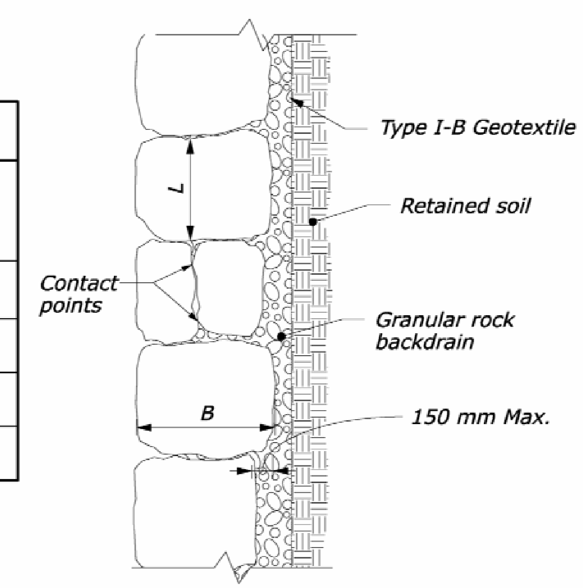
Rockery Design Data:

Friction angle, Ø = ___ °
Cohesion, c = 0
Bulk unit weight, γ_R = 23.5 kN/m³
Allowable bearing pressure = ___ KPa

A. Minimum cut slope for design purposes only. Actual cut slope batter may be greater 6.

B. Where "none" is indicated, no structures, vehicular traffic, or other surcharges can occur within a zone defined by an imaginary plane extending from the back of the base rock at an inclination of 1V:1.5H.

__________ surcharge of ___ KPa located ____ from back face of rockery.



BASE ROCK PLAN VIEW
See Note 2

NOT TO SCALE

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

METRIC SPECIAL

ROCKERY
TYPICAL SECTIONS & DETAILS
Sheet 1 of 2

SPECIAL
M252-A

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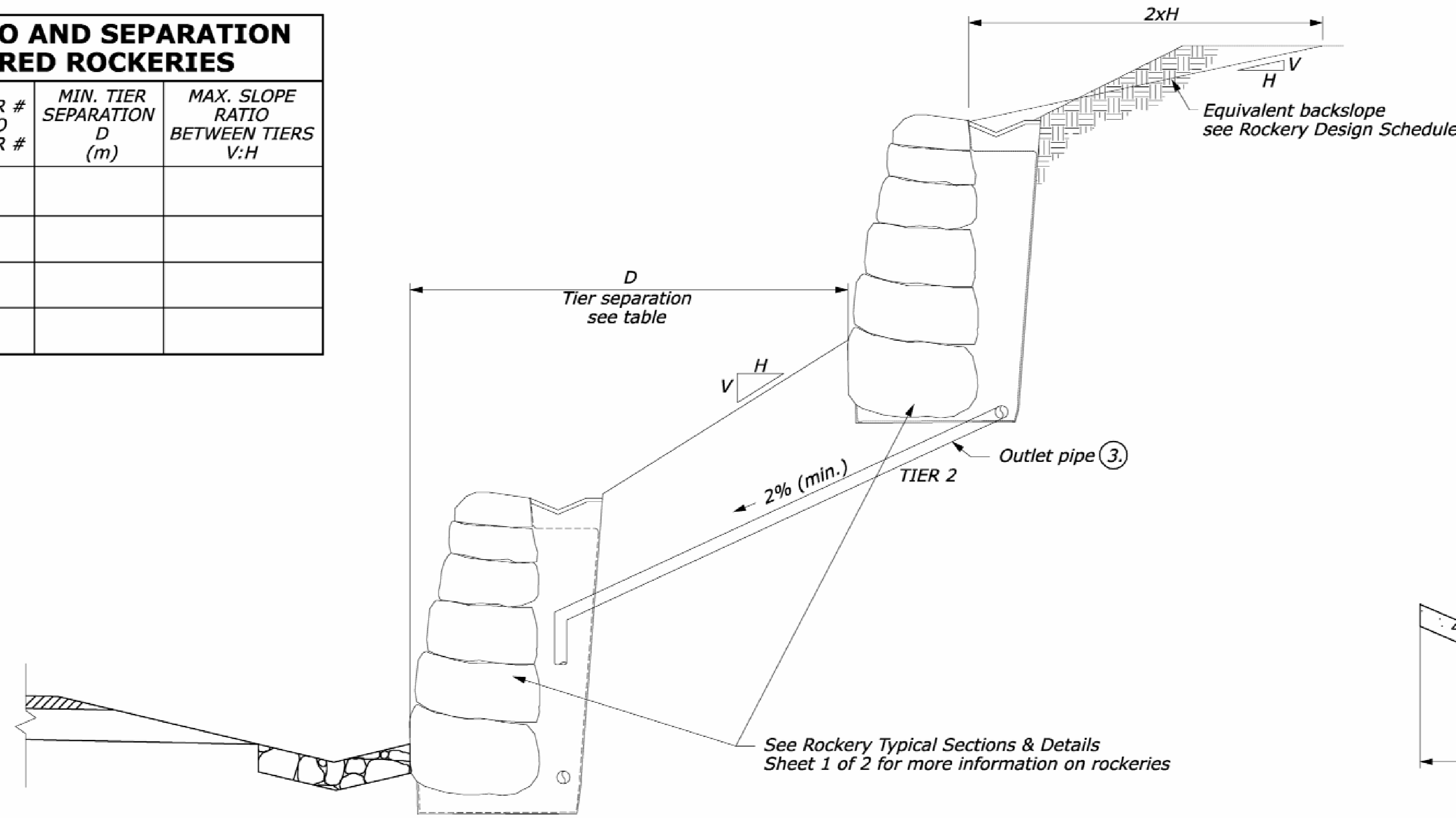
REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS

SLOPE RATIO AND SEPARATION FOR TIERED ROCKERIES

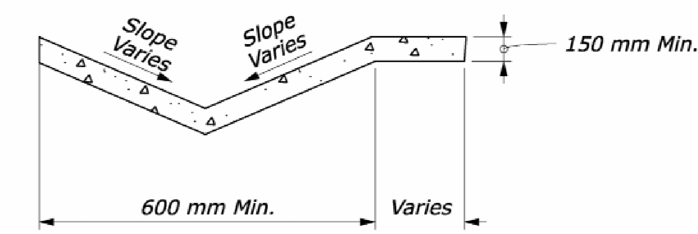
STATION		TIER # TO TIER #	MIN. TIER SEPARATION D (m)	MAX. SLOPE RATIO BETWEEN TIERS V:H
BEGIN	END			



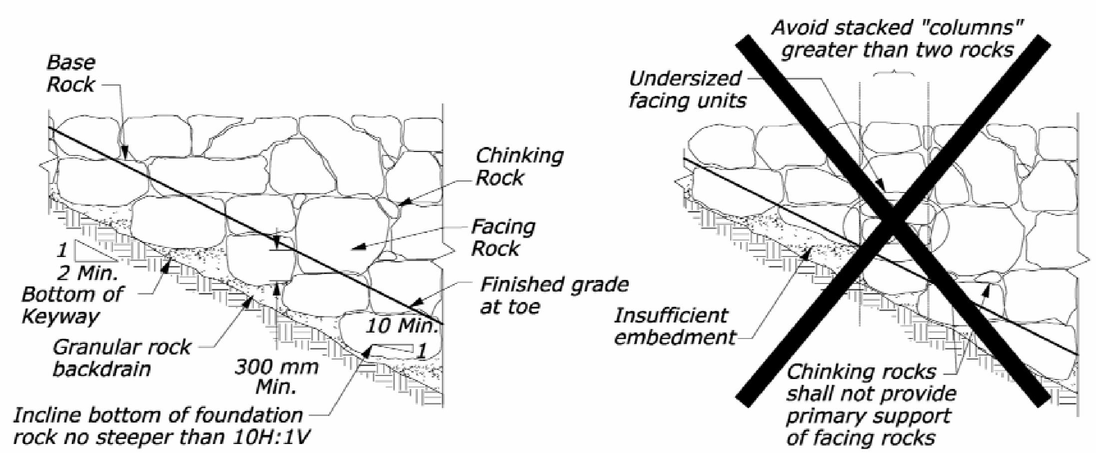
ROCKERY WITH TIERS TYPICAL SECTION

NOTE:

1. See Rockery Typical Sections & Details Sheet 1 of 2.
2. Construct riprap rundown at ends of rockery for catchment of V-ditch drainage.
3. Install 100 mm diameter solid outlet pipe at low points in the rockery and at 30 m (max.) spacing. Do not connect to drainage system for lower tier. Drainage systems for the upper and lower tiers outlet independently.

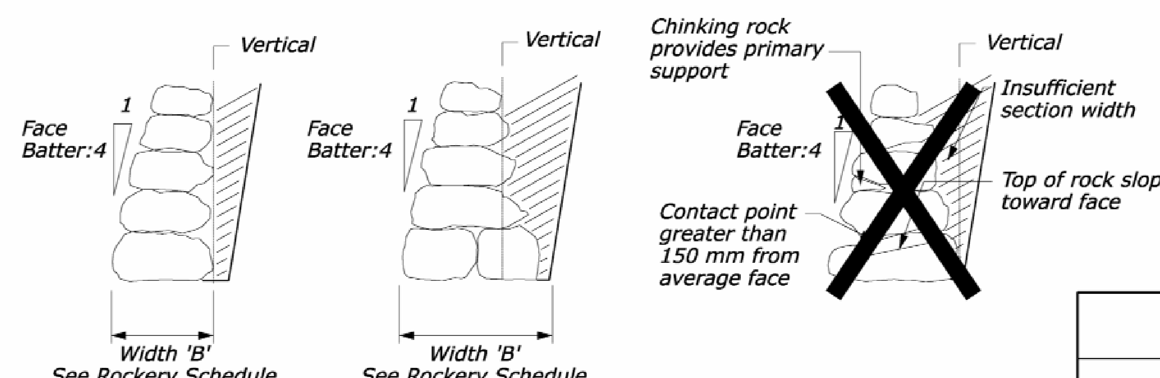


CONCRETE V-DITCH DETAIL



CORRECT INCORRECT

PARTIAL TYPICAL PROFILE



CORRECT CORRECT INCORRECT

SECTION PROPERTIES

NOT TO SCALE

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

METRIC SPECIAL

ROCKERY
TYPICAL SECTIONS & DETAILS
Sheet 2 of 2

SPECIAL
M252-A

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C. Calculation

1. Partial factors of safety (according to Eurocode 7)

2. Comparative calculations

3. Sample calculations

C.1 Partial factors of safety (according to Eurocode 7)

Structural stability (STR)

Demands		Symbol	Value		
Length of time	Condition		BS 1	BS 2	BS 3
permanent	adverse	γ_G	1,35	1,20	1,00
	convenient	γ_G	1,00	1,00	1,00
temporary	adverse	γ_Q	1,50	1,30	1,00
	convenient	γ_Q	0	0	0

Partial factors of safety for Demands ($\boldsymbol{\gamma_E}$) [b]

Soil parameters	Symbol	Value
effective friction angle $\tan\varphi$	$\gamma_{\varphi'}$	1,00
effective cohesion	$\gamma_{c'}$	1,00
undrained shear strength	γ_{cu}	1,00
uniaxial comprehensive strength	γ_{qu}	1,00
specific weight	γ_{γ}	1,00

Partial factors of safety for soil parameters ($\boldsymbol{\gamma_M}$) [b]

Resistance	Symbol	Value		
		BS 1	BS 2	BS 3
bearing capacity	$\gamma_{R;v}$	1,40	1,30	1,20
sliding	$\gamma_{R;h}$	1,10	1,10	1,10
earth resistance	$\gamma_{R;e}$	1,40	1,30	1,20

Partial factors of safety for resistance of retaining structures ($\boldsymbol{\gamma_R}$) [b]

Total Stability (GEO)

Loads		**Symbol**	**Value**		
Length of time	**Condition**		**BS 1**	**BS 2**	**BS 3**
permanent	adverse	γ_G	1,00	1,00	1,00
	convenient	γ_G	1,00	1,00	1,00
temporary	adverse	γ_Q	1,10	1,10	1,10
	convenient	γ_Q	0	0	0

Partial factors of safety for Loads ($\boldsymbol{\gamma_F}$) [b]

Soil parameters	**Symbol**	**Value for consequence class**								
		CC 1			**CC 2**			**CC 3**		
		Design situation			**Design situation**			**Design situation**		
		BS 1	**BS 2**	**BS 3**	**BS 1**	**BS 2**	**BS 3**	**BS 1**	**BS 2**	**BS 3**
effective friction angle $\tan\varphi$	$\gamma_{\varphi'}$	1,10	1,05	1,00	1,15	1,10	1,05	1,30	1,20	1,10
effective cohesion	$\gamma_{c'}$	1,10	1,05	1,00	1,15	1,10	1,05	1,30	1,20	1,10
specific weight	γ_γ	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
undrained shear strength	γ_{cu}	1,20	1,15	1,10	1,25	1,20	1,15	1,40	1,30	1,20
uniaxial comprehensive strength	γ_{qu}	1,20	1,15	1,10	1,25	1,20	1,15	1,40	1,30	1,20

Partial factors of safety for soil parameters ($\boldsymbol{\gamma_M}$) [b]

Resistance	**Symbol**	**Value for all CC, BS**
earth resistance	$\gamma_{R;e}$	1,00
Anchor resistance and other stabilising elements	γ_a	1,00

Partial factors of safety for resistance of slope stability verification ($\boldsymbol{\gamma_R}$) [b]

Static equilibrium (EQU)

Loads		Symbol	Value
permanent	adverse	$\gamma_{G;dst}$	1,10
	convenient	$\gamma_{G:stb}$	0,90
temporary	adverse	$\gamma_{Q:dst}$	1,50
	convenient	$\gamma_{Q:stb}$	0

Partial factors of safety for loads ($\boldsymbol{\gamma_F}$) [b]

Soil parameters	Symbol	Value
effective friction angle $\tan\varphi$	$\gamma_{\varphi'}$	1,25
effective cohesion	$\gamma_{c'}$	1,25
undrained shear strength	γ_{cu}	1,40
uniaxial comprehensive strength	γ_{qu}	1,40
specific weight	γ_{γ}	1,00

Partial factors of safety for soil parameters ($\boldsymbol{\gamma_M}$) [b]

C.2 Comparative calculations

Comparative Calcalution A

Calculation: EC 7, BS1, CC1
Earth pressure after DIN 5085:2011
Ea and Ep applied

Software: GGU Gabion 5
Version 5.13, 21.01.2013

number	adjustable parameters									results			
	soil					blocks							
	γ [kN/m³]	$\varphi [^\circ]$	δ	δ / φ	c [kN/m²]	joint thickness [m]	block size h*w [m]	μ	γ [kN/m³]	η(sliding) foot	η(bearing capacity)	η(EQU)	η(sliding) blocks
1	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**0,5**	20	0,790	0,743	0,499	0,859
2	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**0,75**	20	0,790	0,743	0,499	0,573
3	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**1**	20	0,790	0,743	0,499	0,430
4	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**1,25**	20	0,790	0,743	0,499	0,344
5	20	30	2/3φ	0,667	0	**0,01**	0,5*1,0	0,75	20	0,786	0,753	0,495	0,569
6	20	30	2/3φ	0,667	0	**0,05**	0,5*1,0	0,75	20	0,792	0,670	0,450	0,565
7	20	30	2/3φ	0,667	0	**0,1**	0,5*1,0	0,75	20	0,801	0,599	0,407	0,561
8	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**0,5**	20	0,809	0,540	0,366	0,833
9	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**0,75**	20	0,809	0,540	0,366	0,555
10	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**1**	20	0,809	0,540	0,366	0,416
11	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**1,25**	20	0,809	0,540	0,366	0,333
12	20	30	φ	**1**	0	0,001	0,5*1,0	**0,5**	20	0,692	0,836	0,585	0,770
13	20	30	φ	**1**	0	0,001	0,5*1,0	**0,75**	20	0,692	0,836	0,585	0,513
14	20	30	φ	**1**	0	0,001	0,5*1,0	**1**	20	0,692	0,836	0,585	0,385
15	20	30	φ	**1**	0	0,001	0,5*1,0	**1,25**	20	0,692	0,836	0,585	0,308
16	20	30	φ	**1**	0	**0,01**	0,5*1,0	0,75	20	0,688	0,833	0,584	0,509
17	20	30	φ	**1**	0	**0,05**	0,5*1,0	0,75	20	0,695	0,763	0,535	0,510
18	20	30	φ	**1**	0	**0,1**	0,5*1,0	0,75	20	0,707	0,685	0,471	0,512
19	20	30	φ	**1**	0	**0,15**	0,5*1,0	**0,5**	20	0,717	0,622	0,429	0,770
20	20	30	φ	**1**	0	**0,15**	0,5*1,0	**0,75**	20	0,717	0,622	0,429	0,513
21	20	30	φ	**1**	0	**0,15**	0,5*1,0	**1**	20	0,717	0,622	0,429	0,385
22	20	30	φ	**1**	0	**0,15**	0,5*1,0	**1,25**	20	0,717	0,622	0,429	0,308
23	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**18**	0,866	0,698	0,477	0,628
24	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**22**	0,726	0,793	0,526	0,527
25	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**24**	0,672	0,846	0,548	0,487
26	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**26**	0,625	0,902	0,568	0,453

Comparative Calcalution B

<u>Calculation:</u> EC 7, BS1, CC1
Earth pressure after DIN 5085:2011
Ea applied
Ep limited to 0,001kN/m²

<u>Software:</u> GGU Gabion 5
Version 5.13, 21.01.2013

number	adjustable parameters									results			
	soil					blocks							
	γ [kN/m³]	$\varphi [°]$	δ	δ / φ	c [kN/m²]	joint thickness [m]	block size h*w [m]	μ	γ [kN/m³]	η(sliding) foot	η(bearing capacity)	η(EQU)	η(sliding) blocks
1	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**0,5**	20	0,797	0,810	0,499	0,920
2	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**0,75**	20	0,797	0,810	0,499	0,613
3	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**1**	20	0,797	0,810	0,499	0,460
4	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**1,25**	20	0,797	0,810	0,499	0,368
5	20	30	2/3φ	0,667	0	**0,01**	0,5*1,0	0,75	20	0,793	0,804	0,495	0,611
6	20	30	2/3φ	0,667	0	**0,05**	0,5*1,0	0,75	20	0,801	0,745	0,450	0,617
7	20	30	2/3φ	0,667	0	**0,1**	0,5*1,0	0,75	20	0,812	0,681	0,407	0,625
8	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**0,5**	20	0,823	0,627	0,366	0,950
9	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**0,75**	20	0,823	0,627	0,366	0,633
10	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**1**	20	0,823	0,627	0,366	0,475
11	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**1,25**	20	0,823	0,627	0,366	0,380
12	20	30	φ	**1**	0	0,001	0,5*1,0	**0,5**	20	0,698	0,882	0,585	0,806
13	20	30	φ	**1**	0	0,001	0,5*1,0	**0,75**	20	0,698	0,882	0,585	0,537
14	20	30	φ	**1**	0	0,001	0,5*1,0	**1**	20	0,698	0,882	0,585	0,403
15	20	30	φ	**1**	0	0,001	0,5*1,0	**1,25**	20	0,698	0,882	0,585	0,322
16	20	30	φ	**1**	0	**0,01**	0,5*1,0	0,75	20	0,694	0,881	0,584	0,534
17	20	30	φ	**1**	0	**0,05**	0,5*1,0	0,75	20	0,702	0,814	0,536	0,541
18	20	30	φ	**1**	0	**0,1**	0,5*1,0	0,75	20	0,716	0,741	0,473	0,551
19	20	30	φ	**1**	0	**0,15**	0,5*1,0	**0,5**	20	0,728	0,681	0,429	0,840
20	20	30	φ	**1**	0	**0,15**	0,5*1,0	**0,75**	20	0,728	0,681	0,429	0,560
21	20	30	φ	**1**	0	**0,15**	0,5*1,0	**1**	20	0,728	0,681	0,429	0,420
22	20	30	φ	**1**	0	**0,15**	0,5*1,0	**1,25**	20	0,728	0,681	0,429	0,336
23	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**18**	0,874	0,771	0,477	0,673
24	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**22**	0,732	0,856	0,526	0,563
25	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**24**	0,677	0,907	0,548	0,521
26	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**26**	0,629	0,960	0,568	0,484

Comparative Calcalution C

Calculation: EC 7, BS1, CC1
Earth pressure after DIN 5085:2011
E0 applied
Ep limited to 0,001kN/m²

Software: GGU Gabion 5
Version 5.13, 21.01.2013

number	adjustable parameters									results			
	soil					blocks							
	γ [kN/m³]	$\varphi [°]$	δ	δ/φ	c [kN/m²]	joint thickness [m]	block size h*w [m]	μ	γ [kN/m³]	η(sliding) foot	η(bearing capacity)	η(EQU)	η(sliding) blocks
1	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**0,5**	20	1,311	2,160	0,399	1,345
2	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**0,75**	20	1,311	2,160	0,399	0,897
3	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**1**	20	1,311	2,160	0,399	0,673
4	20	30	2/3φ	0,667	0	0,001	0,5*1,0	**1,25**	20	1,311	2,160	0,399	0,538
5	20	30	2/3φ	0,667	0	**0,01**	0,5*1,0	0,75	20	1,306	2,127	0,390	0,894
6	20	30	2/3φ	0,667	0	**0,05**	0,5*1,0	0,75	20	1,351	2,389	0,349	0,924
7	20	30	2/3φ	0,667	0	**0,1**	0,5*1,0	0,75	20	1,412	2,864	0,387	0,966
8	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**0,5**	20	1,472	3,504	0,492	1,511
9	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**0,75**	20	1,472	3,504	0,492	1,007
10	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**1**	20	1,472	3,504	0,492	0,756
11	20	30	2/3φ	0,667	0	**0,15**	0,5*1,0	**1,25**	20	1,472	3,504	0,492	0,604
12	20	30	φ	**1**	0	0,001	0,5*1,0	**0,5**	20	1,182	1,538	0,469	1,214
13	20	30	φ	**1**	0	0,001	0,5*1,0	**0,75**	20	1,182	1,538	0,469	0,809
14	20	30	φ	**1**	0	0,001	0,5*1,0	**1**	20	1,182	1,538	0,469	0,607
15	20	30	φ	**1**	0	0,001	0,5*1,0	**1,25**	20	1,182	1,538	0,469	0,485
16	20	30	φ	**1**	0	**0,01**	0,5*1,0	0,75	20	1,172	1,498	0,456	0,802
17	20	30	φ	**1**	0	**0,05**	0,5*1,0	0,75	20	1,209	1,622	0,410	0,827
18	20	30	φ	**1**	0	**0,1**	0,5*1,0	0,75	20	1,216	1,839	0,364	0,836
19	20	30	φ	**1**	0	**0,15**	0,5*1,0	**0,5**	20	1,311	2,097	0,318	1,345
20	20	30	φ	**1**	0	**0,15**	0,5*1,0	**0,75**	20	1,311	2,097	0,318	0,897
21	20	30	φ	**1**	0	**0,15**	0,5*1,0	**1**	20	1,311	2,097	0,318	0,673
22	20	30	φ	**1**	0	**0,15**	0,5*1,0	**1,25**	20	1,311	2,097	0,318	0,538
23	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**18**	1,427	2,738	0,389	0,977
24	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**22**	1,212	1,807	0,407	0,829
25	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**24**	1,127	1,575	0,421	0,771
26	20	30	2/3φ	0,667	0	0,001	0,5*1,0	0,75	**26**	1,053	1,415	0,438	0,721

C.3 Sample calculations

Model Rock Wall

Ea, Ep

Norm: EC 7
Berechnungsgrundlagen:
Aktiver Erddruck nach: DIN 4085
Ersatzerddruck-Beiwert [-] = 0.200
Passiver Erddruck nach: DIN 4085:2011
γ_G = 1.35
γ_Q = 1.50
γ_{Ep} = 1.40 (Gleiten)
Faktor(Ep) = 0.50 (Grundbruch/Stützlinie)
Grenzzustand EQU:
$\gamma_{G,dst}$ = 1.10
$\gamma_{G,stb}$ = 0.90
$\gamma_{Q,dst}$ = 1.50

Setzungen:
Steifemodulprofil und
Setzungsanteile in den kennzeichnenden Punkten
infolge Gesamtlasten

Tiefe [m u. GS]	Es [MN/m²]	s(links) [cm]	s(rechts) [cm]
> 0.00	20.00	0.28	0.43

Grenztiefe mit p = 20.0 %
Grenztiefe = 2.09 m u. GS
a = 100.00 m

b = 1.00 m
sigma (links) = 25.04 kN/m²
sigma (rechts) = 109.30 kN/m²
Setzungen in den kennzeichnenden Punkten:
links: s = 0.28 cm
rechts: s = 0.43 cm

Bemessung:
Exzentrizität e(Fuß) = 0.105 m
Maßgebend: g+q
$V_{Fuß}$ = 67.17 kN/m (mit $E_{pv,mob\,k}$)
$H_{Fuß}$ = 15.70 kN/m (mit $E_{ph,mob\,k}$)
$M_{Fuß}$ = 7.02 kN·m/m (mit $E_{p,mob\,k}$)
$E_{p,mob,k} = 0.50 \cdot E_{p,k}$
$E_{pv,mob\,k}$ = 0.62 kN/m ; $E_{ph,mob\,k}$ = 1.07 kN/m
b = 1.000 m ; a = 100.000 m
b/6 = 0.167 m ; b/3 = 0.333 m
σ_1/σ_2(Fuß) = 25.0 / 109.3 kN/m²

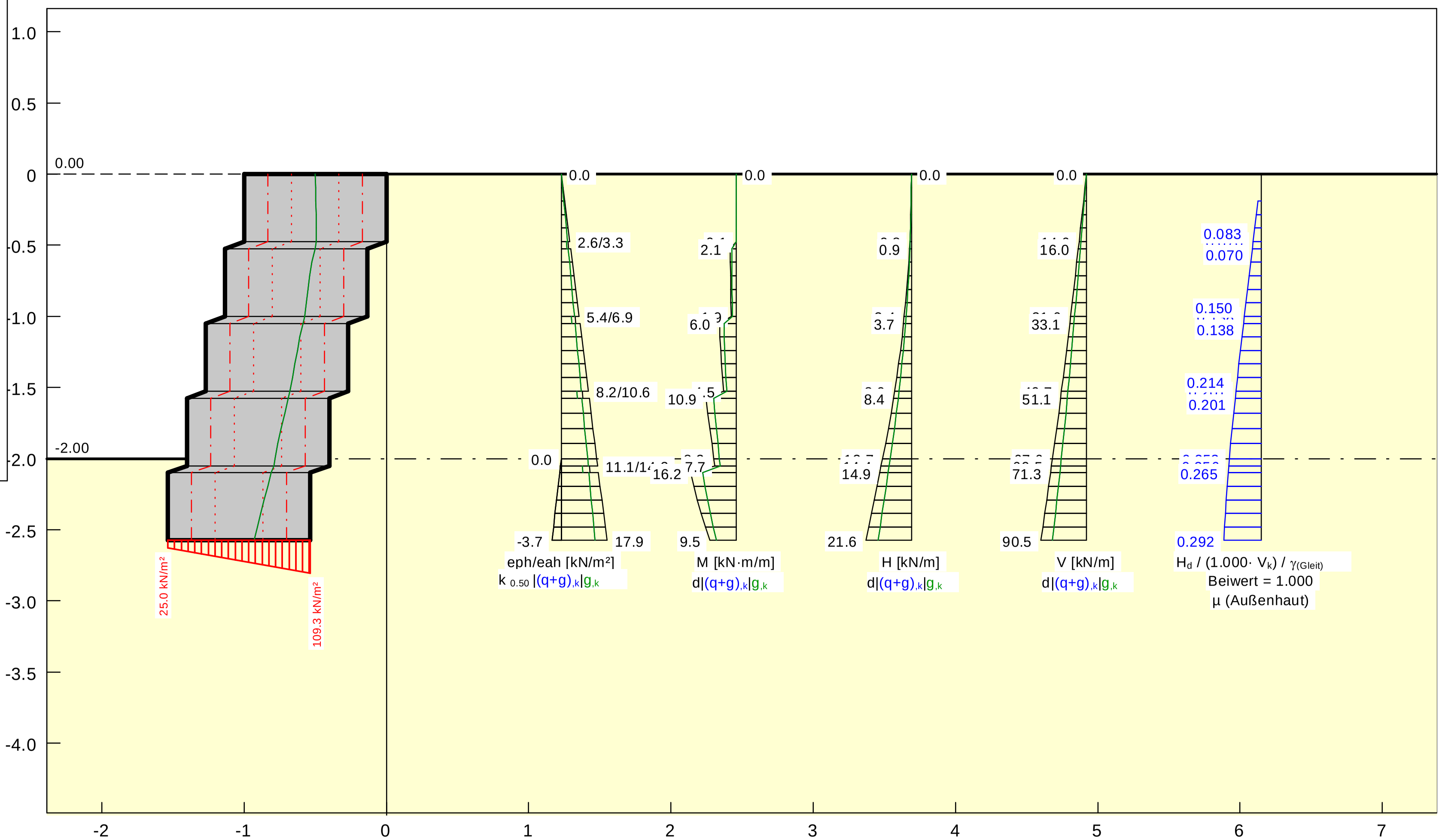
Nachweis EQU:
Tiefe = -2.05 m
$M_{stb} = 52.8 \cdot 1.00 \cdot 0.5 \cdot 0.90 = 23.77$
$M_{dst} = 12.0 \cdot 1.10 = 13.23$
$\mu_{EQU} = 13.23 / 23.77 = 0.557$

Gleitsicherheit ohne Erdwiderstand
$\mu(\text{Gleit}) = H_d / (V_k \cdot \tan(\varphi) / \gamma(\text{Gleit}) + E_{p,d})$ =
22.6 / (67.2 · tan(30.0°) / 1.10 + 0.0) = 0.642

μ(Grundbruch) = 0.81
mit: φ_k = 30.0°; c_k = 0.0 kN/m²
γ_2 = 20.00 kN/m²; $\sigma_ü$ = 11.5 kN/m²

γ = 22.00 kN/m³
E-Modul = $2.500 \cdot 10^{+4}$ kN/m²
Stützlinie liegt zwischen
1. und 2. Kernlinie auf der Erdseite
Kubatur = 2.575 m³/m

1. Kernweite
2. Kernweite
Stützlinie (g+q)
Stützlinie (g)



Boden	γ_k [kN/m³]	γ'_k [kN/m³]	φ_k [°]	$c(a)_k$ [kN/m²]	$c(p)_k$ [kN/m²]	δ/φ aktiv	δ/φ passiv	Bezeichnung
	20.0	10.0	30.0	0.0	0.0	1.000	1.000	Sand

Florian Steiner, BSc.

Model Rock Wall

Ea, Ep limited

Norm: EC 7
Berechnungsgrundlagen:
Aktiver Erddruck nach: DIN 4085
Ersatzerddruck-Beiwert [-] = 0.200
Passiver Erddruck nach: DIN 4085:2011
γ_G = 1.35
γ_Q = 1.50
γ_{Ep} = 1.40 (Gleiten)
Faktor(Ep) = 0.50 (Grundbruch/Stützlinie)
Grenzzustand EQU:
$\gamma_{G,dst}$ = 1.10
$\gamma_{G,stb}$ = 0.90
$\gamma_{Q,dst}$ = 1.50
Passiver Erddruck auf 0.0 kN/m² begrenzt.

Setzungen:
Steifemodulprofil und
Setzungsanteile in den kennzeichnenden Punkten
infolge Gesamtlasten

Tiefe [m u. GS]	Es [MN/m²]	s(links) [cm]	s(rechts) [cm]
> 0.00	20.00	0.28	0.43

Grenztiefe mit p = 20.0 %
Grenztiefe = 2.08 m u. GS
a = 100.00 m

b = 1.00 m
sigma (links) = 23.81 kN/m²
sigma (rechts) = 109.30 kN/m²
Setzungen in den kennzeichnenden Punkten:
links: s = 0.28 cm
rechts: s = 0.43 cm

Bemessung:
Exzentrizität e(Fuß) = 0.107 m
Maßgebend: g
$V_{Fuß}$ = 66.55 kN/m (mit $E_{pv,mob\,k}$)
$H_{Fuß}$ = 16.77 kN/m (mit $E_{ph,mob\,k}$)
$M_{Fuß}$ = 7.12 kN·m/m (mit $E_{p,mob\,k}$)
$E_{p,mob,k} = 0.50 \cdot E_{p,k}$
$E_{pv,mob\,k}$ = 0.00 kN/m ; $E_{ph,mob\,k}$ = 0.00 kN/m
b = 1.000 m ; a = 100.000 m
b/6 = 0.167 m ; b/3 = 0.333 m
σ_1/σ_2(Fuß) = 23.8 / 109.3 kN/m²

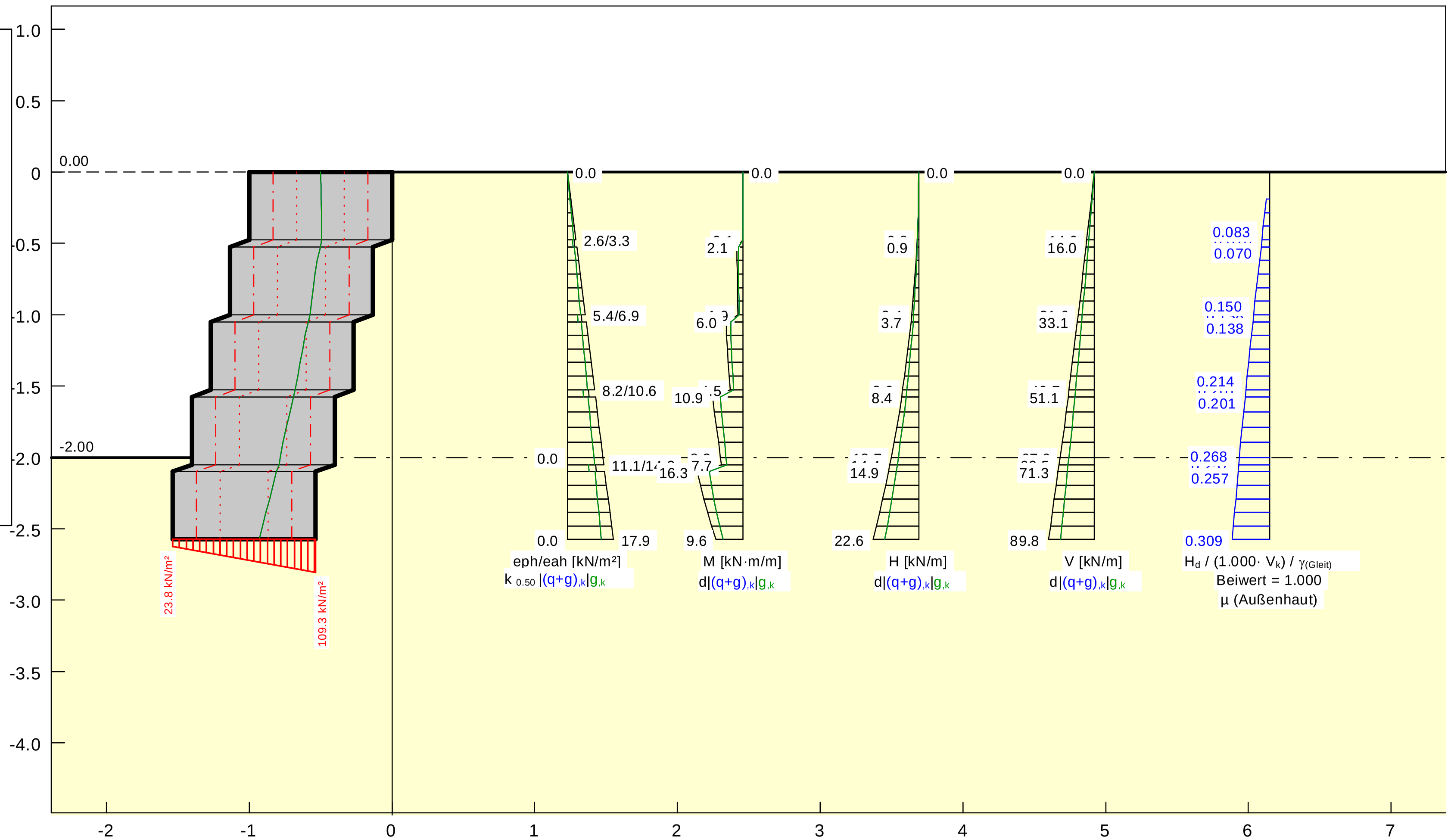
Nachweis EQU:
Tiefe = -2.05 m
$M_{stb} = 52.8 \cdot 1.00 \cdot 0.5 \cdot 0.90 = 23.76$
$M_{dst} = 12.0 \cdot 1.10 = 13.24$
μ_{EQU} = 13.24 / 23.76 = 0.557

Gleitsicherheit ohne Erdwiderstand
$\mu(\text{Gleit}) = H_d / (V_k \cdot \tan(\varphi) / \gamma(\text{Gleit}) + E_{p,d})$ =
22.6 / (66.6 · tan(30.0°) / 1.10 + 0.0) = 0.648

μ(Grundbruch) = 0.86
mit: φ_k = 30.0°; c_k = 0.0 kN/m²
γ_2 = 20.00 kN/m²; $\sigma_ü$ = 11.5 kN/m²

γ = 22.00 kN/m³
E-Modul = $2.500 \cdot 10^{+4}$ kN/m²
Stützlinie liegt zwischen
1. und 2. Kernlinie auf der Erdseite
Kubatur = 2.575 m³/m

1. Kernweite
2. Kernweite
Stützlinie (g+q)
Stützlinie (g)



Boden	γ_k [kN/m³]	γ'_k [kN/m³]	φ_k [°]	$c(a)_k$ [kN/m²]	$c(p)_k$ [kN/m²]	δ/φ aktiv	δ/φ passiv	Bezeichnung
	20.0	10.0	30.0	0.0	0.0	1.000	1.000	Sand

Florian Steiner, BSc.

Model Rock Wall

E0, Ep limited

Norm: EC 7
Berechnungsgrundlagen:
Aktiver Erddruck nach: DIN 4085
Ersatzerddruck-Beiwert [-] = 0.200
Passiver Erddruck nach: DIN 4085:2011
γ_G = 1.35
γ_{E0g} = 1.20
γ_Q = 1.50
γ_{Ep} = 1.40 (Gleiten)
Faktor(Ep) = 0.50 (Grundbruch/Stützlinie)
Grenzzustand EQU:
$\gamma_{G,dst}$ = 1.10
$\gamma_{G,stb}$ = 0.90
$\gamma_{Q,dst}$ = 1.50
Passiver Erddruck auf 0.0 kN/m² begrenzt.
Erdruhedruck

Setzungen:
Steifemodulprofil und
Setzungsanteile in den kennzeichnenden Punkten
infolge Gesamtlasten

Tiefe [m u. GS]	Es [MN/m²]	s(links) [cm]	s(rechts) [cm]
> 0.00	20.00	0.43	0.41

Grenztiefe mit p = 20.0 %
Grenztiefe = 2.29 m u. GS
a = 100.00 m

b = 1.00 m
sigma (links) = 81.55 kN/m²
sigma (rechts) = 69.85 kN/m²
Setzungen in den kennzeichnenden Punkten:
links: s = 0.43 cm
rechts: s = 0.41 cm

Bemessung:
Exzentrizität e(Fuß) = -0.013 m
Maßgebend: g
$V_{Fuß}$ = 75.70 kN/m (mit $E_{pv,mob\,k}$)
$H_{Fuß}$ = 33.15 kN/m (mit $E_{ph,mob\,k}$)
$M_{Fuß}$ = -0.97 kN·m/m (mit $E_{p,mob\,k}$)
$E_{p,mob,k} = 0.50 \cdot E_{p,k}$
$E_{pv,mob\,k}$ = 0.00 kN/m ; $E_{ph,mob\,k}$ = 0.00 kN/m
b = 1.000 m ; a = 100.000 m
b/6 = 0.167 m ; b/3 = 0.333 m
σ_1/σ_2(Fuß) = 81.5 / 69.9 kN/m²

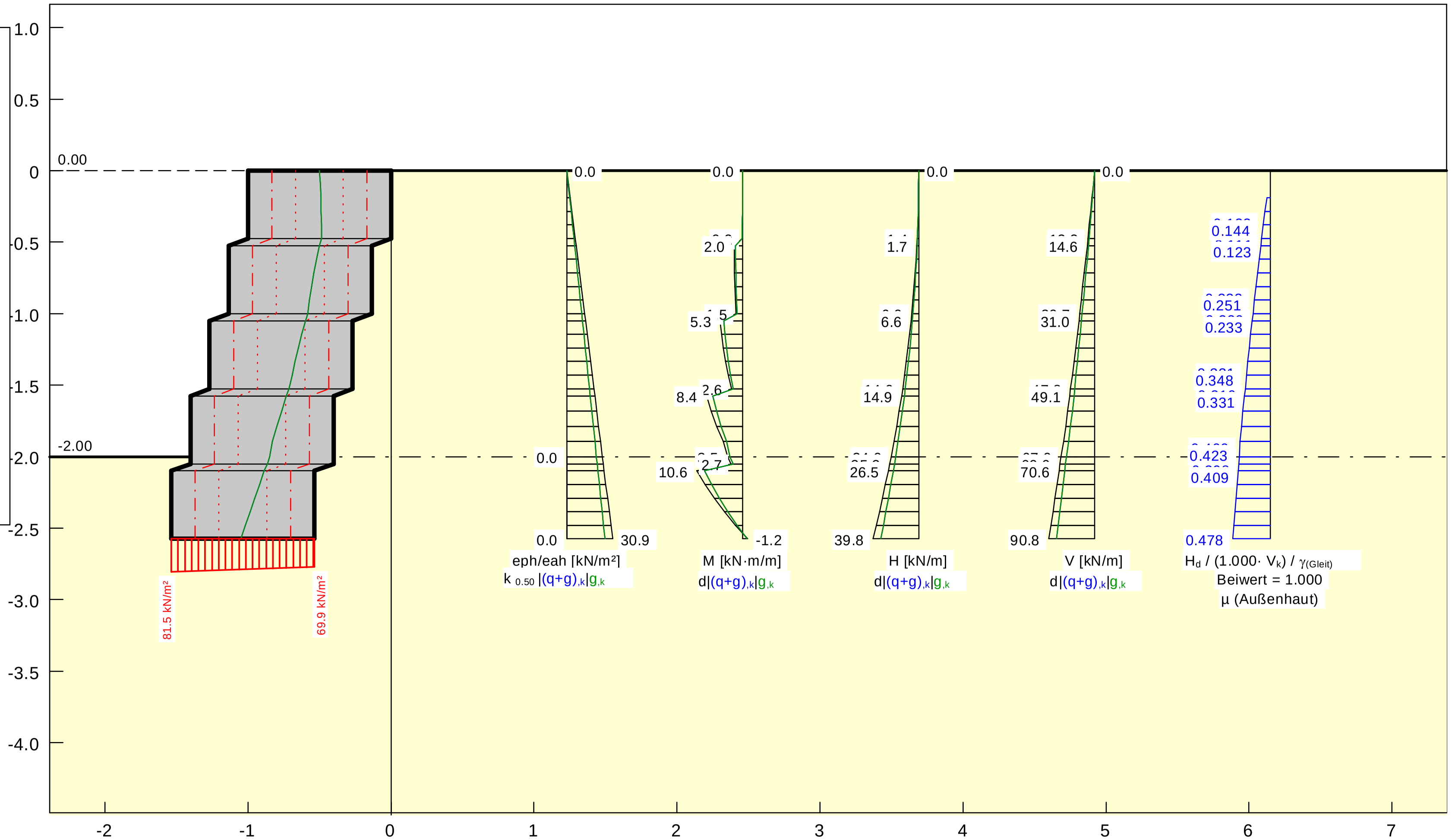
Nachweis EQU:
Tiefe = -1.52 m
$M_{stb} = 40.9 \cdot 1.00 \cdot 0.5 \cdot 0.90 = 18.42$
$M_{dst} = 7.0 \cdot 1.10 = 7.68$
μ_{EQU} = 7.68 / 18.42 = 0.417

Gleitsicherheit ohne Erdwiderstand
μ(Gleit) = H_d / ($V_k \cdot \tan(\varphi)$ / γ(Gleit) + $E_{p,d}$) =
44.8 / (75.7 · tan(30.0°) / 1.10 + 0.0) = 1.126

μ(Grundbruch) = 1.43
mit: φ_k = 30.0°; c_k = 0.0 kN/m²
γ_2 = 20.00 kN/m³; $\sigma_ü$ = 11.5 kN/m²

γ = 22.00 kN/m³
E-Modul = $2.500 \cdot 10^{+4}$ kN/m²
Stützlinie liegt zwischen
1. und 2. Kernlinie auf der Erdseite
Kubatur = 2.575 m³/m

1. Kernweite
2. Kernweite
Stützlinie (g+q)
Stützlinie (g)



Boden	γ_k [kN/m³]	γ'_k [kN/m³]	φ_k [°]	c(a)$_k$ [kN/m²]	c(p)$_k$ [kN/m²]	δ/φ aktiv	δ/φ passiv	Bezeichnung
	20.0	10.0	30.0	0.0	0.0	1.000	1.000	Sand

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