Impact of Drilling in Embankment Dams

A Comparative study between Water Powered DTH Hammer Drilling Technology and Hydraulic Top Hammer Drills

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Abstract

Large geotechnical structures such as embankment dams and tailings dams are subject to high safety requirements. One of the key requirements is long-term stability in order to avoid incidents and failures as well as to ensure environmental safety during and after the operational phases. To ensure security, the instrumentation of existing embankment dams and tailings dams, using drilling technologies, is necessary for surveillance. Drilling in the structures of dams or in their foundations, however, always entails certain risks. Therefore, the selection of drilling technologies must be carried out carefully, taking into account the condition and sensitivity of each dam section. A continuous evaluation and development of existing drilling requirements is an important safety aspect.

The main objective of this comparative study, implemented in collaboration between LKAB, Wassara AB, Sweco Infrastructure AB and Luleå University of Technology in Malmberget, Sweden, is to analyse the influence on surrounding soil of a water powered DTH hammer system and top hammer drillings with different setups during the drilling process. Covering an area of 800 m², an artificial dam with a height of 3 m was built and compacted in layers. The soil used was characterised as gravelly sand. Drillings were done vertical and inclined. Weight sounding tests before and after drilling, excavation of boreholes as well as soil sampling for further laboratory analysis have all been conducted. Complementing this research project, one vertical borehole with a depth of 30 m has been drilled with Wassara’s Lost Hammer concept (LHC) in the tailings dam of LKAB in Malmberget.

This study concludes by stating that, for the analysed drilling formation, the average radius of influence on soil was identical (0.49 m along the borehole axis) for both the water powered DTH and the hydraulic top hammer system with a pre-drilling protective casing. Top hammer drillings in combination with a casing drilling system (Symmetrix) indicate an increased average zone of influence of 0.72 m around the boreholes. Measurements collected while drilling, correlated with laboratory analyses and weight sounding tests, reveal that the rearrangement of soil particles depends on the interaction of applied down-thrust, the vibration of the drill string, the amount of drained flushing water towards the adjacent soil and existing pressure conditions within the embankment structure.
Zusammenfassung

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1 Introduction

The instrumentation of embankment dams as well as tailings dams for different surveillance purposes is necessary for an early detection of structural changes in order to prevent incidents and failures. The installation of stand pipes in order to measure the seepage line or inclinometer devices to measure lateral displacements of dam shoulders, for instance, during operation of dams is possible by the use of drilling technology.

Drilling applications in existing dam structures or in the foundation, however, represent always risks. Therefore, the selection between different existing drilling technologies must be carried out carefully, taking into account the condition and sensitivity of different dam sections. Over the recent decades, several national authorities and dam operators established different regulation concerning drilling operations. A continuous evaluation and development of drilling requirements is an important safety aspect.

In contrast to the development efforts in the area of enhanced and efficient drilling technologies, the share of research for the investigation of influences of drilling technologies on different soil structures is still limited. Thus, this research study, implemented in collaboration between LKAB, Wassara AB, Sweco Infrastructure AB and Luleå University of Technology in Malmberget, Sweden, can be considered as an important first research step of a planned comprehensive drill research program in Sweden.

The main objective of this comparative study is to analyse the influence of drilling technologies on gravely sand, a sensitive soil characteristic to the drilling process, in connection with its use in embankment dam structures. In this context, a direct comparison between hydraulic top hammer drilling and a water powered DTH hammer, used in connection with Wassara’s Lost Hammer Concept (LHC) for dam instrumentation, has been implemented within a reference project. In addition to a detailed technical explanation of the top hammer system and the LHC, the design and construction characteristics of embankment dams and tailings dams as well as general drilling risks are comprehensively discussed within the thesis.
2 Design and Construction of Embankment Dams

Embarkment dams, usually constructed for use as Water Retention Dams (WRD) for several industrial purposes, are divided in two groups depending on main construction material used, i.e. earth-filled and rock-filled embankment dams (Narita 2000). The International Commission on Large Dams (ICOLD) defined an earth-filled dam as an embankment dam, consisting of soil with more than 50 volume-% of fill. In this context soil is understood as clay, silt, sand, or gravel (Kjærnsli et al. 2003). A Rock-fill dam on the other hand, after ICOLD, is described as an embankment dam, consisting of rock with more than 50 volume-% of fill. Thus, rock material is obtained from rock quarry or rock excavation of natural boulders and stones (Kjærnsli et al. 2003). Between 1998 and 2000 approximately 1,000 high dams, defined as dams with a height of more than 15 m after ICOLD, have been constructed. 80 % of them are embankment dams and 20 % concrete dams (Narita 2000). Embankment dams are very common in Norway and Sweden, whereas natural moraine is mainly used in the construction of the impervious embankment core (Kjærnsli et al. 2003). In Figure 1 the year of completion and number of large dams of altogether about 10,000 in Sweden during the 20\textsuperscript{th} century is shown.

![Figure 1: Large hydropower dams in Sweden arranged by year of completion (ICOLD 1998).](image)

Approximately 60 % of Sweden is covered by moraine, defined as carried and deposited material by a glacier (Benckert and Eurenius 2001). After Kjærnsli et al. (2003), moraine is a broadly sorted combination of stones, gravel, sand, silt and clay.

The following sections describe typical embankment dam and construction types and a discussion of general structural elements and their function.
2.1 Types of Embankments Dams

Earth-fill and rock-fill dams are common classifications of embankment dams, as described previously. Furthermore, two types of embankment dams are characterised as “homogeneous embankment” and “zoned embankment”, after Stephens (2010). According to this, a homogeneous earth-fill embankment dam is constructed by one main kind of low permeability material having flat slopes, which provides structural and seepage resistance in combination with a drainage system, see Figure 2 (Narita 2000). Zoned embankment dams usually are constructed as rock-fill dams, on the other hand, they are consisting of several main structural elements as an impervious core, filters, draining and transition zones, and supporting fill. The foundation represents the basic stable level for the construction of an embankment dam.

Nowadays, several construction types are available and some common types are illustrated below from Figure 2 to Figure 5. In general, the embankment dam type is determined by careful consideration concerning the geology and topography conditions on site, environmental and climate restrictions, the availability and quality of construction material, and the operation purpose.

![Figure 2: Cross-section of a homogeneous earth dam (Narita 2000).](image)

Swedish embankment dams are usually constructed as zoned dams with an impermeable central core, see Figure 3, respectively an inclined core, after Figure 4 (RIDAS 2012).
The inclined core variant, for instance, is used in case of a steep inclined dam foundation, where a vertical connection between an impervious foundation and dam core is not possible, and where different construction processes are performed for material insertion (Narita 2000).

Figure 5 shows a rock-fill dam with an impervious facing and a transition downstream. This construction tends to have problems caused by external erosion due to the location of this leak-proof layer. Asphalt concrete and bituminous sealings are common used facing materials (Narita 2000).
2.2 Structural Components

Within this section the emphases in connection with embankment dams are discussed according to the following list on the basis of Figure 6:

- foundation,
- impervious core,
- filter and drain including filter criteria and layer construction,
- and supporting fill.

Further components, illustrated in Figure 6, are explained in section 2.2.5.

Besides the used international English literature, this section considered as well the Swedish guideline RIDAS (2012), known as “Hydropower Industry Dam Safety Guidelines” published by Svensk Energi AB.

![Cross-section of a typical embankment dam with component terms](image)

*Figure 6: Cross-section of a typical embankment dam with component terms (FEMA 2011).*
2.2.1 Foundation

The foundation of an embankment dam is in general depending on local topography, loads of the embankment dam itself and the reservoir, and the permeability conditions of the overburden respectively the foundation rock (Kjærnsli et al. 2003). Consequently, the foundation and overlaying dam elements must form an effective and stable unit. Furthermore, it is important that the selected dam design and monitoring system, for seepage and pore pressure analyses in the foundation, are suitable for the specific ground conditions (Kjærnsli et al. 2003).

In the case of a thin cover by overburden or an exposed rock foundation surface, the rock may be analysed by geological mapping (Kjærnsli et al. 2003). For more details it is common, after Kjærnsli et al. (2003), to perform drillings for core sampling and pumping tests to determine the rock foundation permeability, for instance a Lugeon Test (Figure 7). By using this constant-head-type test, based on boreholes with a diameter 50 to 80 mm, the foundation rock permeability is determined. Thereby, water at constant pressure is injected into an isolated place within the borehole, constructed by a packer system. The pressure by a manometer at the water house and the loss of water over a 5 minute interval by a flowmeter are measured (Kjærnsli et al. 2003). According to Kjærnsli et al. (2003), a repetition of measurement must be performed until two comparable results are given. The given results unit is Lugeon, defined as a water amount loss of 1 liter per minute and metre borehole length with an overpressure of 1 MPa (10 bar) (Quiñones-Rozo 2010).
It can be assumed that the measured water loss (lugeon-value) occur due to an existing crack mesh in the borehole adjacent to the rock. Thus, the lugeon-value can be converted, after Kjærnsli et al. (2003), to the following coefficient of permeability (hydraulic conductivity).

\[ 1 \text{ L (lugeon)} \approx 1.5 \cdot 10^{-7} \text{ m/s} \]

In this context, embankment dam foundations on bedrock can be considered as suitable, if the lugeon values do not overstep 1 to 4 L (Kjærnsli et al. 2003). So, a foundation having slightly higher permeability than a typical moraine core, with a permeability range from \( 10^{-7} \text{ m/s to } 10^{-9} \text{ m/s} \), can be assumed (Kjærnsli et al. 2003). RIDAS recommend a more stringent permeability value of dam foundations. According to this guideline, a value of water loss less than 1.0 l/min, m, MPa 1 L, is appropriate (RIDAS 2012). The target value of less than 1 L is lower than the recommended range of 1-4 L, after Kjærnsli et al. (2003), and may provide a higher foundation quality.

Treatments and preparation of rock foundations and used overburden on site are required, if hydraulic conductivity limits are exceeded. The state of art is to perform grouting drillings until a stable and impervious rock zone is reached respectively down to a depth of two thirds of the dam height (Kjærnsli et al. 2003). In addition, weathered or effected surface rock of unusable quality has to be removed by excavation down to an unweathered rock layer and weak zones must be covered by concrete.
2.2.2 Impervious Core

Following, specification requirements of core material for the use in embankment dams with an impervious core of moraine, common in Sweden and Norway, are described.

The key function of the impermeable core is to ensure an effective sealing against water movements downstream respectively upstream through the embankment body. The impervious core of an embankment dam can be classified into two groups, i.e. with a centrally located vertical core and an inclined core respectively, see section 2.1 (Narita 2000). Regardless of the type, they are usually divided into several horizontal layers as filter, drain, and supporting zones (Kjærnsli et al. 2003).

Most suitable materials for cores are impervious materials with well graded properties that provide appropriate compactions (RIDAS 2012). Thus, the moraine should consist of silty sand with a relatively low proportion of stones (RIDAS 2012). In general, the maximum stone size of layered zones, after Kjærnsli et al. (2003), should be limited to 1/2 to 2/3 of the used layer thickness.

Swedish moraine frequently contains 30 % of fine material, defined after RIDAS (2012), with a particle diameter < 0.06 mm. Moraine, used as core material, should have a fine content at least 15 % of dry weight with regards to material < 20 mm (RIDAS 2012). According to this, the aim is to achieve a hydraulic conductivity of $3 \times 10^{-7}$ m/s or less (RIDAS 2012).

Additionally, the guideline RIDAS recommend maximum water content of 3 % in the moraine above the optimal water content ($W_{opt}$) on the “wet site” to ensure core reliability. In order to ensure settlement safety, the air content ratio of the moraine is limited to 10 % (RIDAS 2012). The observance of a certain compaction degree was used in Sweden before the described regulation on air content ratios. Both requirements exist, after Narita (2000), in order to maintain required strength and permeability in the field. Thus, strength and permeability of compacted soils are related in advance with dry density by use of laboratory test results (Narita 2000).

These laboratory analyses on impervious soils are primarily conducted by Proctor tests where maximum dry density and optimum water content values are measured (FEMA 2011). According to this, the maximum dry density is also known in international literature as “Proctor Density” (FEMA 2011).
The following Figure 8, after RIDAS (2012), illustrates paradigmatic the acceptable zone (green parallelogram) giving a satisfied compaction of the moraine unity on the basis of a moisture-density curve. Gained results are provided on the basis of laboratory tests of samples from the used construction material.

![Figure 8: Acceptable zone of moraine material concerning dry density, water content and air space ratio (RIDAS 2012). Figure modified by Jörg Riechers.](image)

For the compaction of impermeable moraine, vibratory roller machines usually use in contrast to coarse filter and drain material higher frequencies and lower amplitudes (Kjærnsli et al. 2003). A range of 6-8 passes for compaction of each layer with a usual height of 0.5 m, after Kjærnsli et al. (2003), is performed by a 10-15 t vibratory roller. Generally, the compaction work and layer thickness must be coordinated between the existing water content and the content of fines to ensure an effective degree of compaction (Kjærnsli et al. 2003). During construction several samples must be taken for laboratory tests concerning water content, dry density, permeability and grain-size distribution to ensure acceptable quality (Kjærnsli et al. 2003). Such tests are also carried out during construction of other zones of embankment dams, for instance, the filer and drainage zone, despite the number of samples are less.
2.2.3 Filter and Drain

Filters, used to prevent movements of soil particles due to water flow from respectively between zones and foundations of embankment dams, are known historically since hundreds of years (FEMA 2011). In this context, after Kjærnsli et al. (2003), the upstream filter prevents the washing out of fine particles due to waves and rapid drawdowns of the reservoir level. The downstream filter opposite the impervious core filters the existing seepage water (Kjærnsli et al. 2003). Two basic mechanisms generate the particle movements within and below the embankment, after FEMA (2011): internal erosion and backward-erosion (piping). Internal erosion can lead to mobilization of soil particles due to excessive flow rates. On the other hand a backward-erosion, also known as piping effect, is caused by internal erosion and can be explained as a removal of embankment respectively foundation material by flowing fluids through small cross sections “pipes” from the downstream side to the upstream side (FEMA 2011). In this context, a filter protects the embankment dam against leakages that could cause water loss or structural failure. According to FEMA (2011), approximately 50% of all embankment dam failures are caused of a high flow rate of seepage water.

The principle of a self-healing by clogging is described in detail by Figure 9 to Figure 11, after FEMA (2011). Eroded soil from a crack is stopped at the surface of the filter and therefore the flow in the crack is stopped, as shown in Figure 9.

![Figure 9: Movement of soil particles in a cracked zone is stopped at the filter face (FEMA 2011).](image)

High water gradient causes hydraulic fracturing around the crack and filter zone. As a result, a widening of the open pipe in the soil occurs by high water pressures and velocities, as illustrated in Figure 10.
The widening of adjacent soil continues until the hydraulic gradient is reduced, see Figure 11. The produced filter cake has a very low permeability and prevents a soil movement through the filter range. The spaces within the filter zone below the filter cake are open and enable seepage drainage towards the drain system.

A filter, as described before, intercepts the seepage flow (water) from the zone with a high hydrostatic gradient and reduces them to nearly 100 % by using the adjacent drainage zone (FEMA 2011). Typical proceeds of the phreatic surface and the drop down of the hydrostatic head over a certain distance are illustrated in Figure 2 to Figure 5, section 2.1. Filters are designed in combination with an efficient drain system. This system enables the drainage of incoming water through the filter away from the embankment dam, mainly build of particles in size of sand and gravel (Kjærnsli et al. 2003). Thus, the pressure behind the core is reduced.

Following, filter criteria, layer thickness, and compaction specifications of filter materials are explained.
Filter Criteria

Sand, gravel and crushed rock are usually used in a certain distribution for filter constructions (RIDAS 2012). In this context, specific requirements exist concerning the grain size distribution of filter materials. A broad range of filter material is used for dam constructions with different criteria and requirements for each country (Kjærnsli et al. 2003).

After Kjærnsli et al. (2003), it is necessary that the grains resist compaction loads during the construction without breakage to a large scale, which would change the required grain-size distribution and, thus, the properties of the soil. A weather and crumble resistance during the period of dam use must also be guaranteed (Kjærnsli et al. 2003). International literature commonly uses the material, which has to be protected as a starting point (design criteria) for considerations. In practice, that means that a certain level (layer thickness) represents the starting point for the construction of a different dam element, for instance. Thus, the grain-size distribution of the filter material is determined according to the existing respectively planned impervious moraine core. Another starting point, for instance, is the filter material when the transition zone to the adjacent rock-fill zone is determined.

In Table 1, after Kjærnsli et al. (2003), standard criteria for the grain-size distribution of filter materials is presented. Thereby, the letters $D$ represents the filter grain size and $d$ the size of the core material. The suffixes represent the percentage by weight of the material that is finer than the determined size (Kjærnsli et al. 2003).
It is practical to judge the suitability of filter material by its grain-size distribution curve compared to the moraine core (Kjærnsli et al. 2003). After Kjærnsli et al. (2003), ideal materials for filter zones adjacent to a Norwegian moraine core are broadly graded gravelly sands without stones with a size of not more than 20-50 mm. Swedish moraine frequently contains 30% of fine material (RIDAS 2012). Thus, the percentage of fines is defined as the proportion of particles with a grain-size < 0.06 mm from a material with a maximum size smaller than 20 mm. In the majority of cases, the criteria D₁₅ < 0.7 mm is sufficient for efficient downstream filters (RIDAS 2012). In addition it is important to avoid grain separation. This can be ensured by using uniform grain-size distribution curves without sharp turns (RIDAS 2012).

The following Table 2, after RIDAS (2012), presents the maximum stone size of filter material to avoid separation. Further criteria details and information are given in the guideline RIDAS.

### Table 1: Common grain-size distribution of filter material (Kjærnsli, Valstad et al. 2003).

<table>
<thead>
<tr>
<th>Filter material</th>
<th>Uniform material (design criteria)</th>
<th>Well graded material (design criteria)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(d_{60}/d_{10} &lt; 1.5)</td>
<td>(d_{60}/d_{10} &gt; 4)</td>
</tr>
<tr>
<td>Uniform filter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(D_{60}/D_{10} &lt; 1.5)</td>
<td>(5 &lt; d_{50}/d_{50} &lt; 10)</td>
<td>(5 &lt; d_{50}/d_{50} &lt; 15)</td>
</tr>
<tr>
<td>Well graded filter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(D_{60}/D_{10} &gt; 4)</td>
<td>(4 &lt; d_{15}/d_{15} &lt; 6)</td>
<td>(4 &lt; d_{15}/d_{15} &lt; 40)</td>
</tr>
<tr>
<td></td>
<td>(d_{50}/d_{50} &lt; 25)</td>
<td>(d_{50}/d_{50} &lt; 25)</td>
</tr>
<tr>
<td></td>
<td>(d_{15}/d_{85} &lt; 5)</td>
<td>(d_{15}/d_{85} &lt; 5)</td>
</tr>
<tr>
<td>Minimum (D_{10}) of filter material, mm</td>
<td>Maximum (D_{90}) of filter material, mm</td>
<td><em>(D_{90}) = Regulations for filter sand &lt; 20 mm are not necessary</em></td>
</tr>
<tr>
<td>&lt; 0.5</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>0.5-1.0</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>1.0-2.0</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>2.0-5.0</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>5.0-10.0</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>10.0-50.0</td>
<td>60</td>
<td></td>
</tr>
</tbody>
</table>
Layers and Compaction

Filter zones are commonly constructed, according to RIDAS (2012) with a width of 2-3 m. The filter material is usually spread out into several horizontal layers and the compaction should be conducted with heavy vibratory rollers (Kjærnsli et al. 2003). In certain narrow areas walk-behind vibratory plate compactors can be used, after FEMA (2011). Limitations of each compacting machine must be respected, after Kjærnsli et al. (2003), to achieve an optimal compaction rate for each filter layer. The layer thickness is limited due to important requirements concerning density, strength and deformation resistance (Kjærnsli et al. 2003). Therefore, the layer thickness should be equal to the optimal compacting depth to ensure a connection effect on the layer boundary (Kjærnsli et al. 2003). Furthermore, (Kjærnsli, Valstad et al. 2003), recommend the same layer thickness as used in the adjacent core.

In general, the compaction effect depends on the compactor weight, amount and speed of passes, the frequency and amplitude of the produced vibrations by the compactor and soil properties (Kjærnsli et al. 2003). For the compaction of coarse filter materials the vibratory machines use normally low frequencies and greater amplitudes than for cohesive materials (core) (Kjærnsli et al. 2003). A range of 2-4 passes for compaction of each layer, after Kjærnsli et al. (2003), is performed by medium weight rollers (6-8 t) or heavy weight compactors (> 15 t). Thereby, the highest density is obtained at a depth of 0.4-0.5 m below the subsurface (Kjærnsli et al. 2003).

Furthermore, several recommendations are given by RIDAS (2012) for the period of construction. According to that, on site managers should take into consideration:

- prevent soiling (clogging by fine contents) of the filter and drainage zone,
- control and evaluation of filter material,
- ensure the composite between the filter and adjacent material,
- monitoring of layer thickness and uniformity,
- and measurement of layer compaction.
2.2.4 Supporting Fill

The stability of an embankment dam is mainly provided by the thickness and density (mass) of the supporting fill, and the slope inclination (RIDAS 2012). High pore pressure due to an insufficient filter and drainage system and high rates of leakages due to failures in the core, for instance, affects the supporting fill and reduces the embankment dam stability (RIDAS 2012). Therefore, a high resistance against internal erosion, caused by leakage flows, is important. In this context, the leakage is known from literature as design leakage flow (Kjærnsli et al. 2003). Knowledge about the shear strength along the sliding surface, after RIDAS (2012), is also necessary. A critical stability situation emerges, after Kjærnsli et al. (2003), due to rapid drawdown. Thus, the transient pore pressure during the drawdown is equal to the static pressure at maximum water level, including a lower decline of the total stress, and on the other hand the external pressure adapts to the actual drawdown level (Kjærnsli et al. 2003). This topic is considered in Table 3 by safety factors for different loading conditions. According to RIDAS (2012), long time experiences and laboratory test must be taken into account.

Table 3: Overview on safety factors for different loading conditions (RIDAS 2012).

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>Status</th>
<th>Safety factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Finished dam construction, empty impoundment</td>
<td>1,5</td>
</tr>
<tr>
<td>2</td>
<td>Operation of the dam with static water flow through the embankment</td>
<td>1,5</td>
</tr>
<tr>
<td>3</td>
<td>Extreme condition: dam crest overtopping due to high water inflow (higher than design flood)</td>
<td>1,3</td>
</tr>
<tr>
<td>4</td>
<td>After a fast decrease of water level</td>
<td>1,3</td>
</tr>
</tbody>
</table>

Kjærnsli et al. (2003) recommend, as in the prior filter section, that the same layer thickness for the supporting fill should be used as in the adjacent material. Furthermore, a layer thickness of up to 2 m for the supporting fill, compacted by heavy vibratory rollers > 15 t with 6-8 passes, is possible in practice (Kjærnsli et al. 2003).
2.2.5 Further Components of Embankment Dams

In addition to the previously described main components, this subsection describes further typical elements of embankment dams in relation to Figure 6, section 2.2.

The following terms and explanations, unless otherwise marked, refer to the manual in design and construction of filters for embankment dams of the Federal Emergency Management Agency (FEMA 2011).

The CUTOFF TRENCH is located within an excavation in the foundation soil below the impervious core and is integrated in this construction. It is intended to reduce the seepage below the dam construction. An UPSTREAM SHELL respectively the DOWNSTREAM SHELL, also known as supporting fill is as a zone with soil to stabilise and support the core. In case of a rock-fill dam, this section is constructed with blasted rock (Kjærnsli et al. 2003). The internal zone between the filter zone and the supporting fill is called TRANSITION ZONE. The main task is to enable a transition in grain size between two non-compatible material zones, i.e. supporting fill and filters. A filter zone, usually build in a one respectively two layer type is not shown in Figure 6. In the context of rock-fill embankments, normally fine blasted rock is used for construction of the transition zone (Kjærnsli et al. 2003). A CHIMNEY DRAIN, is built mainly of particles in size of gravel, enables the drainage of incoming water through the chimney filter into the horizontal blanket drain system.

Furthermore, this component has a transition function between the chimney filter and the downstream shell. An efficient drainage system is essential, preventing the water table flowing towards the downstream slope (Berghe et al. 2011). The mentioned BLANKET DRAIN zone enables a hydrostatic pressure relief for pervious foundations. Thus, a protection against particle movements should be guaranteed and occurring seepage water can be discharged to the toe drain. A TOE DRAIN therefore is the last element of a drain system that discharge drained water away from the dam toe. The zone between the impervious core and the chimney drain is defined as CHIMNEY FILTER.
This component is primarily consisting of sand-size particles and protects the core from internal erosion and piping. The piping effect has already been described in subsection 2.2.3. This process can lead to critical backward rising erosions, which can be prevented by a cutoff wall (Huber 2008). This CUTOFF WALL, also known as a grout curtain, backfills joints, fractures within the rock to prevent seepage flow and piping. A mixture from soil, cement and bentonite is used for grout curtains. A RELIEF WELL is usually used for collecting seepage water that cannot be collected by toe drains. This collected water can be transferred to a collecting drainage ditch. A DRAINAGE DITCH constructed as an open trench downstream, also collect seepage water. Upstream located IMPERVIOUS BLANKETS are normally integrated into the core and is most often used to seal the foundation soil. This element extends the seepage water way and increments the head loss zone for and embankment dam on permeable foundations. The last explanation of this section is the RIPRAP and BEDDING. The riprap is an external security layer protecting layer on the upstream slope against erosion by reservoir waves and ice movements. Below this layer the bedding is building up a transition zone to the supporting fill protecting against particle movements of the riprap after reservoir drawdown. This transition zone is also designed according to the filter criteria in order to prevent a flush out when the riprap is eroded respectively destroyed.
3 Tailings Dams and Impoundments

The previously described types of embankment dams are usually constructed to be used as WRD for the production of drinking water, as well for irrigation and hydropower applications. Tailings dams, on the other hand, enables tailings impoundments due to its damming function. Geotechnical constructions as tailings impoundments are constructed in layers of soil and rock, usually based on a soil foundation, and retained by embankments, i.e. tailings dams (Bjelkevik 2005b).

The main purpose of a tailings impoundment is to store tailings and process water from industrial activities as mining (Environmental Protection Agency 1994). According to Benckert and Eurenius (2001), normally the ore in the mining process has a size less than 0.01-1.0 mm, the iron content is removed and the remaining waste product (tailings) are pumped in a slurry to an impoundment. These tailings facilities often consist of tailings ponds and subsequent clarification ponds. In that case the process water from the tailings pond discharges into the clarification ponds and afterwards the clarified water is reused either as process water or as a fluid for the transport of tailings to the tailings dams (European Commission 2004). Excess water from this cycle, which complies with the European and Swedish water quality standards, is diverted to the receiving water, as described in the reference document of the European Commission (2004).

In Figure 12, a paradigmatic cross-section of a typical tailings dam is shown, including a schematic water cycle of a Tailing Management Facility (TMF), after European Commission (2004).

![Figure 12: Cross-section: Tailings dam water cycle (European Commission 2004).](image)
In general civil engineers are familiar with construction details of standard embankment dams in use of a WRD. Being an expert in the area of tailings facilities, including impoundments and tailings constructions, more further knowledge are necessary. After Benckert and Eurenius (2001), tailings are, by volume, one of most handled products in the world, with approximately 25-30 Mt per year only in Sweden. Due to remaining ore in tailings, with possible negative impacts for the phreatic water (ground water), special construction measures have to be implemented (Benckert and Eurenius 2001).

The Swedish guideline GruvRIDAS (2007), known as “Mining Industry Guidelines for dams” and published by Svensk Energi AB / SveMin, has been prepared on the basis of RIDAS. Therefore, the structural components and requirements are similar to WRDs, described in section 2. The following sections describe typical impoundment types, structures and design methods as well as characteristics of Swedish tailings dams.

### 3.1 Impoundment Types

Just like embankment dams, several types of impoundments can be used and some of them are more common than others. According to the Environmental Protection Agency (1994) the Ring-Dike, In-Pit, Specially Dug Pit, and some Valley designs are the four main types of impoundments. In addition the choice of impoundment type, after Environmental Protection Agency (1994), is mainly dependent on place conditions, topography, and of course economic considerations. The amount of fill material used to construct a dam is related to costs and therefore most operating tailings dams correspond to the Valley design, due to a minimized material requirement by using the valley topography (Environmental Protection Agency 1994).

Following, important topographic and design differences between the impoundment designs Valley and Ring-Dike are explained.
3.1.1 Cross-Valley Design

As described in the introduction of this chapter, financial advantages arises by using topographic depressions as a store basin for tailings and as well by a reduced dam size and construction effort. Another advantage, according to Environmental Protection Agency (1994), is a reduction of air dispersion of tailing particles. In Figure 13, typical variations of valley impoundments are shown.

![Figure 13: Overview on Valley Impoundments. a) Single Cross-Valley, b) Multiple Cross-Valley (Vick 1990).](image)

It is obvious that a Single Cross-Valley (a) is comparable with WRD by general construction and topographic conditions. According to Environmental Protection Agency (1994), valleys should be located near the head of the drainage basin to minimize inflows. The Multiple Cross-Valley is to be considered as a variation of several Single Cross-Valley constructions.

3.1.2 Ring-Dike Design

In contrast to the Valley design, the Ring-Dike impoundment is used in areas with a flat topography without depressions. In that case, embankment dams are required to enclose the whole impoundment (Environmental Protection Agency 1994).
In some cases, the site topography enables a reduction or, on the other hand, an expansion of needed embankment parts, see Figure 14.

![Figure 14: Overview on Ring-Dike impoundments. a) Single, b) Segmented design (Vick 1990).](image)

After Environmental Protection Agency (1994), the construction of these dams with a low height is comparable to dams for valley impoundments concerning the use of tailings, waste rock and additional natural materials. In general, the material requirement for dam constructions is higher than for all other existing impoundment designs, whereby the cost is affected. Due to the typical flat topography, after Environmental Protection Agency (1994), their configuration enables a high flexibility. The quantity of pond water is limited to the inflow of process water and precipitation (Environmental Protection Agency 1994).

As already described, the increased demand of dike material is influencing the construction costs. Furthermore, an increased length of embankment dams may increase the hazard of failures (Environmental Protection Agency 1994). An impairment of the landscape painting and possible wind erosion of the tailings, after Environmental Protection Agency (1994), are possible disadvantages of this impoundment design. On the other hand, the Environmental Protection Agency (1994) points out that it may be possible to achieve a total containment and collection of waste water using an appropriate combination of low permeable cores, liners, and drainages.

Seepage control, i.e. acid and heavy metal drainage, is an environmental concern with tailings impoundments and ring-dikes therefore have an advantage over most other impoundment designs (Environmental Protection Agency 1994).
3.2 Structures and Design Methods

Embankment structures can be distinguished by construction type. The first type is a non-permeable tailings retention dam, constructed at full height at the beginning and similar to WRDs, and the second type is a much more common permeable raising tailings dam (Environmental Protection Agency 1994). Due to the fixed dam height of retention dams the availability of storage capacity for tailings is limited. According to Environmental Protection Agency (1994) this construction type, as shown in Figure 15, is comparable to a water retention dam with regards to soil quantities and qualities, surface and groundwater controls, and statically considerations.

![Figure 15: Cross-section of a tailings retention dam, comparable to typical WRDs (Environmental Protection Agency 1994, Vick 1990).](image)

The construction of raised tailings dams, however, starts with a starter dam (dike), followed by successive lifts of dam height with increasing volume of tailings in the impoundment (Environmental Protection Agency 1994). Raised tailings dams impound high volume of tailings and even a high water level. According to Berghe et al. (2011), the water is leaking through the dam body and the foundation soil. Therefore it is indicated that an efficient drainage system is essential, preventing the water table flowing towards the dam slope. It is common to install a network of drainage pipes connected to a main pipe at the embankment bottom to drain the seepage water (Berghe et al. 2011). However, all tailings dams have to be constructed with greatest possible safety, and also continuously reviewed as conditions change with time (Benckert and Eurenius 2001).

In addition, raised tailings dams can be classified in two categories (European Commission 2004):

- Dams with tailings and a low permeable core, and
- dams with tailings in structural zone.
3.2.1 Dams with Tailings and a Low Permeable Zone

This dam category enables a low permeable zone due to tailings deposition in the range of the upstream dam slope (European Commission 2004). Between this deposition and the starter dam a filter zone is installed, see Figure 16. The deposited tailings are building up a beach, after Bjelkevik (2005b) an area of tailings between the water edge and the dam crest as a result of a sedimentation process. Inflowing process water must not rise above the upper edge of the beach, because this water would enter dam material with a higher permeability (European Commission 2004). Due to this existing sensitive area, a continuous monitoring program must be implemented and therefore a permeability barrier must be installed before beach developments (European Commission 2004).

![Figure 16: Cross-section of a tailings dam with a low permeable core zone (beach) (European Commission 2004).](image)


3.2.2 Dams with Tailings in Structural Zone

Upstream, downstream, centerline, and a combination of the types are the main design methods of raised tailings dams, after Vick (1990), detailed described below. In contrast to non-permeable tailings retention dams, these dams are usually constructed of natural soil, tailings from the mining process, and crushed waste rocks (Vick 1990). As a consequence of using waste rocks and tailings, these tailings dams provides financial advantages for the industrial companies (Environmental Protection Agency 1994). The incremental raising structure of each design method is shown in Figure 17.
Figure 17: Tailings dams design methods: a) Upstream, b) Centerline, c) Downstream (Vick 1990).

**Upstream Method**

The upstream method, as shown in Figure 18, represents an inwards raised embankment dam above the deposited tailings (DHI Water Environment Health 2007). This method continuously uses the advantages of a low permeable zone due to tailings deposition. In general, the lower the difference between the free water level and dam crest is, the higher the phreatic surface (hydraulic gradient) of the embankment dam, leading to a higher risk of incidents and failures (European Commission 2004). Thus the filter and drain system, after DHI Water Environment Health (2007), are very important components for this design type to lowering the phreatic line and provide stability.

Figure 18: Incremental raising upstream tailings dam (Vick 1990).
Downstream Method

The downstream design method, as shown in Figure 19, represents a downstream raised and expanding dam structure with a crest movement as the dam height is rising (DHI Water Environment Health 2007). Based on a starter dam constructed of borrow material, comparable to the upstream design method, the next raise is placed on the downstream slope by reaching the maximum impoundment level (European Commission 2004). Therefore, in contrast to the upstream method, this variant enables a rising tailings storage space with each dam heightening. On the other hand the dam toe is moving at each construction step.

![Figure 19: Incremental raising downstream tailings dam (Vick 1990).](image)

Tailings, normally are used as construction material and separated by hydrocyclones (Figure 20), may be used for the major dam construction (European Commission 2004).

![Figure 20: Hydrocyclones on a dam crest (European Commission 2004).](image)
After the tailings slurry is dewatered to a pulp density of approximately 60 to 70 %, the remaining slime is removed by passing the stream through the hydrocyclones (European Commission 2004). According to the European Commission (2004), hydraulic backfill by hydrocyclones presents permeability coefficients in the range of $1 \times 10^{-7}$ to $1 \times 10^{-4}$ m/s.

Also this design method enables the installation of a filter and drain system to control the phreatic line. After Environmental Protection Agency (1994), it is recommended to place a pervious sandy drainage layer or other drain systems prior to each downstream increase to reduce the hydraulic gradient. An example is, after Environmental Protection Agency (1994), to incorporate, at each raise of the embankment dam, a chimney drain (Figure 19) near the upstream surface slope with a connection to a blanket drain at the embankment basement.

**Centerline Method**

The centerline design method, as shown in Figure 21, is a compromise between the downstream and upstream design method, also based on a starter dam constructed of borrow material (Bjelkevik 2005b). From the tailings discharge line (Figure 21-a) on the dam crest the deposited tailings are building up a low permeable beach zone, as described before in this chapter. Prior to the impoundment is full, the next embankment stage is built on the beach and on the downstream slope of the starter dam, see Figure 21-b (Bjelkevik 2005b).

The staged construction is comparable to the downstream method and also the maintenance of the beach, in order to lower and control the phreatic surface, is necessary. The centerline design method is also equipped with a filter and drain system, as also integrated the downstream and upstream method (Bjelkevik 2005b).
Other combinations of engineering methods are also possible and existing. Construction changes are also conceivable with changing conditions concerning mining processes and topographical limitations. An example is the tailings dam in Malmberget, Sweden, whose design method was changed at a specific level from upstream to downstream design (European Commission 2004). More details about Malmberget's tailings dam are discussed in chapter 6.
3.3 **Swedish Tailings Dams**

Swedish mining companies belong to the largest enterprises of Europe. Mines and neighbouring tailings dams in operation as well as hydropower dams are illustrated in Figure 22. Due to a large amount of process waste accrues companies and authorities are dealing with construction, operation, and remediation of tailings storage facilities (TSF), including tailings dams, impoundments, decant facilities and spillways (Bjelkevik 2005b). After mining companies have dealt with TSFs themselves in the beginning of the twentieth century, during 1960 and 1970 consultancy service became more important when problems occurred (Benckert and Eurenius 2001). The big expansion of consulting and monitoring activities is a result of the increasing height of tailings dams, the public and authority awareness of risks and the appearance of incidents and failures (Benckert and Eurenius 2001).

*Figure 22: Swedish mines and tailings dams in operation, 2005 (left hand side) (Bjelkevik 2005a) and Swedish dams including the classification of height (right hand side) (Swedish National Grid 2011).*
Furthermore, in 1997 the Swedish mining company Boliden started an initiative to develop manuals for operation tailings dams, based on the Swedish Dam Safety Guidelines (RIDAS) (Benckert and Eurenius 2001). Other mining enterprises followed this, after Benckert and Eurenius (2001), with site specific operation manuals. In addition, the Swedish guideline GruvRIDAS, published by Svensk Energi AB / SveMin, has been prepared on the basis of RIDAS.

The following Table 4 shows general data about Swedish tailings dams and storage capacities of the three main mining companies LKAB, Boliden Mineral and Zinkgruvan Mining (Benckert and Eurenius 2001).

Table 4: General data about Swedish tailings dams and capacities (Benckert, Eurenius 2001). Foot print area, according to (Bjelkevik 2005b).

<table>
<thead>
<tr>
<th>Name, Owner</th>
<th>Start</th>
<th>Ore type</th>
<th>Construction Type *</th>
<th>Material</th>
<th>Foundation material</th>
<th>Height</th>
<th>Capacity **</th>
<th>Foot print area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gillervattnet, Boliden</td>
<td>1953</td>
<td>Zn,Cu, Pb,Ag, Au</td>
<td>ds, moraine</td>
<td>moraine</td>
<td>12 m</td>
<td>20 Mm³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ryllshytte-magasinet, Boliden</td>
<td>1965</td>
<td>Cu,Zn</td>
<td>ds, moraine</td>
<td>moraine</td>
<td>19 m</td>
<td>6.5 Mm³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Svappavaara, LKAB</td>
<td>1965</td>
<td>From Kiruna</td>
<td>us, moraine</td>
<td>moraine</td>
<td>18 m</td>
<td>6 Mm³ 1.2 m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aitik, Boliden</td>
<td>1967</td>
<td>Cu,Ag, Au</td>
<td>us, moraine</td>
<td>moraine, tailings &amp; waste rock</td>
<td>50m</td>
<td>250 Mm³ 12.0 m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Enemossen, Zinkgruvan</td>
<td>1976</td>
<td>Zn,Pb, Ag</td>
<td>ds, cl &amp; us, moraine &amp; waste rock</td>
<td>moraine/rock</td>
<td>25 m</td>
<td>7 Mm³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kiruna, LKAB</td>
<td>1976</td>
<td>Fe</td>
<td>cl, moraine</td>
<td>moraine</td>
<td>15 m</td>
<td>10 Mm³ 2.6 m²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Laisvall, Boliden</td>
<td>1978</td>
<td>Pb,Zn, Ag</td>
<td>us, tailings &amp; moraine</td>
<td>moraine</td>
<td>40 m</td>
<td>20 Mm³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Malmberget, LKAB</td>
<td>1978</td>
<td>Fe</td>
<td>ds, moraine &amp; waste rock</td>
<td>moraine</td>
<td>32 m</td>
<td>20 Mm³ 1.8 m²</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* ds = downstream, us = upstream, cl = centerline

** includes tailings and clarifications impoundment storage space

All operating tailings dams according to Table 4 in Sweden have a separate water pond and clarification pond for a second sedimentation step and recirculate stored water to the mining process, as described in section 3 (Benckert and Eurenius 2001). After Benckert and Eurenius (2001), the largest TSF is situated at the Aitik mine, including a clarification pond with a water reservoir capacity of 15 Mm³.
Moraine as construction material for tailings has always been used in Sweden because approximately 60% of Sweden is covered by moraine (Benckert and Eurenius 2001). Based on knowledge and experience of WRDs the tailings dams has being constructed with a moraine core, filter layers, a drainage system and supporting fills (Bjelkevik 2005b). After Bjelkevik (2005b), the use of moraine for dam constructions was probably affected by the availability of this material on site. Nowadays, due to international experience in the 90s concerning the hydraulic gradient and the phreatic surface, the dam design changed from standard earth fill dams to the upstream and centerline design, as described in section 3.2, with a limited moraine use for the dike and sometimes for the core, and use of tailings for the remaining dam construction (Bjelkevik 2005b).

3.3.1 Remediation

As mentioned earlier in this section, the public as well as the mining industries increasing environmental awareness led to a new way of environmental thinking (Benckert and Eurenius 2001). Thus, the philosophy of tailings dams remediation in Sweden, according to Benckert and Eurenius (2001), is equated with nuclear wastes. So, for design purposes a long-term period is defined to be 1000 years. Within this period the tailings dam passes the phases operation, transition and long-term phase, whereas mining companies also have to ensure environmental safety after when mining process is closed (Benckert and Eurenius 2001). Environmental safety is determined by the factors physical and chemical stability of the dam, whereby the fundamental failure mechanisms for the cases long-term stability, extreme events and slow degradation must be considered (Benckert and Eurenius 2001).
4 Instrumentation and Monitoring

Instrumentation of embankment dams for different surveillance respectively monitoring purposes is necessary for an early detection of structural changes in order to prevent incidents and failures. Possible failure mechanisms within the embankment structure, as piping in embankment dams (section 2.2.3), are illustrated in Figure 23 based on long-term experience of existing structures (Bettzieche 2008).

Figure 23: Different types of failure mechanism of embankment dams (Bettzieche 2008). Modified and translated by Jörg Riechers.

Description: a) Overtopping, b) Instability foundation slip due to soft subsoil, c) Seepage/ piping within and below the embankment dam, d) Slope breakage due to rapid drawdowns of the reservoir, e) Lost of freeboard due to settlement, f) Slope breakage on downstream side due to insufficient shell stability, g) Cracks due to dehydration and shrinkage, h) Liquefaction

In 2012 the German Association for Water, Wastewater and Waste (DWA) developed and published recommendations on structural surveillance of water barrages as embankment dams and concrete dams (Regulation No. DWA-M 514). Each of the following subsections discusses its application purpose, installation process respectively measurement characteristics. Furthermore, proposed number and position of each surveillance method, according to DWA-M 514, are taken into account. Thereby the focus is on water level monitoring in order to keep the seepage line under control, pore water pressure measurements with piezometers and displacement measurements by means of an inclinometer system.
4.1 Monitoring Wells and Piezometers

Monitoring wells, also known as observation wells, are used to monitor the water level of the embankment dam in order to oversee the seepage line. In reverse, the functionality of the filter adjacent to the impervious core can be controlled. Generally, the seepage line decreases from the core to the downstream side of the dam shoulder. Monitoring wells which are arranged in a row, installed after drilling as exemplary shown in Figure 24, enables the measurement of single water level values.

![Figure 24: Schematic view of monitoring well (left hand side) and of open standpipe piezometer (U.S. Army Corps of Engineers 1995).](image)

Piezometers are usually installed prior to or during construction of the dam as well as during its operating life in form as an open standpipe piezometer, see Figure 24. Most Piezometers have been installed for the surveillance of construction performance during the first filling and/or for long-term monitoring (U.S. Army Corps of Engineers 1995). The objective of this measurement device is to measure the pore water pressure at a certain position. The pore water pressure is known as the total stress transmitted through pore water, whereby the total stress can be described as an acting force within a certain area (U.S. Army Corps of Engineers 1995). Generally, different types of piezometer as pneumatic, electrical devices, etc. are available. Piezometers and monitoring wells can be used for slope stability analyses due to the acting seepage line.
Furthermore, the measurement of pore water pressure enables monitoring of the performance of a cutoff wall, as far as available. In addition, the acting hydraulic pressure can be measured to control the foundation stability. A seepage control is possible by monitoring the water level changes in combination with a measurement device in the horizontal/blanket drain, see Figure 25.

Figure 25 shows an example of positions for pore water pressure measurements (piezometers) in an impervious core as well as monitoring wells for water level measurements and seepage control in the downstream shelf of an embankment dam, according to DWA-M 514. Generally, regulations respectively requirements concerning drilling operations in embankment dams and its foundations must be observed, see subsection 5.1.

*Figure 25: An embankment dam with impervious core and installed piezometers (DWA-M 514 2011). Modified and translated by Jörg Riechers.*
4 Instrumentation and Monitoring

4.2 Inclinometer

This deviation measurement instrument is used for a wide variety of applications, such as slope monitoring for water retention dams and tailings dams as well as in the context of highway and railway constructions. Usually it is installed, after MSHA (2009), into a vertical or inclined borehole to measure lateral displacements of slopes or soil structures. Figure 26 shows examples of positions for geodetic measurements on the dam crest and the berm dam crest of an embankment dam, according to DWA-M 514. It is recommended to use at least three geodetic measurement points at a distance of 50 m in a row on the dam crest. In addition, inclinometers can be installed during to or after construction in boreholes at surveillance spots.

![Cross-section and top view of inclinometer installation](image)

*Figure 26: Possible positions for measurements of vertical and horizontal displacement in an embankment dam (DWA-M 514 2011).*

Furthermore, horizontal applications are available for receiving profiles beneath structures or embankments (MSHA 2009). The distribution of displacement (Figure 27) along the borehole depth can be measured whereby, after Berghe et al. (2011), a data processing enables the derivation of cumulative movements at any depth. Thus, measured and derived parameters, by using an inclinometer, allow a risk assessment for the embankment slopes (Berghe et al. 2011).
In this context, the most commonly used in-situ inclinometers monitor displacements normal to the axis of a pipe by passing a measuring probe along the pipe, see schematic installation in Figure 27 (MSHA 2009). This measuring device, is illustrated in detail in Figure 28, and consists of the following elements (MSHA 2009):

- guide casing including tracking grooves (permanently installed in the borehole),
- backfill between guide casing and adjacent material (location fixation),
- probe with a pair of wheels,
- electrical cable connecting the probe and the readout equipment,
- readout equipment (manually respectively continuously monitoring).
Inclinometers are usually installed during construction in embankment or concrete dams. Furthermore, the installation is normally fixed in a stable layer with very low movements like natural rock. Without a fixed base, after MSHA (2009), the top of the inclinometer casing must be measured before each measurement. Displacement measurements are taken to enable a comparison of measurements at different times. Subsequently installations are possible, but Berghe et al. (2011) recommends a quick implementation after the dam has reached a certain height to avoid measurement failures due to increasing shear load by heightening.

The accuracy of normal measurement devices is an important factor for their usability. Important parameters for inclinometers by contrast are the repeatability of measurements and the resolution of readings. The main objective by using of inclinometers is to measure the movement of the soil relative to a starting point respectively day. Thus, the repeatability determines the accuracy of readings at each repeated reading (Measurand 2012). On the other hand, the resolution, after Measurand (2012), describes the smallest change in a reading which is able to display. Hence, this parameter is limited by the logging equipment due to the supported number of bits carrying measuring information (Measurand 2012). Inclinometers of the company Durham Geo Slope Indicators, for instance, have a repeatability of 20 arc-seconds (± 0.1 mm/m) with a resolution of 9 arc-seconds (0.04 mm/m).

The installation of an inclinometer instrument is also possible during operation of dams, tailings dams, etc., by drilling a borehole in the structure and insertion of a suitable instrument. One example worth to be mentioned here is a flexible accelerator array, installable within drill pipes or other pipes with a narrow diameter, for monitoring of slope displacements and featured additional measurement possibilities. Further details of this special inclinometer array are described in subsection 5.4.2.
5 Drilling in Embankment Dams

This chapter deals with drilling in embankment dams for geotechnical investigation, instrumentation and remediation. Apart from the description of possible drilling risks, an overview on conventional drilling technologies is given. The main focuses are the description of the standard hydraulic top hammer drilling technology and the water powered DTH hammer drilling technique, especially the Wassara W35 for dam instrumentations, their mode of operation, as well as their applicability.

5.1 Drilling Risks

Drilling applications in existing embankment dams or in their foundations as boreholes for exploration and surveillance purposes represent always an interference in the structure. Nevertheless, drilling may be the only option available, after U.S. Army Corps of Engineers (1995), when the installations of new subsurface instruments or, for instance, simple water wells for surveillance purposes are needed in operating dam structures. Therefore, the choice between different drilling technologies and eventual drill fluids must be analysed carefully in advance (U.S. Army Corps of Engineers 1995).

The use of compressed air or water, after U.S. Army Corps of Engineers (1995), during drilling can negatively affect the embankment respectively foundation material by fracturing and erosion of material. Fracturing can be described as a process of cracking soil/rock by pressurised media. Generally, these effects are not only linked to embankment dams but also to other interferences by drilling operations in soil and underground formations. Therefore, conventional geotechnical investigations have been conducted in this thesis before and after drilling within the framework of the reference drilling project, as described in chapter 6.

Drilling in the sensitive impervious core of an embankment must be performed with the greatest of care and precision (U.S. Army Corps of Engineers 1995). Generally, dam owners avoid penetrating the dam core unless it is really necessary. According to U.S. Army Corps of Engineers (1995), several incidents occurred in the past among drillings with compressed air, air with foam and other circulating drilling fluids in boreholes. Consequently, different regulations respectively requirements concerning drilling operations in embankment dams and its foundations were established by several national authorities and dam operators.
The U.S. Army Corps of Engineers (USACE), for instance, developed and published in 2006 generally applicable engineering regulations for drillings into embankments (Regulation No. 1110-1-1807).

Thus, the use of compressed air, air with foam or any other gas or water as the circulation medium within the borehole is prohibited for drilling in embankment dams and/or their foundation (U.S. Army Corps of Engineers 2006). On the other hand, the use of auger drilling and rotary sonic drilling, also known as sonic drilling, are indicated as acceptable methods and the general use is described in detail (U.S. Army Corps of Engineers 2006). If auger drilling is inappropriate, after U.S. Army Corps of Engineers (2006), the use of cable tool drilling or rotary drilling with engineered drilling fluids is permitted for certain applications. Guidelines for rotary drilling and especially for the use of muds (drill fluids) are described detailed in the appendix of regulation 1110-1-1807. In addition to the policies of drilling technology this regulation mandates high qualification requirements for all persons involved in drilling preparations and operations.

In general the use of top hammers as well as the use of a hollow stem auger for drilling in unconsolidated/ loose material without flushing fluids is usually preferred respectively permitted from different authorities. Furthermore, the use of Wassara’s water powered DTH hammers can be used for pre-drillings in dam and reservoir structures for jet-grouting purposes and the construction of cut-off walls to minimize water exchange (Wassara AB 2012b). This is in line with the mentioned exception in 1110-1-1807, whereby the use of drilling alternatives is permitted if recommended techniques are unpractical. Thus, drilling through pervious rock-fill respectively gravel sections of foundations and embankments, after U.S. Army Corps of Engineers (2006), are possible exemptions.
5.2 Hydraulic Top Hammer Drilling

Drilling systems, after Bruce (2002), must generally offer continuous and straight penetration properties in varying drill applications from soft to hard material respectively stable to fractured formations. Rotary, rotary percussive and rotary vibratory (sonic), after Bruce (2002), are the main categories of rock drilling. The focus of this master thesis is on rotary percussive systems, in terms of drilling technologies. A distinction is made between down-the-hole hammers (DTH), also referred to as in-the-hole hammers (ITH) for underground work, and top hammers. This section describes the working principle of top hammers and typical areas of application. Drill characteristics of DTH hammers, as well as the differences between air and water powered hammer systems are given in section 5.3 to 5.3.5.

A hydraulic top hammer system, also known as hydraulic drifter, is equipped with a regular up and downward moving piston inside of a cylinder, see Figure 29 (Rabia 1985). The repetitive down stroke of the piston converts potential energy of compressed hydraulic oil into kinetic energy (force) (Rabia 1985). This kinetic energy is transmitted to the drill string and forwarded to the drill bit down the hole in the form of compressive waves (Rabia 1985).

![Figure 29: Rotary percussive methods: Top hammer (left side), and down-the-hole hammer (right side) (Rabia 1985).](image-url)
As a result, the drill bit crushes the rock respectively soil and subsequently the returning force causes a backward movement of the piston to the starting position. The described operation principle is illustrated in Figure 30.

![Figure 30: Operation principle of hydraulic top hammers (Zablocki 2006).](image)

The energy, in form of travelling strain waves, after Rabia (1985), decreases with increasing length of the drill string (borehole depth). Energy losses occur, after Dessureault (2003), due to the pipe resistance, friction at couplings and joints, vibration, noise and as well as the wearout on the bit. The percentage of losses by pipe joints, after Corcoran (2009), varies between 4 % and 6 %. Stresses in the steel components as piston and drill string, for instance, must be limited because of durability requirements (Dessureault 2003). Thus, the maximum borehole depth respectively borehole length is limited to approximately 60 m, according to Bruce (2002), depending on the available torque, piston weight and the impact power of the hydraulic drifter (Figure 31). Thereby, the penetration rate noticeable decreases from a length of 20 m (Corcoran 2009).

![Figure 31: Hydraulic percussive hammer (hydraulic drifter) for underground applications (Corcoran 2009).](image)
In the 1960, top hammers employed compressed air with approximately 6 bar operating pressure (Atlas Copco Rock Drills AB 2008). Step by step, after Atlas Copco Rock Drills AB (2008), they were replaced by hydraulic top hammers with higher efficiency and a working pressure up to 25-28 bar. Around 25 to 30 % of the energy input is transferred to the rock on the bottom at hydraulic top hammers, compared approximately 5 % at pneumatic top hammers (Dessureault 2003). Another advantage, after Dessureault (2003), is a noise reduction of 8-10 dB (decibels) with the hydraulic hammer type. Nowadays, operating pressures around 180 bar and a maximum impact stroke repetition of approximately 2,000 l/min are usually used for different applications above ground and underground. These data are based on product information of different companies.

The hydraulic drifter is mounted on the mast of the drill rig and usually works with a rotation speed of approximately 80 to 160 RPM to generate borehole diameters of around 102 mm (Bruce 2002). Unlike DTH hammers, due to historical reasons, top hammers drillings are rotating counter clockwise. Besides the dependencies mentioned for optimal drillings with high rates of penetration, an optimal feed is important to keep the drill bit in contact with the formation surface, i.e. weight-on-bit (Rabia 1985). Compressed air, water or a mix of both mediums is transmitted to the bit through the annular space between drill string and casing to clean the borehole and flushing out the soil respectively rock fragments, i.e. cuttings (Rabia 1985).

Underground applications for hydraulic top hammers and air powered DTH hammers, for instance, are slot and production drillings. Furthermore, top hammers are used for several structural applications with limited depth as boreholes for exploration and surveillance purposes in embankment dams or foundation drillings, for instance. In the context of drilling in existing embankment dams, the use of top hammers as well as the use of auger drilling without flushing fluids is usually preferred respectively permitted from different authorities. More details are given in the previous section 5.1.
5.2.1 Performance and Drilling Accuracy

The purpose of this section is to provide a performance overview of the top hammer system in terms of penetration rate and drilling accuracy (deviation), in comparison with air powered DTH hammers. Differences between applications above ground and underground are taken into account. Generally, it is not easy to execute direct comparisons between different systems because of varying ground conditions.

The mine profitability, after Corcoran (2009), is influenced by the borehole deviation in terms of inadequate fragmentation and low ore recovery. Furthermore, he points out that approximately 71% of Canadian and US underground drilling applications are performed by DTH hammers for borehole diameters between 76 mm-305 mm (3”-12”) and 27% by top hammers for smaller hole diameter from 41 mm-113 mm (1 5/8”-4.5”). Thus, top hammer drilling tends to have deviations of 5% to 10%, with a significant increase of the inaccuracy after approximately 20 m depth. The deviation of air driven DTH hammers can be specified in the range of +/− 1%, based on past experience (Corcoran 2009). Based on similar experience in the use of top hammers, according to Atlas Copco Rock Drills AB (2008), many mines restrict the drilling length to 20 m without utilisation of guide rods behind the drill bit respectively to a length of 30 m with guidance equipment.

A comparative drilling study obtained in a hard limestone with a borehole diameter of 165 mm (6 1/2”) showed, after Halco (2012), a ROP of 40 m/hr for the DTH system and a ROP of 35 m/hr for the top hammer alternative. Unlike drilling in hard formations, top hammers usually do not drill efficient in soft rock respectively soil conditions in comparison with DTH hammers. The experience also shows that the ROP of a top hammer system decrease in not solid formations (Halco 2012). In addition to the already presented applications, long-term experience of the company Sweco Infrastructure AB exist in geotechnical drillings for exploration and surveillance purposes in embankment dams or foundations, for instance, in Sweden and Norway. Accordingly, the drilling of a water observation well (20-25 m deep) with a top hammer system including a filter installation in the downstream filter section of an embankment dam, for instance, takes about 2-4 days depending on the soil structure. The most time-consuming tasks are the installation of the filter pipe in connection with the subsequent removal of casing rods.
5.3 Water Powered DTH Hammer Drilling Technology

All information on water powered DTH hammer drilling technology correlates to technological developments by Wassara AB, hereinafter referred to as ‘Wassara’. The following terms and explanations in this section, which apply to all available hammer sizes, unless otherwise marked, refer to Riechers (2010). Beyond that, the emphasis is on Wassara’s W35 hammer development for embankment dam instrumentations, see section 5.4.

Water powered DTH percussion hammer drilling technology is designed for drilling in soil as well as hard and abrasive rock as granite and sandstone, for instance. Underground applications of this system, for example, are slot and production drillings. Furthermore, water powered DTH hammers are also used for several structural engineering applications as boreholes for exploration, surveillance and production purposes, anchor drillings as well as for investigation and remediation drillings in different dam types (Wassara AB 2012c).

The DTH hammer is guided by a clockwise rotating drill string, which is used to transfer pressurised water to the hammer operating down the hole. Pressurised water is fed by a swivel (Figure 32) above the rotating head at the drill rig. This water is used to drive the hammer, whereas the water after each hammer operating cycle is used to clean the borehole and flushing out the cuttings.

![Swivel including high-pressure hose mounted on a rotating head (Riechers 2010).](image)

After each blow, the water is transmitted to the bit to cool it and to flush the cuttings out of the borehole through the annular space between drill pipe and borehole wall (Smoltczyk 2003).
Below in this section, the operating principle of the hammer is described in particular. The displaced drilling fluid, a mix of cuttings and process water, is forwarded to a sedimentation container respectively under certain circumstances to a recirculation unit to minimise water loss (Nordell et al. 1998).

### 5.3.1 Pump and Drilling Fluid

The required operating pressure and the flow rate of the water, generated by a high pressure pump, depend on the size of the used DTH hammer and its condition, see Table 5. The selection of the hammer, in turn, depends on the desired borehole diameter. The need of water pressure and flow rate is influenced as follows:

- the larger the hammer diameter the, the higher the water flow rate,
- working pressure is depending on the hammer diameter and its condition,
- working pressure is also based on existing soil/ rock strength,
- the larger the cracks in the formation, the higher the water consumption,
- by increased hammer wear, the water consumption of the hammer increases.

It should be noted that, unlike air driven DTH hammers, existing phreatic water is not affecting the penetration rate of water powered DTH hammers.

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</thead>
<tbody>
<tr>
<td>64</td>
<td>W 50</td>
<td>895</td>
<td>950</td>
<td>54-58</td>
<td>170</td>
<td>80-130</td>
</tr>
<tr>
<td>82</td>
<td>W 70</td>
<td>1098</td>
<td>1170</td>
<td>78-87</td>
<td>180</td>
<td>130-260</td>
</tr>
<tr>
<td>95</td>
<td>W 80</td>
<td>977</td>
<td>1049</td>
<td>80-92</td>
<td>180</td>
<td>130-260</td>
</tr>
<tr>
<td>115</td>
<td>W 100</td>
<td>1185</td>
<td>1270</td>
<td>102-111</td>
<td>180</td>
<td>225-350</td>
</tr>
<tr>
<td>140</td>
<td>W 120</td>
<td>1495</td>
<td>1572</td>
<td>120-128</td>
<td>180</td>
<td>300-450</td>
</tr>
<tr>
<td>165</td>
<td>W 150</td>
<td>1495</td>
<td>1605</td>
<td>140-162</td>
<td>150</td>
<td>350-500</td>
</tr>
</tbody>
</table>
The supply of fresh water takes usually place through a hydrant and a temporarily water tank in avoidance of pressure changes. In general, the used water should be clean of impurities and abrasive fine particles as quartzite as well as having a high pH value to obtain a corrosion protection. Alternatively, lake water can be used by a submersible drainage pump equipped with a pre-filter able to filter greater particles than 50 µm (0.05 mm).

In contrast to air, water can be described as an incompressible fluid due to a very low rate of volume change. The modulus of elasticity (E-value), defined as the ratio between stress and strain, of water \((2.1 \cdot 10^9 \text{ N/m}^2)\) is approximately 10,000 times higher than of air \((1.5 \cdot 10^5 \text{ N/m}^2)\). Pressure (stress) is given as the quotient of the force and a surface area.

\[
P = \frac{F}{A} \quad \Rightarrow \quad 1 \text{ Pa} = 1 \, \frac{\text{N}}{\text{m}^2} \quad \Rightarrow \quad 100 \text{ kPa} = 100 \, \frac{\text{kN}}{\text{m}^2} = 1 \text{ bar} = \text{water column of 10 m}
\]

The hydrostatic distinguished by consideration of pressure between water pressure and absolute water pressure. As a result, the absolute water pressure included the atmospheric pressure \((P_0 = 101325 \text{ Pa})\) on the water surface. On the other hand, the water pressure increases with depth \((z)\) without consideration of atmospheric pressure, see the following equations (6.1) and (6.2).

Water pressure: \[P = \rho \cdot g \cdot z \quad [\text{Pa}] \quad (6.1)\]

Absolute water pressure: \[P = \rho \cdot g \cdot z + P_0 \quad [\text{Pa}] \quad (6.2)\]

\[P_0 = 101325 \quad \text{Pa}\]

Due to the previous described incompressibility of the water, the required pump power is lower than of usually used air compressors for air driven DTH hammers, despite a significant higher pressure rate. The possible drilling depth by using air powered DTH hammers is limited due to compaction effort and maximal available pump capacity unlike to the innovative water driven DTH hammers.

*Table 6: High-pressure pumps, usually used to drive DTH hammers of Wassara (Wassara AB 2012b).*

<table>
<thead>
<tr>
<th>Pump model</th>
<th>Max. flow rate [l/min]</th>
<th>Max. water pressure [bar]</th>
<th>Required max. power [kW]</th>
</tr>
</thead>
<tbody>
<tr>
<td>WASP 50 Diesel</td>
<td>137</td>
<td>170</td>
<td>43</td>
</tr>
<tr>
<td>WASP 80 Diesel</td>
<td>219</td>
<td>200</td>
<td>80</td>
</tr>
<tr>
<td>WASP 150 Diesel</td>
<td>476</td>
<td>200</td>
<td>173</td>
</tr>
</tbody>
</table>
In opposition to air pressure hoses, a water pressure hose presents no danger in case of bursting despite a higher pressure around 180 bar due to the incompressible characteristics of water. Generally, the drilling fluid is referred to a controlled circulating fluid or gas in the borehole. In this context the fluid is defined as a mix of gas respectively water and additives like weighting agents, for instance, to enable a higher level of borehole stability. The main functions are to clean the borehole and flushing out the cuttings, cooling the drill bit and to guarantee optimum borehole stability. In case of drilling operations in unstable rock or fine grained soil formations, additional protective casings must be used to stabilise the borehole wall. The upflow velocity of the drilling fluid is depending on the used medium, the volume of flow, borehole and drill pipe diameter. In addition to the borehole stability, an optimal matching drilling fluid to the respective drilling reduces the mechanical wear of used tools. More details are given in section 5.3.4.

Due to the already described hammer requirements, water powered DTH hammers are used without additives in the water. Uncompressed water, escaping from flushing channels of the drill bit after each hammer operating cycle, cleans the borehole and flushing out the cuttings. The cuttings are flushed out of the borehole as slurry of water and broken rock/soil fragments through the annular space between drill pipe and borehole wall, i.e. reverse circulation drill (Smoltczyk 2003). Through the use of water as a flushing medium, unlike air DTH hammers, a risk by fracturing of adjacent formations does not exist. Furthermore, the hydrostatic pressure stabilise the borehole and the specific heat capacity of water, \( c = 4,187 \text{ J/kg·k} \), removes process heat of the drilling from the borehole. Besides the hydrostatic pressure, the fluid of water and solids is building a filter cake around the borehole wall to stabilise the hole and reduce the drill fluid loss. The maintenance of an efficient filter cake is generally an important task of the operator during drilling.
In contrast to the high upflow velocities between 20 and 60 m/s of air, used for air powered DTH hammers, the water driven hammer alternative indicates velocities from 0.4 to 2.0 m/s despite higher working pressures, depending on the annular space in the borehole. Thus, the uncompressed water does not influence the stability and structure of adjacent ground whereby drillings besides respectively below sensible structures are possible. Other differences compared to air powered DTH hammers and Top-hammer drillings exists. Water driven percussion DTH hammers provide a reduction of noise emission and air pollution as a result of the position of the working hammer (down the hole) and due to the dust binding effect of the water. Furthermore, no additives as oil, for instance, are used to lubricate Wassara’s DTH hammers, usually used during drillings with standard DTH air hammers.

5.3.2 Hammer Design

Water powered DTH hammers, as already described, are designed for operation down-the-hole and belong to the group of rotary percussive technology. The piston casing is manufactured with passive stabilisers and defined as a ribbed piston casing. This design, amplified by the use of an additional guided tube above the hammer, enables guided drillings with reduced annual space of 2 mm between the hammer body (steering ribs) and borehole wall, see Figure 33.

![Figure 33: Distance to the borehole wall; hammer and additional stabiliser (Riechers 2010).](image)

These passive stabilisers in combination with a low upflow velocity of the drill fluid, low water pressure after the kinetic energy is transmitted as a stroke of the piston to the underground, an optimal feed force and optimised drill bit revolutions enables straight boreholes and high penetration rates (see subsection 5.3.1). The following subsection 5.3.3 discusses the operating principle of the hammer in detail followed by information on drill bits and drilling adjustments.
5.3.3 Operating Principle

In general, the operation principle of water powered DTH hammers distinguishes from the pneumatic DTH variant in the used working and flushing medium. The function of DTH hammers essentially depends on two related hammer parts, the valve and piston (Riechers 2010). Following, the internal process of the hammer is described, after Riechers (2010), step by step.

The cycle starts with the lowest position of the piston towards the drill bit (left side of Figure 34). In this position, the kinetic force is transmitted as a stroke of the piston to the bit shank and forwarded to the rock. The piston movement is achieved by the acting potential energy of compromised high-pressure water. Subsequently the returning force causes a backward movement of the piston to the starting position. In this process, the low-pressure connection between the piston and the control valve is open and the control valve is also moving upwards.

![Figure 34: Operation principle: 1. Cross-section of a DTH hammer (Riechers 2010).](image)

High-pressure water is entering the low-pressure connection due to a valve connection opening by the upward movement of the piston. This pressure change in the control valve drive area causes a turnaround of the control valve movement towards the bit shank (Figure 35).

![Figure 35: Operation principle: 2. Cross-section of a DTH hammer (Riechers 2010).](image)
The piston drive area can be pressurised when the piston has reached its down position, due to a reopening of the valve connection. A striking movement of the piston towards the bit starts, due to acting high-pressure water (Figure 36).

![Figure 36: Operation principle: 3. Cross-section of a DTH hammer (Riechers 2010).](image)

The previously sealed valve port opens due to the downward movement of the piston and the dewatering process of the control valve area begins (Figure 37). The drainage causes a water pressure reduction whereby the control valve moves towards the opposite direction of the piston back into its initial position.

![Figure 37: Operation principle: 4. Cross-section of a DTH hammer (Riechers 2010).](image)

After the kinetic force is transmitted as a stroke of the piston to the bit shank, and the control valve area is completely dewatered, the operating cycle of the hammer starts again (Figure 38). The discharged water is used to clean the borehole bottom and to flush the fragments out of the borehole. Depending on the hammer size, pressure level and mechanical soil/rock properties, the repetition of this related hammer parts takes place with high frequencies.
An example is the Wassara W100 hammer, normally operating with a frequency of approximately 60 Hz in hard rock. Hz is the symbol of frequency with the SI unit 1/s, defined as the number of cycles per seconds of periodic execution.

Water powered DTH hammers as well as the pneumatic DTH variants normally utilise an additional check valve to prevent upward movements of fine drill cuttings during drill stops through the hammer body. These cuttings represent a high potential risk of damage for internal hammer parts by the resumption of drill operation. Wassara’s DTH hammers are not equipped with an internal check valve. In order to achieve an additional protection of the threat of the hammer and to enable a quick exchange of a worn out check valve it is commonly mounted within a short pipe above the hammer, see Figure 39.
5.3.4 Drill Bits and RPM

The rotation of the drill string respectively of the drill bit is specified as revolution per minute (RPM) with the SI unit $1\text{ rpm} = 1\cdot(60\text{s})^{-1}$. More often used is the unit $1/\text{min}$ (per minute) with a 60 times higher numerical value. The aim is to loosen sufficiently large rock/soil fragments that can be flushed out as fast as possible off the borehole with a minimum of energy input. Therefore, an effective borehole cleaning reduce the amount of re-crushed cuttings whereby the energy input and bit wear can be reduced.

The penetration rate, after Kahraman et al. (2003), is an important parameter for drilling cost estimation and project planning. This value is usually indicated in the unit m/min or cm/min and also known as rate of penetration (ROP). Rock properties like compressive strength [MPa] (Figure 40) and abrasiveness influences the penetration rate of percussion drill and erosive wear of the bit Kahraman et al. (2003). The abrasiveness of rock, after Dessureault (2003), is basically influenced by the proportion of quartzite, see Figure 41.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Specific gravity (m³/m³)</th>
<th>Grain size (mm)</th>
<th>Swell factor (mm)</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intrusive</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diorite</td>
<td>2.65-2.85</td>
<td>1.5-3</td>
<td>1.5</td>
<td>170-300</td>
</tr>
<tr>
<td>Gabbro</td>
<td>2.85-3.2</td>
<td>2</td>
<td>1.6</td>
<td>260-350</td>
</tr>
<tr>
<td>Inclined</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td>2.7</td>
<td>0.1-2</td>
<td>1.6</td>
<td>200-350</td>
</tr>
<tr>
<td>Andesite</td>
<td>2.7</td>
<td>0.1</td>
<td>1.6</td>
<td>300-400</td>
</tr>
<tr>
<td>Extrusive</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>2.8</td>
<td>0.1</td>
<td>1.5</td>
<td>250-400</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>2.7</td>
<td>0.1</td>
<td>1.5</td>
<td>120</td>
</tr>
<tr>
<td>Trachyte</td>
<td>2.7</td>
<td>0.1</td>
<td>1.5</td>
<td>330</td>
</tr>
<tr>
<td>Conglomerate</td>
<td>2.6</td>
<td>2</td>
<td>1.5</td>
<td>140</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2.5</td>
<td>0.1-1</td>
<td>1.5</td>
<td>160-255</td>
</tr>
<tr>
<td>Slate</td>
<td>2.7</td>
<td>1</td>
<td>1.25</td>
<td>70</td>
</tr>
<tr>
<td>Sedimentary</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dolomite</td>
<td>2.7</td>
<td>1-2</td>
<td>1.6</td>
<td>150</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.6</td>
<td>1-2</td>
<td>1.55</td>
<td>120</td>
</tr>
<tr>
<td>Limerock</td>
<td>1.5-2.6</td>
<td>1-2</td>
<td>1.1</td>
<td>30-100</td>
</tr>
<tr>
<td>Gneiss</td>
<td>2.7</td>
<td>2</td>
<td>1.5</td>
<td>140-300</td>
</tr>
<tr>
<td>Marble</td>
<td>2.7</td>
<td>0.1-2</td>
<td>1.6</td>
<td>100-200</td>
</tr>
<tr>
<td>Metamorphic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Quartzite</td>
<td>2.7</td>
<td>0.1-1</td>
<td>1.55</td>
<td>160-220</td>
</tr>
<tr>
<td>Schist</td>
<td>2.7</td>
<td>0.1-1</td>
<td>1.6</td>
<td>60-400</td>
</tr>
<tr>
<td>Serpentine</td>
<td>2.6</td>
<td>-</td>
<td>1.4</td>
<td>30-150</td>
</tr>
<tr>
<td>Slate</td>
<td>2.7</td>
<td>0.1</td>
<td>1.5</td>
<td>150</td>
</tr>
</tbody>
</table>

*1 MPa = 1 MN/m² = 10 kg/cm² = 142.2 psi

Figure 40: Different rock properties (Dessureault 2003).
Therefore, various DTH drill bit designs exist with different buttons/inserts on the surface for a variety of applications, as illustrated in Table 7.

<table>
<thead>
<tr>
<th>Type of surface</th>
<th>Button form</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat front</td>
<td>Conical (ballistic)</td>
</tr>
<tr>
<td>Concave front</td>
<td>Semiballistic</td>
</tr>
<tr>
<td>Convex front</td>
<td>Spherical</td>
</tr>
</tbody>
</table>

The selection of surface design and button form in order to achieve high rates of penetration and long bit life is depending on formation properties as described before. Nowadays, buttons/cutters made of tungsten carbide are often covered by a thin layer of special carbon mixture to improve the penetration rate and reduce the button wear. These PDC bits, using polycrystalline diamond compact buttons respectively cutters and provide high shear strength resistance.
Drill bits with full ballistic buttons and a concave front, for instance, are suitable for applications in medium hard to hard and abrasive formations. For applications in very hard and stable formations bits with a convex front and full ballistic buttons are preferred. For drilling in hard and fractured rock formations by contrast it is important to secure an evenly rock erosion to enable effective drillings and boreholes with a high level of accuracy. Drill bits with a flat front and spherical buttons support this intention. Typical formations are granite, limestone and basalt, for instance. For less hard formations and unconsolidated soil a concave respectively convex front can be used with full ballistic buttons. For drilling in formations with intrusions of clay and silt the focus is on the amount of inserts and the spacing respectively an efficient bit cleaning to avoid clustering and wearout of the bit.

Generally, different DTH drill bit brands can be used with water powered DTH hammers. Wassara recommends using drill bits semiballistic buttons. During different R&D projects very good penetration rates in hard rock could be achieved with the latter variant. Special drill bits for Wassara’s DTH hammers are equipped with drainage notches on the bit shaft to avoid cavitation damage on the bit and hammer piston, see Figure 42. Cavitation is described, after Franc and Michel (2005), as the arising of vapour cavities within liquid mediums like water in conditions of reduced pressures and constant ambient temperature. If this vapour phases collapse close to material surfaces, damage caused by acting forces arise.

The second difference is a constant diameter of the flushing channel at DTH drill bits of Wassara, unlike bits for air driven DTH hammers. Due to lower flushing velocities, the channel diameter of Wassara hammers and drill bits are lower than that of a bit for an air system. Thus, an air drill bit used for drillings with water powered DTH hammers damages the piston of the hammer (Figure 42).

![Figure 42: Characteristics of drill bits for water powered DTH hammers (Riechers 2010).](image)
In addition to the drill bit, an optimal clockwise rotation of the bit and drill string reduces the amount of re-crushed cuttings and bit wear respectively enables high rates of penetration. RPM and feed force during drillings are regulated by operator of the drilling rig. Theoretically, the optimal RPM is depending on the outer diameter of the drill bit, diameter of the used buttons as well as the used operating frequency of the hammer, see Figure 43. The standard operating frequency is 60 Hz, used for most Wassara percussion hammers. In this context, the optimal range of RPM of a W 100 water powered DTH hammer can be specified with 50 to 90 revolutions per minute. In contrast, standard air DTH equipment usually rotates with 20 to 30 revolutions per minute.

![Figure 43: Advantages of an optimum RPM (Riechers 2010).](image)

According to Wassara, a maximum and optimum rotation can be calculated by the following equations in consideration of the outer diameter of the bit, penetration depth of the buttons and a maximum speed of 0.5 m/s at the outmost button of the bit. The maximum speed is based on experience to reduce the wear of the buttons.

**Maximum RPM:**

\[
N_{\text{max}} = \frac{f \cdot d \cdot 60}{D \cdot \pi} \left[ \frac{1}{\text{min}} \right] \quad (6.3)
\]

**Optimal RPM:**

\[
N_V = \frac{30,000}{D \cdot \pi} \left[ \frac{1}{\text{min}} \right] \quad (6.4)
\]

\(f = \text{hammer/ percussion frequency, often 60 Hz}\)
\(d = \text{Ø button [mm]}\)
\(D = \text{Ø drill bit [mm]}\)
5.3.5 Performance and Drilling Accuracy

The purpose of this section is to provide a performance overview of water powered DTH hammers in terms of penetration rate and drilling accuracy, in comparison with standard air DTH hammers. The following data, unless otherwise marked, was published in July 2010 and presented at the Geothermal Conference in Karlsruhe (Germany) as well as at the 1. Sustainable Earth Science Conference in Valencia (Spain) by Riechers (2010). It is not possible to make a direct comparison between top hammers, presented in sections 5.2, and this DTH system based on already existing data bases. Chapter 6 presents performance details based on a reference project that has been implemented for this research project.

In Stockholm, 29 geothermal wells (Ø 115 mm) were drilled with a planned depth of 220 m and 250 m for supplying two adjacent terrace houses. For clarity and avoidance of mutual influence all boreholes were separated according to the used drilling technology into drilling fields. In field I, 14 boreholes were drilled with an air powered DTH drilling system (Atlas Copco TD 40/ COP 44 Gold). Wassara’s water powered DTH hammer (W 100) was used in Field II for 15 boreholes.

According to the Swedish Geological Survey the bedrock in the drilling area has an irregular alternation of granite and gneiss. The gneiss shows intrusions of granite with a compressive strength of > 200 MPa and pegmatite courses. The plastically deformed rock is very fragile and an increased water flow in the subsurface is normally expected.

Despite the demanding drill formation, good drilling results could be achieved with the water powered DTH system. Comparing the average ROP of drilled boreholes in each field, the water powered DTH hammers offers a penetration rate of 0.52 m/min which is 24.6 % lower than the ROP of 0.69 m/min in field I. However, the performance of water driven DTH system is not affected by increasing borehole length and surrounding phreatic water.
Three-dimensional measurements have been implemented for almost all boreholes to analyse the borehole accuracy in terms of deviation. In general, the deviations of the boreholes from the perpendicular axis in field I up to a depth of approximately 40 m were 50% higher compared to field II, see Figure 44. Furthermore, the maximum deviation of the boreholes, which were drilled with the water powered DTH system, is 48 m. This corresponds to a 65% lower deviation compared to the straightest borehole drilled with an air powered DTH hammer, see Figure 44.

![Figure 44: Measured borehole deviations (Riechers 2010).](image)

Similar empirical values are described in a report from 1998, according to Nordell et al. (1998). Thus, Wassara’s penetration rate is clearly higher at greater depth and the technology enables drilling in hard, fractured and water rich formations (Nordell et al. 1998). Within a drilling project for the Swedish telephone company TELIA with water powered DTH hammers, penetration rates could be logged for 57 boreholes. Drill speeds between 0.5 m/min and 1.0 m/min as well as the average penetration rate of 0.6 m/min, after Nordell et al. (1998), could be determined.
5.4 Wassara W 35 DTH hammer for Embankment Dam Instrumentations

Research and developments on existing DTH hammer versions and drilling equipment as well as the development of technologies for new fields of application are an important part of Wassara’s business activities. The new W 35 DTH hammer and accessories, as illustrated in Figure 45, is one of the latest developments. The hammer and a special drill pipe between the piston casing and the ordinary pipes together forming a drilling system for new fields of applications. Thus, the objective is not only to develop a new hammer, but instead to design an advanced system for instrumentation installations in dams or other constructions respectively formations.

The basic idea is to develop a complete drill system with a small diameter which enables after drilling subsequent utilisation for dam instruments. Thereby, the drill pipes are used in the subsurface after completion of the borehole as casing rods for different measurement instruments down the hole. Consequently, this concept for drilling and instrumentation is referred to as Lost Hammer Concept (LHC) due to the remaining small W 35 DTH hammer in the borehole.

In order to ensure an adequate hydraulic connection between the internal pipe volume and the borehole surrounding ground water a flow tube is mounted in the first perforates drill pipe, as shown in the centre of Figure 45. This flow tube, which is kept in place by the acting water head, enables water feeding to the W 35 hammer during drilling.

Figure 45: View and cross-section of Wassara W 35 DTH hammer (Wassara AB 2012a).
After completion of the drilling, the flow tube must be pulled out of the borehole by a special lifting device, see Figure 46. The mentioned lifting device is described and illustrated in subsection 5.4.1.

![Figure 46: Principle of the LHC: lifting device is in position before lock into place (upper figure), hydraulic connection is opened (lower figure) (Wassara AB 2012a).](image)

As a result, the hydraulic connection is opened, whereby measurement instruments can be installed and starting the record of data. The usable internal pipe diameter is limited to 28 mm. With a suitable measuring device the distribution of displacements and temperature information, for instance, can be measured. One possibility and its benefit is described in subsection 5.4.2. Furthermore, this system can be used as a water observation well for different purposes in embankment dams or other soil construction applications. The decisive advantage is the simple and fast installation without a protective casing and conventional procedures such as drilling, flushing, installation of the observation well including a filter pipe and final insertion of filter sand. Furthermore, the small borehole diameter of 43 mm as well as the reduced drill pipe diameter of 35 mm reduces the interference in the embankment structure respectively in other soil or rock formations.

The W 35 DTH hammer, except its small size, is similar to other hammer variants of Wasara. Design and operating principle, as described in chapter 5.3, are also comparable. The main differences lie in the smaller diameter, whereby the internal parts had to be adjusted, the hammer frequency and especially a reduced flow rate. According to this, the operating frequency is 110 Hz at 180 bar working pressure for drilling in hard rock with a compressive strength $> 200$ MPa as Swedish granite, for instance. Unlike the frequency, the needed water flow is reduced due to the reduced internal hammer volume.
This prototype hammer is currently undergoing different field tests. Based on these drilling investigations a reference project for application of the complete LHC in embankment structures has been implemented. Additional information on this project can be found in chapter 6, followed by the presentation of analyses and results in chapter 7.

5.4.1 Construction Details of the Lifting Device

The purpose of this section is to describe and illustrate the lifting device, which has been mentioned in the introduction of chapter 5.4.

This lifting equipment consists principally of a threaded rod, a connection device at the upper end of the rod and a female part (sleeve) of the catch equipment at the lower end. Principally, the catch equipment works like a slot and key system. In detail, the lifting device, illustrated in Figure 47, consists of the following elements:

- Sleeve (1),
- Locking arm (2),
- Cylindrical pin (3),
- Guiding and connection device (4),
- Threaded rod (5),
- O-ring (6)

![Figure 47: View and cross-section of the lifting device (Wassara AB 2012a).](image-url)
The following Figure 48 shows the perforated drill pipe (filter pipe) including a threaded joint to a connection pipe towards the hammer on the left hand side. Furthermore, the lifting equipment with a connected steel cable (o.d. 3 mm) is illustrated in Figure 48. On the right hand side of the filter pipe the male part of the slot and key system is visible. This hollow plug connection (Figure 49) is mounted on a small pipe which enables a controlled feeding of water towards the DTH hammer. Between this small interior pipe and the perforated drill pipe the flow tube is arranged, see Figure 49.

Figure 48: View of the filter pipe and lifting device that is connected to a steel cable.

Figure 49: Hollow plug connection on the filter pipe (left); detail of filter pipe and installed flow tube (right).
5.4.2 Suitable Dam Instrumentation

This subchapter describes a measurement device for determining horizontal displacements of soil piles respectively engineered constructions, installable in the drill rods (i.d. 28 mm) of the LHC, see Figure 50. This instrument of the Canadian company Measurand is called ShapeAccelArray (SAA). This array uses tri-axial chip based accelerometers, located either in 0.5 m or 0.3 m intervals within connected segments covered by a waterproof layer (Rollins et al. 2009). It is available up to a length of 100 m and can be installed in any kind of conduits with a minimum inner diameter of 27 mm (Measurand 2012). Thereby, the upper end of this array has to be fixed at the pipe by a clamp, for instance, to enable a fixed position above ground.

Relating to the LHC, drilling down to a stable formation without horizontal movements is recommended in context with installation of SAA (red line, Figure 50), also described for standard inclinometer systems in section 4.2. In complementary to the recording of magnitude and direction, vibration and temperature data area are also recorded at each increment of eight segments. These additional parameters are logged, by the use of a segment length of 0.5 m, every 4 m along the installed array and are processed according to the principle in Figure 51.
The temperature data along the array enables the detection of seepage water respectively the scope seepage changes within an embankment dam for instance. This is possible due to the interconnection of water ($\lambda \approx 0.56 \text{ W/mK}$) inside of the LHC pipes and the surrounding water gradient. Furthermore, the heat conductivity, $\lambda$, of steel with approximately 46 W/mK encourages the detection of temperature anomalies along the borehole wall. These temperature anomalies in turn can be used for seepage estimations. On the other hand, vibration information enables the determination of influences on embankment dams due to earthquakes for instance. Another advantage compared to commonly used inclinometer is the possibility of reuse. Due to a non-existing permanently installed guide casing, unlike normal inclinometers, this measurement system can be usually pulled up and reinserted in another observation hole.

According to Rollins et al. (2009), SAA provides a comparable precision as standard inclinometer systems, described in section 4.2. This complies with information based on long-term field measurements (two years) on repeatability and resolution of the producing company Measurand. Thus, repeatability of 20 arc-seconds (0.1 mm/m) with a resolution of 2 arc-seconds (0.01 mm) by using a 32-bit data system could be determined. Figure 52 presents an example of measured soil movements during a slope monitoring project in a highway embankment.

Figure 52: Example of measured soil movements in a highway embankment (Dasenbrock 2010).
6 Reference Project

This chapter presents an overview of the reference project implemented in collaboration between Luossavaara-Kiirunavaara (LKAB), Wassara AB, Sweco Infrastructure AB and Luleå University of Technology in 2012. The aim was to make a direct comparison between hydraulic top hammer drilling, presented in section 5.2, and the new Lost Hammer Concept which includes a water powered DTH hammer of Wassara, as described in chapters 5.3 and 5.4, in embankment structures. In this context, this comparative study aims to examine the influence of the drilling process on a certain soil characteristic in connection with its use as filter material in embankment dam structures. Drillings and measurements were conducted in Malmberget (Sweden) on the mine site of LKAB within the area of the tailings impoundment. On site, the reference project has been carried out in two steps and on two different sites (test fields) with a variety of objectives. The following sections include descriptions of the test fields, characteristics of the test material used in the embankment, explanations of the used drilling setups and measurement equipment as well as the documentation of the drilling and measuring work.

6.1 Test Field I

Test Field I (Figure 53) represents the main field for drilling and measuring work in an artificial dam respectively levee, which has been prepared for this research project.

The embankment was constructed on the mine site of LKAB in Malmberget, see Figure 54. To be more exact, it is located on the north-west area of the tailings dam and created around the GPS-Position (X-7458750,966; Y-167377,314; Z-365,835), according to the Swedish coordinate system SWEREF 99 20 15 - RH2000.
After completing the construction work the test field and the positions of boreholes and in-situ soundings has been measured and marked, see Appendix A. Prior to drilling and measurement work started, an inspection and kick-off meeting with all people involved took place on site.

Further details on the nature and extent of the preparation work, the size of Test Field I and the characteristics of the used material can be found in section 6.1.1 and 6.1.2. Within this field, boreholes with a depth of 3 m were drilled with different drill setups and inclinations, see section 6.1.3. Furthermore, soundings before and after each drilling were conducted as well as sampling from different depths to analyse the influence of each drilling technology respectively drill setups.
6.1.1 Test Field Construction

The main test field with a building area of 800 m² (40 m x 20 m) and a height of 3 m, i.e. > 2400 m³ was built up in layers, see Figure 55. For the construction and pre-compaction of the material a Volvo L350F wheel loader with an operating weight of 54 t and a bucket capacity of 12.7 m³ was used. This field in form of an artificial dam body or levee built up of filter material was founded upon a compacted ground with a plane surface, illustrated in Figure 56 and Figure 57.

![Figure 55: Top view and cross-section of the planned test field.](image)

![Figure 56: Construction of the first layer: material delivery, material distribution and pre-compaction.](image)
Figure 57: Construction of the second layer.

For the construction and compaction ramps with an inclination ratio of 1:6 have been used, see Figure 58. The other sides of the test levee indicates a natural slope angle of approximately 45° which represents normally, after Kjærnsli et al. (2003), roughly the angle of friction for the material.

Figure 58: Finished construction of Test Field I. See also Figure 59.

The layers thickness used was 0.60 m before compaction with a 6 t vibratory roller pulled by a wheel loader. The compaction of each layer was performed with 6 passes without adding water at a speed around 2 kilometres per hour to achieve an optimal rate of compaction and a layer thickness of 0.50 m, see Figure 59. This approach reflects the recommendations for construction and compaction of filter and drain zones in embankment dams as described in section 2.2.3 after Kjærnsli et al. (2003).
The material used for construction is remaining rock material from the mining process in Malmberget with a grain-size distribution less than 100 mm and a high proportion of fines as well as a low percentage of stones. Soil parameters have been determined on the basis of rubber balloon tests, in-situ soundings on site and analyses of samples in the laboratory of Luleå University of Technology. The following section describes these tests and the determined characteristics of Test Field I.

### 6.1.2 Soil Characteristics

In order to describe the properties of the soil/material used for construction of Test Field I in detail, field investigations on site and laboratory analyses at Luleå University of Technology were carried out. These included, among other things, soil sampling of approximately 30 kg in the middle of the test field at a depth of 30 cm below ground surface in order to determine the Proctor density and the grain-size distribution by a wet sieving analysis.

The soil consisted of a mixture of coarse to fine particles including voids. The majority of soil particles, after Bardet (1997), are of mineral origins and the voids are filled with water or air respectively with a combination of these mediums (Figure 60). The degree of soil compaction basically depends on the water content and the grain-size distribution and the compaction method used i.e. compacting energy. By implementation of a compaction test, by literature known as Proctor test, the air fraction of a soil sample is reduced whereby the water content is not affected due its incompressibility (Bardet 1997).
In practice, the proportion of air cannot be reduced totally by compaction effort (Bardet 1997). Nevertheless, the ratio of moisture in soil affects the degree of compaction.

Thus, this laboratory analysis enables the determination of optimal water content (OWC) and a maximum dry density (MDD) for a soil sample (Bardet 1997). This special level of dry density is also known in international literature as “Proctor Density”. By plotting dry density versus water content a Proctor curve becomes visible including one (usually) peak point that reflects the point of intersection between OWC and MDD. In the field of moraine cores for embankment dams, for instance, a maximum moraine water content of 3 % above the optimal water content on the “wet site” (upstream) is recommended to ensure core reliability, after RIDAS (2012). Additional general embankment dam characteristics are described in section 2.2. The modified heavy Proctor test has been implemented by a compaction machine with a compaction cylinder. Each of a total of five soil layers were compacted with 25 blows by a hammer with a weight of 4.5 kg and a drop height of 45.7 cm. Altogether five compaction tests with different water contents were carried out. In this process a MDD value of 2.44 g/cm³ (Proctor density) and an OWC value of 6.4 % were determined. A Proctor compaction curve with the maximum dry density on its peak point for the embankment material is illustrated in Figure 61.

![Figure 60: Compaction test: Impact of compaction in a soil volume (Bardet 1997).](image)

![Figure 61: Proctor curve for Test Field I.](image)
Analysing the grain-size distribution, also known as gradation, of granular material by wet or dry sieving is a commonly used procedure in the area of civil engineering. The main objective is the determination of different percentages of grain sizes within a soil sample (Bardet 1997). The grain-size distribution curve of a soil probe is determined by weight of passing particles through different sieve layers, see Figure 62. For the sieving different mesh sizes were used in the following order: 20.0 mm, 11.2 mm, 5.6 mm, 2.0 mm, 1.0 mm, 0.5 mm, 0.25 mm, 0.125 mm and 0.063 mm.

The proportion of fines less than 0.063 mm, normally analysed with a hydrometer or the pipette method after Bardet (1997), was not analysed. As a consequence of a visible high percentage of plastic fines wet sieving were carried out. Thereby, an unused proportion of the soil from the sample used for the proctor test was dried for 24 hours in an oven at 105°C. After implementation of the sieving the retained soil at each sieve layer has been washed and dried in small marked basins for 24 hours at a temperature of 105°C. Thereafter, weightings were carried out again and the cumulative weight has been used for presenting the grain-size distribution. An error-analysis in terms of identical weights was considered by the comparison of the accumulated weight and the initial sample weight (Bardet 1997).

All values can be plotted as a grain-size distribution curve on a graph with the cumulative percentage of passing particles on the vertical axis and the logarithmic scale of particle sizes on the horizontal axis. Furthermore, the resulting curve can be divided into uniform, well graded or poorly graded grain-size distribution types (Bardet 1997).
In this context, the coefficient of uniformity, $C_U$, and the coefficient of curvature, $C_C$, were calculated for a generally description of the shape and slope of the particle size distribution curve (Bardet 1997):

Coefficient of uniformity [-]:

$$C_U = \frac{D_{60}}{D_{10}}$$

(7.1)

where:

$D_{10}$ = grain-size; suffix represents the percentage by weight of the material that is finer than the determined diameter [10 %],

$D_{60}$ = grain-size; suffix represents the percentage by weight of the material that is finer than the determined diameter [60 %],

$C_U$ = Average slope of distribution between 10 % and 60 %. $C_U = 1.0$ is the smallest possible value and refers to a well uniform composition of comparable grain sizes.

Coefficient of curvature [-]:

$$C_C = \frac{D_{30}^2}{D_{10} \cdot D_{60}}$$

(7.2)

where:

$D_{10}$ = grain-size share; suffix represents the percentage by weight of the material that is finer than the determined diameter [10 %],

$D_{30}$ = grain-size share; suffix represents the percentage by weight of the material that is finer than the determined diameter [30 %],

$D_{60}$ = grain-size share; suffix represents the percentage by weight of the material that is finer than the determined diameter [60 %],

$C_C$ = Shape/ curvature of the distribution curve.

After literature, a well graded gravel exists when $C_U > 4$ and $1 < C_C < 3$ (Bardet 1997).

Calculated values in the context of grain-size distribution are presented in the following Table 8. The corresponding particle size distribution curve is illustrated in Figure 63.

**Table 8: Gradation results for Test Field I.**

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Percentage of retained [%]</th>
<th>Grain-size share</th>
<th>Particle size [mm]</th>
<th>$C_U$</th>
<th>$C_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>36.0</td>
<td>$D_{10}$</td>
<td>0.063</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>54.8</td>
<td>$D_{10}$</td>
<td>0.50</td>
<td>79.4</td>
<td>0.79</td>
</tr>
<tr>
<td>Fine</td>
<td>9.2</td>
<td>$D_{50}$, $D_{60}$</td>
<td>5.00, 2.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$D_{15}$, $D_{30}$</td>
<td>0.17, 0.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$D_{90}$</td>
<td>17.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
A maximum stone size, $D_{\text{max}}$, of 30 mm was determined for sample 1. The $C_C$ value of 0.79 is not within the mentioned range above, which indicates at the first view a not well graded soil. However, by analysing the suitability of this material for the use as a well graded filter material the criteria $\frac{D_{60}}{D_{10}} > 4$ must kept in mind among other criteria in context to the core, see section 2.2.3. A value of 80 could be determined, which represents a well graded grain-size distribution. Grain separations are not expected due to the absence of sharp turns of the distribution curve. By analysing the criteria of maximum diameter for filter material this impression can be confirmed.

In context to the determined $D_{10}$ value of 0.063 mm the corresponding $D_{90}$ value of 17 mm is less than the permitted maximum $D_{90}$ value of 20 mm. Generally, it should be mentioned that suitability analyses must be performed in context to the respective core material of an embankment dam. Nevertheless, this grain-size distribution can be classified as suitable for instrumented test drillings in this study. The higher proportion of sand and fines makes the test field more sensible for influences by drilling operations, whereby obvious effects can be expected. In addition, this soil can be classified according to the Swedish triangular classification chart. Each side of the triangle is scaled into 100 sections, representing the percentage of contents fine, sand and gravel (Karlsson and Hansbo 1984). By using this triangle the soil is named as gravely Sand (gr Sa).
Grain-size distribution analyses including sieving and the calculation of uniformity coefficients were also implemented for all samples taken after instrumented test drillings from excavation pits, as described in section 6.1.6. These results are presented in relation to other analyses and to this basic distribution curve, named Sample 1, in chapter 7.1.3.

Furthermore, four rubber balloon tests were carried out on site at four different points at a depth of 30 cm below ground surface. Thereby, the achieved dry density of the compacted soil can be determined with a rubber balloon apparatus, as illustrated in Figure 64.

![Figure 64: Schematic view of the rubber balloon apparatus (left hand side) (Jantzer 2009) and its use in field I (right hand side).](image)

The overall objective is to control the compaction degree of the same material/soil as investigated by a Proctor test in the laboratory. In this context, the determination of bulk density and water content of the compacted soil as well as the calculation of the dry density is possible by using the following formulas (ASTM D 2167-94 2001):

**Bulk density (wet):**

\[
\rho_{\text{wet}} = \frac{M_{\text{wet}}}{V_h} \quad (7.3)
\]

where:

- \( \rho_{\text{wet}} \) = wet bulk density (in-place wet density) in g/cm³,
- \( M_{\text{wet}} \) = mass of the moist soil from the excavated test hole in g,
- \( V_h \) = soil volume of the test hole in cm³, measured by rubber balloon method.
Dry density: \[ \rho_d = \frac{\rho_{wet}}{(1 + \frac{w}{100})} \] (7.4)

where:
- \( \rho_d \) = dry density (in-place dry density) in g/cm³,
- \( \rho_{wet} \) = wet bulk density (in-place wet density) in g/cm³,
- \( w \) = moisture (water) content of the soil from the excavated hole, expressed as a percentage of the dry mass of soil. \( w = \frac{m_{wet}}{m_d} \)

Degree of compaction (Bardet 1997):
\[ R_d = \frac{\rho_d}{\rho_{pr}} \cdot 100\% \] (7.5)

where:
- \( R_d \) = Degree of compaction on site in %,
- \( \rho_d \) = Maximum dry density (in-place dry density) by the rubber balloon test in g/cm³,
- \( \rho_{pr} \) = Proctor density by a compaction test in the laboratory in g/cm³.

The water content has been determined by soil samples from the excavated small holes for the balloon tests. Great care was taken in sampling and carrying out the balloon test to ensure representative samples. These samples were sealed on site in air-tight sample bags and marked for subsequent laboratory tests.

Calculated single values for the rubber balloon test are given in the following Table 9 and in more detail in Appendix C2. Furthermore, average values determined as a reference for the description of Test Field I are included.

<table>
<thead>
<tr>
<th>No.</th>
<th>Bulk density ( \rho_{wet} ) [g/cm³]</th>
<th>Water content [g]</th>
<th>Water content [-]</th>
<th>Dry density ( \rho_d ) [g/cm³]</th>
<th>Average ( \rho_d ) [g/cm³]</th>
<th>Proctor density ( \rho_{pr} ) [g/cm³]</th>
<th>Degree of compaction [%]</th>
<th>Average degree of compaction [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>2.01</td>
<td>14</td>
<td>0.02</td>
<td>1.98</td>
<td>2.11</td>
<td>2.44</td>
<td>81.12</td>
<td>87</td>
</tr>
<tr>
<td>B2</td>
<td>2.22</td>
<td>20</td>
<td>0.03</td>
<td>2.16</td>
<td>2.11</td>
<td>2.44</td>
<td>88.69</td>
<td>87</td>
</tr>
<tr>
<td>B3</td>
<td>2.07</td>
<td>17</td>
<td>0.02</td>
<td>2.02</td>
<td>2.11</td>
<td>2.44</td>
<td>82.93</td>
<td>87</td>
</tr>
<tr>
<td>B4</td>
<td>2.38</td>
<td>22</td>
<td>0.04</td>
<td>2.29</td>
<td>2.11</td>
<td>2.44</td>
<td>93.83</td>
<td>87</td>
</tr>
</tbody>
</table>

The average in-place density (dry density) of 2.11 g/cm³ differs only moderately from the optimal Proctor density of 2.44 g/cm³, determined with an optimal water content of 6.4% in the laboratory. Hence, an average degree of compaction of 87% could be identified. This value represents a good compaction, taking into account that no additional water has been used during compaction of Test Field I.
The soil samples obtained in balloon tests were also used to determine the specific gravity, \( G_s \), known from literature also as particle density or specific density. The specific gravity specifies the ratio of the weight of a unit volume of a material to the identical volume of a reference volume as water, for instance (Bardet 1997). The pycnometer method was used according to the standard ASTM D 854-92 for determining this soil parameter. For each soil sample two pycnometer tests have been implemented.

Specific gravity \([-\] (Bardet 1997):

\[
G_s = \frac{(W_2-W_1)}{[(W_4-W_1)-(W_3-W_2)]} \tag{7.6}
\]

where:

- \( W_1 \) = weight of a dry and clean pycnometer in g,
- \( W_2 \) = weight of a pycnometer filled with dry soil in g,
- \( W_3 \) = weight of a pycnometer filled with soil and water addition in g,
- \( W_4 \) = weight of a pycnometer filled with distilled water in g.

The degree of soil porosity essentially determines the resistance to dewatering. In other words, the porosity determines the ratio between the volume of cavity (volume filled with air) and the total volume of a certain soil sample. Thus, the porosity affects the density of materials. The porosity has been determined according the following formula.

Porosity \[%\] (Bardet 1997):

\[
n = 1 - \frac{\rho_d}{\rho_s} \cdot 100 \tag{7.7}
\]

where:

- \( \rho_d \) = dry density (in-place dry density) from the field g/cm³,
- \( \rho_s \) = particle density from the pycnometer test in g/cm³.

Average values of five pycnometer tests and a decisive mean value of the specific gravity as well as the porosity were calculated by using the formulas above, see Table 10.

**Table 10: Specific gravity and porosity.**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Average ( G_s ) [g/cm³]</th>
<th>Bulk density ( \rho_w ) [g/cm³]</th>
<th>Dry density ( \rho_d ) [g/cm³]</th>
<th>Porosity [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>2.978</td>
<td>2.013</td>
<td>1.979</td>
<td>33.53</td>
</tr>
<tr>
<td>B2</td>
<td>3.119</td>
<td>2.219</td>
<td>2.164</td>
<td>30.63</td>
</tr>
<tr>
<td>B3</td>
<td>2.953</td>
<td>2.072</td>
<td>2.024</td>
<td>31.48</td>
</tr>
<tr>
<td>B4</td>
<td>3.096</td>
<td>2.257</td>
<td>1.968</td>
<td>36.44</td>
</tr>
<tr>
<td>B5</td>
<td>2.970</td>
<td>2.376</td>
<td>2.290</td>
<td>22.92</td>
</tr>
<tr>
<td>Overall average</td>
<td>3.023</td>
<td>2.187</td>
<td>2.085</td>
<td>31.00</td>
</tr>
</tbody>
</table>
The determined overall average of specific gravity of 3.02 g/cm³ is about 11.85 % higher than of 2.70 g/cm³, usual assumed for typical soil. This increase can be attributed to the enclosed iron ore content in the soil.

Beyond these laboratory analyses, an in-situ weight sounding test (WST) was conducted before drilling in the centre of the test field, as described in section 6.1.5, in order to achieve initial values of the constructed field. Based on this sounding resistance information, a density range and the effective angle of shearing resistance ($\phi'$) have been estimated according to Swedish experience on cohesionless soils, see Table 11. Further details on the objectives respectively on the development of these commonly used in-situ technique can be found in section 6.1.5. A site plan with sampling respectively test positions can be found in Appendix A.


<table>
<thead>
<tr>
<th>Test No.</th>
<th>Used weight [kg]</th>
<th>Average resistance over 3 m [half-turns /0.20 m] (range)</th>
<th>Effective angle of shearing resistance, $\phi'$ [°]</th>
<th>Density index (very soft to very dense)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1 centre</td>
<td>100</td>
<td>125.7 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
</tbody>
</table>

The determined average resistance of 125.7 half-turns/ 0.20 m, over a depth of 3 m, represents a very dense soil. By reverse conclusion this value confirms the determined degree of compaction, presented above.

Finally, it should be noted, that the determined characteristics of Test Field I are assumed to be homogeneous over the whole field for subsequent analyses, although there are variations in reality.
6.1.3 Drilling Operations and Drill Setups

All boreholes within Test Field I and the deep borehole (Test Field II) were drilled with an instrumented drilling rig of Sweco Infrastructure AB. A GM 75 GT drilling rig of Swedish Geomek AB (Figure 65) was used to drill a total of nine boreholes with a depth of 3 m in Test Field I with different drilling technologies, setups and inclinations. Test Field I represents an artificial dam respectively levee with specific soil characteristics as presented in the previous sections. This drilling rig is extremely manoeuvrable and usable for all terrains. Furthermore, it is equipped with a rotating head and a hydraulic drifter (top hammer).

![Figure 65: Used drilling rig: Geomek GM 75 GT.]

In Table 12 general performance data of the used drilling rig GM 75 GT are shown.

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>3200 kg without rods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>3800 mm</td>
</tr>
<tr>
<td>Length</td>
<td>1850 mm</td>
</tr>
<tr>
<td>Width</td>
<td>2100 mm</td>
</tr>
<tr>
<td>Height</td>
<td></td>
</tr>
<tr>
<td>Undercarriage and power pack</td>
<td></td>
</tr>
<tr>
<td>Steel reinforced, rubber tracks, hydraulic tightening</td>
<td></td>
</tr>
<tr>
<td>8 wheels, 2 track pairs, 2 driving motors-brakes, max travelling speed 5km/h</td>
<td></td>
</tr>
<tr>
<td>Capacity of fuel and hydraulic tanks</td>
<td>100 l</td>
</tr>
<tr>
<td>Diesel engine to power two hydraulic pumps</td>
<td>70 kW</td>
</tr>
<tr>
<td>Mast</td>
<td></td>
</tr>
<tr>
<td>Feed</td>
<td>Hydraulic cylinder and chain</td>
</tr>
<tr>
<td>Stroke</td>
<td>2400 mm</td>
</tr>
<tr>
<td>Pull down force</td>
<td>40 kN</td>
</tr>
<tr>
<td>Pull up force</td>
<td>70 kN</td>
</tr>
<tr>
<td>Two spindle</td>
<td>Hammer and coring or drilling</td>
</tr>
<tr>
<td>Mast displacement</td>
<td>600 mm</td>
</tr>
<tr>
<td>Clamp</td>
<td>Single</td>
</tr>
<tr>
<td>Mast lifting</td>
<td>Sideway +/- 10 dec.; back and front 10 dec.</td>
</tr>
</tbody>
</table>
The instrumented drilling tests have been conducted separately in lines depending on the drilling technology respectively the technology in combination with a certain drill setup. Within each line three boreholes were drilled with two vertical holes and one inclined test hole (18°) each, see Figure 66.

Figure 66: Drilling with inclination: W 35 (left hand side) and top hammer with Symmetrix (right hand side).
Generally, inclined drilling applications between 10° and 20° in embankment dams are a very common procedure and cause high demands on the drilling equipment. Further details on every line are described below. In addition, a distance of 4 m between each borehole and line enabled drillings and soundings in undisturbed test material. This security distance is four times larger as the recommended distance of 1 m for geotechnical in-situ investigations, according to Svenska Geotekniska Föreningen (SGF).

Line one represents test drillings with Wassara’s W 35 water powered DTH hammer, drilling component of the LHC, as described in section 5.4. Drillings in the second line have been conducted with a top hammer and the Symmetrix overburden drilling method. In contrast to line one and three, a filter pipe including an insertion of filter sand has been carried out within the second vertical borehole. This procedure has been carried out in the same way with standard equipment as commonly used for water observation wells in embankment dams or other geotechnical soil formations. Line three represents instrumented drillings with a top hammer and a standard OD overburden drilling method. The execution of drillings within line three is identically to line one. The variations between each drill setup (Symmetrix and OD) as well as the general explanations of drilling tools are described below. In addition, a site plan with the borehole positions of each drilling setting can be found in Appendix A.

During all drillings Measurement While Drilling (MWD) technology of Environmental Mechanics AB, following referred to as Envi, has been used to monitor up to eight drilling parameters, as described below in section 6.1.4. Furthermore, data of weight sounding tests were recorded automatically by this logging system, as described in section 6.1.5, and used for analyses.
Drill Setups and Applications

Following, all drill setups and applications used for instrumented drillings within Test Field I are explained exemplarily. Thereby, (1) refers to the water powered W 35 DTH hammer, (2) describes the Symmetrix setup including the filter pipe installation, and (3) refers to the standard OD setup. In contrast to (1), the latter two variants have been used with a top hammer.

1 W 35 (W): Setup 1 refers to Wassara’s water powered DTH hammer for the LHC, which has been used for three boreholes with a diameter of 43 mm along line one. The drill bit with a flat front and semiballistic button/inserts has been used. The hammer casing with a diameter of 39 mm was mounted on drill rods with a diameter of 35 mm. A perforated drill rod, as normally used within the LHC above the hammer, was not used for these short test holes. For further details of W 35 and the LHC, see section 5.4.

Figure 67: Starting point for drilling with W 35.

2 Symmetrix (S): The second setup refers to the Symmetrix drilling method that is commonly used for different overburden drilling applications in combination with different drilling technologies as top hammer and DTH systems. Within this project the Symmetrix drilling and casing system was used with a top hammer. Symmetrix consists of pilot pit that drills the main hole and guides the drill string as well as a casing shoe as a basis for the ring bit (casing bit) (Atlas Copco 2008). The casing bit rotates without rotations of the casing rods by the connection to the rotating pilot drill bit, whereby the area for the casing rods is drilled in advance down the hole.
Thus, the drilling takes place in cooperation of both bits and the casing pipe is pulled down simultaneously (Atlas Copco 2008). After reaching the final depth the pilot bit will be unlocked by a reversed drill string rotation and removed out of the borehole, leaving the casing pipe in the ground, see Figure 68.

![Figure 68: Symmetrix drilling principle (left hand side) (Atlas Copco 2008) and view before drilling (right hand side).](image)

With the simultaneous use of the pilot and ring bit, the drilling and casing process can be accelerated. On the other hand, the distinguished drill bit and the subsequent following casing cause a direct influence of the adjacent soil respectively rock formation by the drilling process and used drill mediums. For the instrumented drilling a pilot bit (o.d. 85 mm) screwed onto drill pipes with a diameter of 44 mm was used. The casing bit (o.d. 115 mm, i.d. 100 mm) was screwed on a casing (o.d. 115 mm) by the use of a casing shoe.

In borehole number one of line two a filter pipe were installed including an insertion of filter sand. This procedure has been carried out in the same way with standard equipment as commonly used for stand pipes (water observation). A filter pipe with a length of 1.18 m (o.d. 50.8 mm) and a filter mesh size of approximately 0.9 mm was installed after a thorough flushing of the borehole. Approximately 60 kg of filter sand (grain-size 1.2-2.0 mm) was used for backfilling the annular space between filter pipe and soil, see Figure 69. A final closing by using a concrete plug above the filter distance was not carried out.

![Figure 69: Filterpipe.](image)
3 Overburden drilling (OD): The third drilling setup refers to the overburden drilling method, abbreviated as “OD”. This cased drilling setup can flexible be used for drilling in varying and sensitive conditions. Within the project the OD setup was used in combination with a top hammer. The system consists of two independent drill strings, see Figure 70. Extension rods for the use as an inner drill string, specially designed to withstand percussive forces of the top hammer and a screwed inner bit. A rotary percussive casing equipped with a casing bit represents the second component of the OD drilling method.

![Figure 70: Inner drill bit (left hand side) and casing bit (right hand side) (Boart Longyear 2012).](image)

Both drill strings can be used for drilling independently. In comparison with the customary used double head rotary-percussive systems, the OD system in Sweden is commonly used with a top hammer. In this context, the inner drill rods as well as the casings can be connected with the top hammer. For the test drillings an x-type faced inner bit (o.d. 79 mm) screwed onto extension rods with a diameter of 44 mm was used. The casing bit (o.d. 115 mm, i.d. 79 mm) was screwed on a casing (o.d. 115 mm) by the use of a casing shoe.

The drill process starts with drilling down the casing up to a certain depth without using any drilling fluids by using the top hammer and rotation of the casing rods. Afterwards, the casing rods are disconnected from the top hammer and the inner drill string is connected for drilling the main area within the casing by using penetration forces and if needed drill fluids in small volumes. Thereby, a certain layer of approximately 50 cm down the hole is undrilled in order to avoid hydraulic connection to the adjacent soil/material.
By using this natural plug in combination with casing rods a controlled flushing is achieved. The flushing medium is transferred to the drill bit within the inner drill string. After the water discharge through the bit the cuttings are flushed out of the borehole as slurry of water and broken rock fragments through the annular space between inner pipe and casing. Thus, this drill setup enables drillings with less disturbances of adjacent soil respectively rock due to its independently applicable drill strings and a more secure use of drilling fluids. For drilling in unstable formation or sensitive structures as embankment dams, for instance, this setup is generally used more frequently than Symmetrix.

In borehole number one of line three a filter pipe were installed including an insertion of filter sand, as described for setup 2 (S).

### 6.1.4 Measurement While Drilling (MWD)

Instrumented drillings for ground explorations also referred to as Measurement While Drilling (MWD), have been implemented in the oil and gas industry for decades (Gui and Hamelin 2004). Within the previous years the use of this technique has increased in the field of on-shore drilling for geotechnical applications, for instance. MWD can be described as an approach for continuous recording, measuring, visualizing and processing of drilling parameters (Bruce 2002). According to this, parameters such as penetration rate, hold back, thrust, torque, rotation speed, flow rate, pressure and of course the corresponding time and borehole depth are usually recorded and processed. The general objective of using monitored drilling data, after Gui and Hamelin (2004), is to qualify soil formations and soil respectively rock properties through several drilling parameters in addition to detailed formation investigations. Furthermore, monitored drilling parameters can be used for quantitative and qualitative investigations of the used drilling technology as well as for the detection of subsurface reaction by drilling operations.

All boreholes within Test Field I and the deep borehole (Test Field II) were drilled with an instrumented drilling rig of Sweco Infrastructure AB. Drilling data were recorded and processed according to the Swedish JB-3 standard of SGF with the data recorder “G1” of Envi.
Thus, up to eight drilling parameters were recorded during each instrumented borehole drilling. Following, each drilling parameter is described briefly including correlations with soil respectively rock properties. The second name of each parameter represents the Swedish translation.

1 Time [Tid]: The time is specified in s/0.20 m and represents the time required to drill 20 cm in soil respectively rock formations. This internal resolution represents a standard configuration to record drilling data at every 20 cm. This high resolution allows a visual control during drilling with a very low speed. Furthermore, this parameter indicates the reciprocal of the average drilling speed over 20 cm. According to Envi, this value is recorded and processed to certain Swedish demand. Thus, this value is slightly different to the ROP (2) after conversion.

2 Rate of penetration (ROP) [Borrhastighet]: This drilling parameter, also known as drilling speed respectively penetration rate, is specified in mm/s according to the JB-3 standard. In the case of deeper boreholes this value is usually indicated in m/min or cm/min. Properties of rock and soil formations as well as the experience of drilling operator have a significant effect on the drillability and therefore also on the ROP. Further details are explained in section 5.3.4. In general, the harder the rock respectively the dense the soil the lower the drill speed, taking into consideration a reasonably constant down-thrust (Gui, Hamelin 2004).

3 Down-thrust [Matningskraft]: This parameter indicates the thrust on the bit which particularly affects the drill speed, specified in kN. According to Gui and Hamelin (2004), the rate of penetration is almost proportional to the down-thrust, also known as feed force. In order to obtain information from the ROP the down-thrust should be kept as constant as possible during drilling. By reverse conclusion this parameter can help to describe the penetration resistance respectively the drillability of ground formations. The bigger the grain-size of the soil respectively the higher the density, compressive strength and abrasiveness of rock, the higher the penetration resistance and the lower the penetration rate. Especially for deep drillings the weight on bit (WOB), calculated by subtracting the down-thrust from the hold-back force, is important for an optimal ROP.
4 Rotation speed [Varvtal]: The rotation of the drill string respectively of the drill bit is specified as revolution per minute (RPM) with the SI unit $1 \text{ rpm} = 1 \cdot (60 \text{s})^{-1}$. More often used is the unit l/min. The aim is to loosen sufficiently large rock fragments that can be flushed as fast as possible out of the borehole with a minimum of energy input. Further details are explained in section 5.3.4. In order to obtain reasonable information in terms of penetration rate and torque a reasonably constant rate of RPM should be used (Gui and Hamelin 2004).

5 Drilling fluid pressure [Spoltryck]: This parameter describes the pressure of the drill fluid, specified in MPa (1 MPa = 10 bar). For drillings in this study water was used for cleaning the borehole and also for driving the W 35 DTH hammer. Hence, the water pressure represents the pressurised water used to drive the W 35 hammer. Thereby, uncompressed water after each hammer operating cycle is used to clean the borehole and flushing out the cuttings.

6 Drilling fluid flow rate [Spolflöde]: Water was used as a drilling fluid on site and the respective flow rates have been measured in l/min. Flow rate values correlate directly to the used drilling technology and possible negative interferences of soil and rock formations. Possible risks of drilling in existing embankment dams and foundations are explained in section 5.1. Nevertheless, a sufficient water supply to drive the DTH system respectively to ensure an effective borehole cleaning is necessary as explained in particular in chapter 5.

7 Standby hammer pressure [Tryck på hammare]: This parameter must not be confused with the parameter of hammer pressure used drilling with a DTH hammer, for instance. It represents a standby pressure of the hydraulic system of the drilling rig and the hydraulic drifter, specified in MPa. (Geomek 2012)

8 Force input [Tryck på motor]: The force input [MPa] represents an indirect measurement of the torque by drilling with rotating drill rods based on hydraulic pressure of the drilling rig. (Geomek 2012)

Based on the recorded data during instrumented drilling, in-situ soundings (6.1.5) as well as sampling and laboratory tests (6.1.6) have been conducted for determining the sphere and proportion of influence by drilling technology. Detailed measurement while drilling information of each borehole can be found in Appendix B.
6.1.5 Weight Sounding

Apart from the drilling works, in-situ weight sounding tests (WST), in Sweden known as “Viktsondering”, were carried out by Sweco Infrastructure AB. After the following description of the sounding process in connection with the test drillings the WST method and the purpose of the use of this equipment is described.

Before beginning the drilling work one WST investigation was conducted in the middle of the Test Field I in order to achieve an initial value of the constructed levee, see Figure 71. Following, WST were carried out for each borehole after drilling to investigate the change of in-situ parameters of the filter material by drilling operation in contrast to the first WST before drilling. As already mentioned above in section 6.1.3, in total nine boreholes were drilled with three different drill setups and two different drilling technologies.

![Drilling rig in position for the first WST in the middle of the test field.](image)

In the area of the first borehole of each drilling technology respectively drill setup WST were implemented with two different distances from the drilled borehole. In order to investigate the sphere and proportion of influence by drilling technology respectively the drilling fluid these tests were realized in steps with distance of 20 cm (Figure 72, left) and 50 cm (Figure 72, right) from the borehole.
Due to the high time requirement for each WST of about 40 minutes and a low difference of the measured sounding values, estimated on site, the amount of WST and the distance from the borehole have been modified for the following investigations. Thus, WST next to the remaining six boreholes has been carried out with a medium distance of 35 cm. WST next to the last borehole of each setup has been carried out with the same inclination of 18° as the boreholes. In general, all weight soundings were carried out in steps between 30 cm and 50 cm with increasing sounding depth. At each step the amount of half rotations has been increased up to the load limited of the sounding pipes by using a weight of 100 kg. Pushing forces between 500 kg and 800 kg have been used to reach the next sounding level.

Weight sounding data were recorded automatically by the measurement equipment of the drilling rig and used for analyses. Furthermore, the WST information next to each borehole, in addition to the MWD data, can be found in Appendix B. Following, background information as well as objectives of this in-situ technique are described.
**Background and Objectives**

The weight sounding test, included in the European standardisation ENV 1997-3.7, is the most commonly used in-situ penetration test in Scandinavia, Finland as well as in Japan (Smoltczyk 2003). It was developed, after Smoltczyk (2003), by the geotechnical section of the Swedish railway administration and was standardized in 1917. This test is usually used for ground investigations in different soil formations in context with other geotechnical applications and construction purposes as foundations and piles, for instance (Smoltczyk 2003). WST can be carried out in most common soils but are, according to Smoltczyk (2003), primarily applied in very soft to stiff cohesive soils and very loose to dense cohesionless soils.

A WST system (Figure 73) consists of a penetrometer build as a screw shaped point with a diameter of 25 mm, different weights (one 5 kg, two 10 kg and three 25 kg), extension rods with a diameter of 22 mm and a handle respectively a rotation head at hydraulic machines (Smoltczyk 2003). By using a hydraulic machine, loads and the number of halfturns can be monitored and processed automatically by electrical sensors (Smoltczyk 2003).

![Figure 73: Overview of a WST system (ENV 1997:3-2000) and a used screw shaped point (right hand side) (Smoltczyk 2003).](image)

The objective of this weight sounding method was to measure the resistance of soil in-situ to the static respectively rotational down thrust of a screw shaped point prior and after drilling (BS EN 1997-2:2007).
According to the European standardization, WST should be carried out as a static sounding in soft soil while a penetration resistance less than 1 KN (a total load of 100 kg) exists. If the value is exceeding this level, the penetrometer should be rotated and the number of halfturns required for 0.2 m penetration (ht/0.2 m) must be recorded. Static loads are available in different steps as 0, 5 kg, 25 kg, 50 kg, 75 kg and 100 kg (BS EN 1997-2:2007).

By using this sounding method different soil properties can be derived from the monitored data to describe the soil resistance. Based on the sounding resistance in halfturns for 20 cm of penetration a density index, the effective angle of shearing resistance ($\varphi'$) as well as the a drained Young’s-Modulus ($E'$) can be estimated for natural cohesionless soils based on Swedish experience (BS EN 1997-2:2007), see Figure 74.

<table>
<thead>
<tr>
<th>Density index</th>
<th>Weight sounding resistance*, half-turns / 0.2 m</th>
<th>Effective angle of shearing resistance*, ($\varphi'$), $^\circ$</th>
<th>Drained Young's modulus*, ($E'$), MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>0–10</td>
<td>29–32</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Loose</td>
<td>10–30</td>
<td>32–35</td>
<td>10–20</td>
</tr>
<tr>
<td>Medium dense</td>
<td>20–50</td>
<td>35–37</td>
<td>20–30</td>
</tr>
<tr>
<td>Dense</td>
<td>40–90</td>
<td>37–40</td>
<td>30–60</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt;80</td>
<td>40–42</td>
<td>60–90</td>
</tr>
</tbody>
</table>

* Before determination of the relative density, the weight sounding resistance in silty soil should be divided by a factor of 1.3.

$^\circ$ Values given are valid for sands. For silty soil, a reduction of 3 $^\circ$ should be made. For gravels, 2 $^\circ$ may be added.

$^*$ $E'$ is an approximation to the stress and time-dependent secant modulus. Values given for the drained modulus correspond to settlements after 10 years. They are obtained assuming that the vertical stress distribution follows the 2:1 approximation.

Furthermore, some investigations indicate that these values can be 50 % lower in silty soil and 50 % higher in gravelly soil. In over-consolidated coarse soil, the modulus can be considerably higher. When calculating settlements for ground pressure greater than 2/3 of the design pressure in ultimate limit state, the modulus should be set to half the values given in this table.

Figure 74: Example of relations between different parameters based on weight sounding resistances (BS EN 1997-2:2007).
6.1.6 Sampling

After each test drilling a walkable pit with a depth of approximately 3.5 m was excavated step by step with an 8 t excavator next to the drill pipe, see Figure 75. Thereby, the bucket was guided along the pipe with a graduated reduction of the distance from 30 cm to 15 cm. The remaining adjacent soil has been removed by the use of shovel (Figure 76, left).

In addition to the WST next to each borehole, as described in section 6.1.5, samples from the adjacent soil of the drill pipes were taken. For each test hole, three disturbed samples, each weighting not less than 2 kg, were taken with a vertical distance of 1 m from the surface and within a radius of approximately 10 cm around the pipe (Figure 76, left). Great care was taken in sampling to ensure representative samples. These samples have been analysed in the geotechnical laboratory of Luleå University of Technology in terms of the grain-size distribution by wet sieving tests as described in section 6.1.2.

Figure 75: Excavation of a pit with a depth of approximately 3.5 m after drilling.

Figure 76: Removal of the remaining soil close to drill pipe, soil sampling and documentation.
The determination of the grain-size distribution after drilling in comparison with detected soil characteristics before drilling operations, as specified in section 6.1.1, besides WST analyses and data of MWD provide a basis for analysing the sphere and proportion of influence by drilling technology. Results and analysis can be found in chapter 7 and 8.

Furthermore, photos and the corresponding depth of the visual effects around the borehole were taken within the pit at each drilling position (Figure 76, right). By the use of a picture processing program the photos of each borehole section were stitched together to a complete borehole length. In addition to MWD and sounding information are presented in Appendix B, grain-size distributions in Appendix C and a view of a few boreholes in Appendix D.

6.2 Test Field II

Test Field II (Figure 77) is not actually a field but a borehole position for the first installation respectively drilling of Wassara’s LHC in a tailings dam. Drilling and installation represent one operation and are no consecutive activities. Thus, after the reaching the final drilling depth the installation of the LHC was completed.

![Figure 77: Position of the deep borehole for long-term measurements.](image)

In contrast to Test Field I, construction and preparation work were not needed before drilling this vertical borehole with a depth of 30 m and a diameter of 43 mm. Furthermore, nor field investigations as implemented in Test Field I were carried out before and after drilling.
Wassara’s W 35 DTH hammer for the LHC was used for drilling, as described in section 5.4. A drill bit with a flat front and semiballistic button/inserts (o.d. 43 mm) was used for drilling. The DTH hammer with a diameter of 39 mm was mounted on the drill rods with a length of 3 m and a diameter of 35 mm. These drill rods have a reduced wall thickness to achieve additional financial benefits, compared to standard drill rods as used for drillings in Test Field I. A reduced wall thickness is possible, because of the unique use of these drill pipes. Thus, the resistance of pipe steal and its threaded connections can be reduced. According to the LHC concept, a perforated drill rod above the hammer was used during drilling respectively the installation of this special stand pipe (observation well), see Figure 78.

During drilling, MWD technology of Envi has been used to monitor eight drilling parameters, as described in section 6.1.4.

The main objective of this first installation of LHC was to figure out whether it is possible to use the prototype DTH hammer W 35, a perforated pipe and special manufactured drill rods, in this new field of application. After drilling, the objective was to pull out the inner flow tube of the perforated drill pipe in order to open the hydraulic connection, as described in section 5.4.1. Furthermore, the recorded drilling information is used for analysing the performance of LHC as well as to determine influences by this drilling technique.
6.2.1 Borehole Position and Dam Characteristics

This borehole is integrated in LKAB’s surveillance concept, compiled in 2011 by the company WSP Civils, which comprised several monitoring boreholes in Malmberget’s tailings dam. To be more exact, this borehole (Figure 79) is located in the C-D-E-F section of the tailings dam and drilled at the GPS-Position (X-7458146,960; Y-168962,062; Z-354.643), according to the Swedish coordinate system SWEREF 99 20 15 - RH2000.

![Figure 79: Location of Test Field II in the C-D-E-F dam at the mine site in Malmberget (Brännström 2012).](image)

The borehole has been positioned parallel to an installed canal (section M1) at the downstream shell of the embankment dam, related to the mentioned surveillance concept, see Figure 80 and Figure 81. This canal is connected to a decant tower within the impoundment to lead the water from the tailings pound into the clarification pond (European Commission 2004).

![Figure 80: Position of the borehole parallel to the cross-section of section M1 (LKAB 2012).](image)
The tailings dam at Malmberget was constructed in 1977 and has been increased several times (European Commission 2004). This tailings dams section C-D-E-F, is constructed as a downstream type, has a current height of 42 m with an inclination ratio of 1:2 for the upstream slope and a ratio of 1:1.5 for the downstream slope (European Commission 2004). The downstream shell (shoulder) is constructed by the use of the material characterised in Table 13 and grain-size distribution curve in Figure 82.

Table 13: Grain-size distribution in the downstream shell (WSP, 2011).

<table>
<thead>
<tr>
<th>Grain-size share</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{15}$ = 4 mm</td>
<td>The suffix represents the percentage by weight of the material that is finer than the determined diameter. The diameter of this soil must be larger than 4 mm.</td>
</tr>
<tr>
<td>$D_{50}$ = 20-50 mm</td>
<td>50% by weight of the material has to be finer than the range of 20-50 mm</td>
</tr>
<tr>
<td>$D_{\text{max}}$ = 300 mm</td>
<td>The maximal stone size is 300 mm.</td>
</tr>
</tbody>
</table>

Figure 82: A typical grain-size distribution curve for used material in the downstream shell.
7 Results

This chapter presents the achieved results from measurements while drilling and in-situ weight sounding information as well as soil analyses in the laboratory. All results are separately presented for each test field, although Test Field II represents only measurement while drilling information.

7.1 Test Field I

7.1.1 Measurements While Drilling (MWD)

Recorded drilling parameters during instrumented test drillings with three different drill setups have been individually analysed for each drill setup by using a spreadsheet program. In addition, a comparison between each setup for different parameters was made. Thereby the focus was on average values of the rate of performance (ROP), down-thrust, and flow rate.

Figure 83 exhibits the overall average of ROP, specified in m/min, as well as the down-thrust, specified in kN, per setup. In computing these values, MWD information of three boreholes for each setup have been taken into account. Setup 1 (W) indicates a 55.6 % higher ROP than setup 2 (S) in context to a comparable down-thrust of both setups. Setup 3 (OD) shows the lowest ROP, which is 40 % lower than setup 2 (S) and 66.7 % lower than setup 1 (W), despite the highest average value of down-thrust.

![Figure 83: Overall average values: Rate of penetration (left hand side), down-thrust (right hand side).](image)

In Figure 84 the process of ROP is plotted in vertical scale to the left and corresponding down-thrust to the right versus depth for each setup. The curves are based on overall average values per setup.
Thus, the differences in respect of penetration rate and down-thrust between each setup, as described above, are visualized with a higher resolution for the entire length of test boreholes. Each curve indicates ups and downs in values during drilling. The highest decrease of ROP in direct relationship to high values of down-thrust appears from a depth of approximately 0.50 m independently of the setup. Single graphs plotted for each measured parameter are shown in Appendix B2.

![Figure 84: Overall average values: Comparison between the process of ROP and down-thrust per setup.](image)

In addition to ROP and down-thrust, overall average values of flow rate (l/min) per setup are shown in Figure 85. In computing these values, MWD information of three boreholes for each setup have been taken into account.

![Figure 85: Overall average values: Flow rate.](image)
In connection with Setup 1 (W) unpressurized water is used to clean/flush the borehole. Setup 1 (W) shows an average flow rate of 22.1 l/min. Setup 2 (S) on the other hand shows an average flow rate of 31.3 l/min. As a result setup 1 (W) indicates a reduction of 29.4 % in flow rate in comparison with setup 2 (S). In contrast to the other setups, no values are available for setup 3 (OD) due to the absence of a continuous water flow during drilling.

7.1.2 Weight Sounding (WST)

Weight soundings tests were carried out after drilling at different distances from the borehole as well as one in the centre of Test Field I (number F1) in order to achieve an initial soil resistance value of this test field, see section 6.1.5.

In total, 12 weight soundings were implemented after drilling, see Appendix A. Based on recorded sounding resistances per borehole a density range and the effective angle of shearing resistance ($\phi'$) have been estimated according to Swedish experience on cohesionless soils, see Table 14. Thus, a soil density index for each WST distance were estimated in context with a calculated average soil resistance over a sounding depth of 3.0 m, see Table 14. Weight soundings next to the drill setup 1 (W) and setup 3 (OD) indicate the highest soil density “very dense”. In contrast, setup 2 (S) represents, with the exception of FS3, a lower density range “dense”.
Table 14: Estimated soil parameters based on weight soundings after drilling and experiences after BS EN 1997-2:2007.

<table>
<thead>
<tr>
<th>Setup &amp; hole No.</th>
<th>WST distance [m]</th>
<th>Used weight [kg]</th>
<th>Average resistance over 3 m [half-turns /0.20 m] (range)</th>
<th>Effective angle of shearing resistance, $\phi'$ [°]</th>
<th>Density index (very soft to very dense)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FW1</td>
<td>0.20</td>
<td>100</td>
<td>80.7 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td></td>
<td>0.50</td>
<td>100</td>
<td>106.8 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td>FW2</td>
<td>0.35</td>
<td>100</td>
<td>88.4 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td>FW3</td>
<td>0.35</td>
<td>100</td>
<td>130.0 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td>FS1</td>
<td>0.20</td>
<td>100</td>
<td>52.5 (40-90)</td>
<td>37-40</td>
<td>dense</td>
</tr>
<tr>
<td></td>
<td>0.50</td>
<td>100</td>
<td>68.8 (40-90)</td>
<td>37-40</td>
<td>dense</td>
</tr>
<tr>
<td>FS2</td>
<td>0.35</td>
<td>100</td>
<td>73.6 (40-90)</td>
<td>37-40</td>
<td>dense</td>
</tr>
<tr>
<td>FS3</td>
<td>0.35</td>
<td>100</td>
<td>117.9 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td>FOD1</td>
<td>0.20</td>
<td>100</td>
<td>85.3 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td></td>
<td>0.50</td>
<td>100</td>
<td>116.3 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td>FOD2</td>
<td>0.35</td>
<td>100</td>
<td>99.8 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
<tr>
<td>FOD3</td>
<td>0.35</td>
<td>100</td>
<td>84.8 (&gt; 80)</td>
<td>40-42</td>
<td>very dense</td>
</tr>
</tbody>
</table>

Monitored WST data and average soil resistances over a depth of 3.0 m in comparison to the first sounding F1 for each borehole respectively setup are presented in Appendix B3 to B5.

In addition to individual average values of soil resistance and an estimated range of soil density, average percentages shown in Figure 86 indicates the change of the soil resistance of Test Field I. To be more specific, the percentages represent the difference in soil resistance between the initial value F1 (before drilling) and the average values of WST next to the borehole. Distinctions between the three drill setups used and test distances from the borehole are taken into account. The values within the third column (Avg. WST 0.20 m, 0.50 m) of the figure represent an average of the first two columns.

Figure 86: Percentage difference in soil resistance between different distances per setup and WST F1.
In context with drill setup 1 (W) and setup 3 (OD), the average values of difference in soil resistance decreased with an increased distance to the borehole. That, in turn, indicates a decreasing influence through the drilling process with increasing distance to the borehole. Thereby, the difference between (W) and (OD) is 3.6 % for a distance of 0.20 m, 7.5 % for a spacing of 0.50 m, 13.5 % for a distance of 0.35 m and 5.6 % for the calculated average of the distances 0.20 m and 0.50 m. Setup 2 (S) exhibits significant higher values, except the value for a distance of 0.35 m.

The process of decreasing differences in soil resistances with increasing distances to the borehole is shown in Figure 87 on the basis of the representative values for the distances 0.20 m, 0.35 m and 0.50 m from Figure 86. A zero-point per setup could be determined for each calculated function respectively forecast trendline (dashed lines). Each zero-point indicates 0 % difference in soil resistance in comparison to the initial value F1. To be more specific, these meter values represents the boundary to an estimated uninfluenced area around the borehole. Setup 1 (W) and setup 3 (OD) indicates a similar zero-point in a distance of 0.49 m to the axis of the borehole. In contrast, setup 2 (S) shows an increased distance of 0.72 m.

Figure 87: Process of percentage difference in sieve passing and calculated linear forecast lines.
7 Results

7.1.3 Grain-size Distribution

Grain-size analyses have been implemented by sieving tests on three samples per borehole in a depth of one, two and three meters next to the borehole wall. For more details see section 6.1.2 and 6.1.6. Thus, the different percentages of grain-sizes within each soil sample could be determined. In total 27 grain-size distribution curves, determining the passing particles through different sieve layers, could be plotted, see Appendix C1.

By using a spreadsheet program, nine average distribution curves (three for each setup) could be determined on the basis of three samples belonging to each borehole number, see Appendix C1. In contrast to the grain-size distribution curve of sample 1, the determined three average curves per drill setup are presented in an overall average curve per setup in Figure 88. The curve of sample 1 represents the unaffected grain-size distribution of soil in Test Field I and serves as a reference curve, as described in section 6.1.2.

![Figure 88: Average grain-size distribution curve for each drill setup.](image)

The course of each determined curve differs slightly from sample 1, independent of its setup affiliation. Thereby, the curve for setup 1 (W) and setup 2 (S) is located below sample 1, unlike setup 3 (OD) which is located above sample 1. Consequently, setup 3 (OD) indicates, in average, an increased percentage of passing, which corresponds to a higher proportion of small to fine particles. The shape and slope of each setup curve is comparable with the curve of sample 1, except a slightly different shape of setup curve OD in the range between approximately 0.60 mm and 10 mm.
In this context, the coefficient of uniformity, $C_U$, and the coefficient of curvature, $C_C$, were determined for a generally description of the shape and slope of the curves. Single values for each sample and average values per borehole have been calculated, see Appendix C1. Further general information is presented in section 6.1.2.

In depth investigations were necessary for detailed analyses of the differences between each average particle size distribution curve in comparison with sample 1. Consequently, the percentage difference in sieve retaining of grains for each drill setup has been calculated on the basis of sample 1, see Figure 89. The percentage change for each mesh size is shown.

![Figure 89: Difference in percentage for each mesh size in the drill setups.](image)

Setup 2 (S) and Setup 3 (OD) indicate generally a similar distribution of percentage difference in sieve retaining between different sizes, whereas setup 3 (OD) shows a higher loss of larger sized grains between 11.2 to 2.0 mm. On the other hand setups 3 (OD) shows a completely opposite trends in the range from 0.25 to 0.063 mm. Setup 1 (W) and setup 2 (S) show a similar pattern with washed out of fines while setup 3 (OD) indicates an increase. Unlike the setups 2 (S) and 3 (OD), setup 1 (W) shows the highest percentage increase for a mesh size of 5.6 mm (4.3 %) and a decreased grain-size distribution between mesh sizes 1.0 and 0.063 mm.
The described differences in sieve test result of grains for each setup, in relation to the grain-size distribution of sample 1, are illustrated in Figure 90 as absolute values. Individual figures for each setup can be found in Appendix C1.

Figure 90: Absolute values of difference in sieve result between the drill setups.

The calculated sum of overall differences for each setup show that setup 3 (OD) indicates the highest value (19.1 %), followed by setup 1 (W) with 17.0 % and setup 2 (S) with 15.9 %.
7.2 Test Field II

7.2.1 Measurements While Drilling (MWD)

During the installation respectively installation of Wassara’s first LHC, referred to as DH1, measurement while drilling technique has been used for recording 30 m of drilling depth. The LHC concept is described in detail in subsection 5.4. Logged parameters were prepared and analysed by using a spreadsheet program as in Test Field I. The focus was directed on average values of the rate of performance (ROP), down-thrust, and flow rate. Certain parameters of DH1 were compared with setup 1 (W) of Test Field I.

Figure 91 indicates the average ROP (m/min) as well as the average down-thrust (kN), over a depth of 30 m. DH1 shows a ROP of 4.4 m/min, which is 38.6 % higher than the overall average of setup 1 (W) in Test Field I. In this context, the down-thrust of 3.1 kN for DH1 is 35.42 % lower than the average value of setup 1 (W) in Test Field I.

In addition, Figure 92 shows a comparison between the process of ROP and the down-thrust of DH1 including a trendline (dashed line) for each parameter over a depth of 30 m. The curve relating to each parameter shows ups and downs during drilling. The trendline of the ROP indicates an exponential decrease with increasing depth. In contrast, the down-thrust shows an opposite trend with a distinct asymptotic increase with growing depth.

![Figure 91: Comparison between DH1 and the overall average of setup W in Test Field I in terms of ROP (left hand side) and down-thrust (right hand side).](image-url)
7 Results

Figure 92: Comparison between the process of ROP and down-thrust and presenting an ROP trendline.

Single process graphs for each measurement parameter can be found in Appendix B6. In purpose of clearness these graphs indicates average values per meter of drilling.

Figure 93 shows overall average values of the flow rate (l/min) of DH1 in comparison with setup 1 (W) of Test Field I. DH1 indicates an average flow rate of 25.2 l/min. Thus, the average flow rate of DH1 is 12.55 % higher than the average value of setup 1 (W) in Test Field I.

Figure 93: Comparison between DH1 and the overall average of setup 1 (W) in Test Field I in terms of flow rate.


8 Discussion

This chapter combines all listed parameter values in order to analyse the sphere and proportion of influence by different drilling technologies on soil particles in context with embankment dams. The analysis starts with a brief discussion on MWD information (subsection 8.1), followed by the analyses of the correlation between drilling parameters and the results of grain-size distributions and weight soundings (subsection 8.2). Finally, discussed aspects are linked to drillings in embankment dams.

8.1 Measurements While Drilling (MWD)

8.1.1 Setup 1 (W)

In the analysis of MWD information it is generally important to discuss single parameters in correlation to each other. Setup 1 (W) in Test Field I indicates the highest average ROP (2.7 m/min), derived from three boreholes, although significant differences between each borehole exists. The third borehole, drilled with an inclination of 18°, indicates the lowest average ROP (1.5 m/min) as well as the lowest down-thrust of 4.2 kN. This is contrasted with the highest ROP (3.9 m/min) in context of a slightly higher down-thrust (4.6 kN) of the first borehole drilled with setup 1 (W). Single diagrams can be found in Appendix B1. Considering, in addition, the average values of each borehole drilled with setup 1 (W) in terms of flow rate and photos of excavated drill pipes, the correlation between these parameter and the influence of adjacent soil becomes clear.

The flow rate of the third inclined borehole indicates, unlike the previous parameter, a decreased value (15.8 l/min) in comparison with identical flow rates of the first two boreholes (25.2 l/min). By analysing these four parameters the significant reduction of ROP despite a comparable value of down-thrust of the third borehole is attributable to a reduced flow rate by 37.3%. Since the corresponding operating pressure is not influenced, it is highly probable that the cleaning performance of the borehole bottom respectively the drill bit and the discharge of water/- cuttings is reduced.
This is once again attributable to the fact that the discharge area has been reduced, because the DTH hammer itself worked without problems. It seems to be obvious that the applied down-thrust was too high to drill an inclined borehole with setup 1 (W). However, this effect is not expected within this soil characteristic. Due to an insufficient bottom cleaning energy is normally lost by recrushing of cuttings. The previous analysis is further evidenced by Figure 94, which shows the influenced soil adjacent to the drill pipes. More photographs are shown in Appendix D1.

![Figure 94: Example of borehole details of setup 1 (W): inclined borehole (left hand side), vertical borehole (middle) and sedimentation of cuttings and fines (right hand side).](image)

In particular, visible observation after excavating of the boreholes, indicate a higher proportion of soil disturbance in terms of flushed away fine grains by acting water along the top surface of the inclined drill pipes. This can be attributed to the natural behaviour of groundwater flow which takes the path of least resistance. On the other hand, both vertical boreholes drilled with setup 1 (W) indicates a significant evenly radius of disturbance of approximately 5 cm around the pipe walls, see Figure 94. Furthermore, all boreholes of setup 1 (W) showed a sedimentation of drill cuttings and rearranged fine soil particles around the drill pipe within the previous influenced/ flushing zone.

Generally, the rearrangement and sedimentation of fine grains by an acting water flow is also described as suffosion. The more uneven the grain-size distribution of soil, the higher the risk of suffosion and larger/- more cavities in the soil structure.
Thus the permeability of the soil and, for instance, the degree of compaction of a certain soil structure can be influenced. An excessive level of down-thrust can enhance this effect by vibration transferred from the drill string to the adjacent soil.

In contrast to MWD information of Test Field I, the drilling respectively installation of DH1 with setup 1 (W) at Test Field II cannot be compared with on-site visual observations below surface nor laboratory analyses. DH1 exhibits a considerably increased average ROP (4.4 m/min) in contrast to a decreased down-thrust by 35.4 % (3.1 m/min) in direct comparison with the overall average of setup 1 (W) in Test Field I. In addition, the existing average flow rate of 25.2 l/min is only slightly increased compared to the overall average of setup 1 (W) in Test Field I. Furthermore, the average flow rate of DH1 is exactly equal to both values of vertical boreholes drilled with setup 1 (W) in Test Field I. As a result of apparently well matching values of down-thrust, RPM and water flow the cleaning performance of the borehole has been improved, whereby the average ROP increased considerably. By analysing the more detailed process lines of ROP and down-thrust of DH1 it becomes apparent that the process of each parameter is levelling off from a depth of approximately 6 m. Visible ups and downs in values are attributable to an irregular subsoil with different grain-sizes up to 300 mm, as described in subsection 6.2.1. General information on used types of stones/ blocks characteristics do not exist. In summary, the drilling process of DH1 went very evenly and controlled with a remarkable low average flow rate and a high ROP.

8.1.2 Setup 2 (S)

Setup 2 (S) in Test Field I exhibits a decreased overall average ROP of 1.5 m/min and a less strongly defined difference between each single borehole. The inclined borehole presents the lowest average ROP (1.2 m/min) and down-thrust (3.7 m/min) as identified above in connection with setup 1 (W). The average ROP of both vertical boreholes drilled with setup 2 (S) is significantly reduced despite a roughly comparable down-thrust applied at setup 1 (W). Single diagrams can be found in Appendix B1 for details. Furthermore, the average value of each borehole drilled with setup 2 (S) in terms of flow rate must be taken into account.
Unfortunately, no flow rate values could be determined for the second borehole. Hence, the computed overall flow rate of setup 2 (W) subject to inaccuracies. Comparing the average flow rate of one vertical borehole (25.2 l/min) with the average flow rate of the inclined borehole (68.7 l/min) an increase of 63.3 % can be determined. Moreover, an increased flushing effort as well as a particular loosened soil formation with cavities adjacent to the borehole (visible after excavation) was noted in the drill protocol. By analysing these parameters it becomes apparent that the lowest average values of ROP and down-thrust as well as the visible disturbance of soil adjacent to the inclined borehole of setup 2 (S) are caused by a significant influenced cleaning performance of the borehole. The nature of this influence of soil respectively the drilling performance is comparable with setup 1 (W), whereas the sphere and proportion is visible increased due to a 77.0 % higher flow rate compared to the inclined borehole of setup 1 (W).

It is highly probable that the upflow of flushing water and cuttings, crushed primarily through the slightly distinguished (eccentric) pilot bit, was prevented by blocked openings between the ring bit and the pilot bit. Hence an increased amount of water was needed to remove the clogging, which in turn enabled suffosion and therefore an unevenly and large radius of disturbance up to 20 cm around the pipe walls was visible, see Figure 95.

*Figure 95: Example of borehole details of setup 2 (S): inclined borehole (left hand side) and influence by installation of a filter pipe in a vertical borehole (right hand side).*
Setup 2 (S) indicates also a higher share of soil disturbance in terms of flushed away fine grains by acting water along the top surface of the drill pipe compared with the lower side. This effect is considerably stronger compared to setup 1 (W). On-site observation of both vertical boreholes drilled with setup 2 (S) shows a more evenly disturbance around the borehole due to a well working drilling respectively borehole cleaning. It should be noted, however, that the occurring suffosion led to a high amount of unevenly distributed small to big cavities next to the vertical boreholes as well.

In addition, the excavated vertical borehole of setup 2 (S) with an installed filter pipe as normally is used for different water observation stand pipes purposes exhibited particularly large cavities and an irregular filling of filter sand along the filter pipe, see Figure 95. After lifting up the drill stin g and the removal of filter sand by hand, a visible share of filter sand stuck on the achieved cavities due increased moisture of soil by acting flushing water. Furthermore, the process of filter sand insertion leads to disjointed sections of filter sand by generated soil plugs vertical to the borehole wall, see the arrows in Figure 95. This is once again attributable to up and downward movements of the casing/ pipe during filter sand insertion. Further figures are shown in Appendix D2.

8.1.3 Setup 3 (OD)

The third drill setup (OD) indicates the lowest overall average ROP of 0.9 m/min and only slightly differences between each single borehole. This parameter is related to the highest values of down-thrust per borehole as well as to an overall average of down-thrust with 8.3 kN. Comparable to the other setups the third inclined borehole of setup 3 (OD) shows the lowest average ROP (0.8 m/min) and down-thrust (6.9 kN). Also vertical boreholes of setup 3 (OD) indicate similar average values of ROP in connection with slightly variation in applied down-thrusts. Single diagrams can be found in Appendix B1. Unlike previous analysed drill setups, no values could be monitored for setup 3 (OD) in terms of flow rate. The measurement device was not available at that time. This fact is attributable to the absence of a continuously water flow during drilling. Consequently, correlations between these values and previous mentioned values of ROP and down-thrust were not possible. By analysing the existing parameters it becomes apparent that the lowest average values of ROP in connection with the highest values of down-thrust are attributable to the absence of water during drilling, the influence of friction as well as to the process of a gradual drilling with casing and pilot bit.
Applied down-thrust forces during drilling cause friction and visible stress marks at soil adjacent to the casing rods/casing bit, see Figure 96. Probably Setup 1 (W) and setup 2 (S), on the other hand, indicate a reduced degree of friction because of a lubricating effect by discharged water from the drill bit to adjacent soil. Referring to drill protocols and on-site observations all boreholes of setup 3 (OD) show a very small proportion on loosen soil formations without noticeable cavities adjacent to the drill pipes. The radius of influence can be specified within a range of 5 to 7 cm. In context with the inclined borehole it becomes apparent that the high effort to clean the borehole bottom is connected with the lowest average value of down-thrust. According to remarks in the associated drill protocol, large quantities of water were used to flush out cuttings within the casing pipe after certain drill intervals. This is once again attributable to bigger drill cuttings respectively an insufficient upflow velocity of the flushing medium.

The excavated vertical borehole of setup 3 (OD) with an installed filter pipe as implemented in context with setup 2 (S) exhibited no noticeable cavities and a less notable irregular filling of filter sand along the filter pipe, see jointed Figure 96. After lifting up the drill sting a removal of filter sand by hand was not necessary and only small share of filter sand stuck on the soil adjacent to the casing pipes. Hence, only the natural moisture of the soil was present without additional flushing water.

*Figure 96: Example of borehole details of setup 3 (OD): inclined borehole (left hand side) and influence by installation of a filter pipe in a vertical borehole (right hand side).*
Furthermore, the process of filter sand insertion appears to be softer to the adjacent soil respectively due to not influenced soil during drilling only a slightly admixtures of soil is visible at the end of the filter pipe. Generally, it is clearly visibly that the filter installation worked very well in context with setup 3 (OD) due to evenly disturbed filter sand along and above the filter pipe having a constant diameter, Figure 96. It seems to be obvious that the process of filter pipe installation and filter sand insertion without previously acting flushing water has less negative effects on the quality of water observation standpipes. Further figures are shown in Appendix D3.

8.2 Weight Sounding (WST) and Grain-size Distribution

In contrast to MWD information, results of implemented weight soundings (WST) on-site respectively grain-size distribution analyses are only available in connection with drillings in Test Field I.

8.2.1 Setup 1 (W)

The evaluation of calculated weight sounding results in term of average soil resistance values over a depth of 3 m for different distances to each borehole drilled with setup 1 (W) shows an increasing average resistance with increasing distance to the borehole axis. That, in turn, indicates a decreasing influence of the drilling process with increasing distance to the borehole. The calculated average soil resistance values for a distance of 0.20 m (80.7 ht/0.20 m) and 0.50 m (106.8 ht/0.20 m) next to the first vertical borehole of setup 1 (W) indicate an increase in average resistance of 24.4 % over a distance of 0.30 m. The second vertical borehole shows a comparable WST value of 88.7 ht/0.20 m. In contrast to both vertical boreholes, the average soil resistance in a distance of 0.35 m to the inclined borehole of setup 1 (W) indicates a value of 130.0 ht/0.20 m, which is higher than the initial value of 125.7 ht/0.20 m (F1, before drilling). Results are shown in Appendix B3 to B5. The comparatively high mean value of WST besides the inclined borehole of setup 1 (W) confirms the assumptions of already analysed MWD parameters and eye observations. This weight sounding were carried out parallel and below to the inclined borehole axis. Thus, the lower proportion of soil disturbance by acting water below the surface of the inclined drill pipe initiated no relevant negative effects on the average soil resistance.
In fact, it is highly probable that the decrease of grain-sizes between 1.0 and 0.063 mm caused the slight improvement of average soil resistance due to the fill of small voids by washed away fine particles.

A zone of influence for each drill setup could be calculated on the basis of overall percentage difference in soil resistance between average WST values for certain distances and the value of F1. Hence, setup 1 (W) indicates a limited sphere of influence to 0.49 m around the borehole axis in the soil structure of Test Field I. Furthermore, analysis of the results of grain-size distribution tests on samples taken after drilling beside of each borehole allow conclusions regarding the change in grain-size distribution by suffosion. Referring to figures in Appendix C1, grain-size distribution curves for each borehole drilled with setup 1 (W) as well as the overall average distribution curve of setup 1 (W) lie below the initial gradation curve of sample 1 (before drilling). Thus, a percentage reduction of passing fine particles is visible. That, in turn, indicates a reduced part of soil particles with a low/ fine diameter retained during sieving analyses. Grain-size distribution analysis of the third inclined borehole of setup 1 (W) shows a major difference in curve shape compared to the curve of sample 1 due to the lowest amount of retained fine particles. Nevertheless, the shape of the overall average distribution curve of setup 1 (W) is comparable to the curve of sample 1.

In addition, percentage differences in sieve retaining of soil in relation to each drill setup were estimated, as presented in subsection 7.1.3 and Appendix C1. The rearrangement and sedimentation of fine soil contents (suffosion) by acting water flow of setup 1 (W) affects the grain-size range between 1.0 and 0.063 mm, identifiable by negative values of percentage difference in sieve retaining. The reduced amount of sieve retaining for a mesh size of 11.2 mm (-1.0 %) in connection with an increased proportion of 2.0 mm (+1.3) is attributable to the drilling/crushing process of setup 1 (W). A significant proportional increase of grains with a size of 20 and 5.6 mm is once again attributable to crushing of existing large rock fragments/ boulders up to 100 mm. Furthermore, the destabilisation of the soil structure, as a result of suffosion and acting flushing water, in combination with vibration due to the reaction between the DTH hammer and the drill bit may cause rearrangements of coarse grain-sizes towards the borehole. Finally, it should be mentioned that already the reduced drilling diameter to 43 mm, which is 62.6 % lower than the outer diameter of the casing bit of setup 2 (S) and setup 3 (OD) minimise the risk of soil disturbance adjacent to the borehole.
8.2.2 Setup 2 (S)

In the surroundings of setup 2 (S), however, the average soil resistance over a depth of 3 m increase to a significant lower extent compared to setup 1 (W) with increasing distance from the borehole axis. Thus, setup 2 (S) indicates a considerable higher zone of influenced soil through the drilling process and a slower reduction of the sphere of influence with increasing distance to the borehole. The calculated average soil resistance values from soundings for a distance of 0.20 m (52.5 ht/0.20 m) and 0.50 m (68.8 ht/0.20 m) next to the first vertical borehole of setup 2 (S) indicate an increase in average resistance of 23.7 % over a distance of 0.30 m. Despite this small percentage increase in average resistance per 0.30 m, a significant reduction in average soil resistance of 34.9 % for a distance of 0.20 m and 35.6 % for a distance of 0.50 m could be determined in comparison with setup 1 (W). The second vertical borehole shows an average WST value of 73.6 ht/0.20 m, measured and calculated in a distance of 0.35 m to the borehole axis. This value is 16.7 % lower than the value for the second borehole of setup 1 (W). The average WST of 117.9 ht/0.20 m in connection to the third inclined borehole of setup 2 (S) indicates only a 6.0 % reduction of average soil resistance compared to the initial average WST value of F1. Details are shown in Appendix B3 to B5. The high mean value of WST besides the tilted borehole of setup 2 (S), as previously indicated in context with setup 1 (W), is attributable to natural behaviour of water flow which takes the path of least soil resistance above the pipe wall. Nevertheless, the slightly reduced average WST value is connected to the lowest monitored average ROP and highest average flow rate of setup 2 (S) and visible unevenly and large radius of soil disturbance. Thus, it becomes apparent that soil structure is more affected below the pipe wall due to acting flushing water in comparison with setup 1 (W). Furthermore, it should be mentioned that in comparison with other drillings and drill setups only the both average WST values besides vertical drilling of setup 2 (S) indicate a lower soil density “dense”.

In addition, setup 2 (S) indicates a calculated zone of influence by drilling of 0.72 m around the borehole axis, which is an increase of 31.9 % compared with setup 1 (W). Based on analysis of the results of grain-size distribution tests the average distribution curve of setup 2 (S) as well as the average single curves per borehole lie below the initial gradation curve of sample 1 with an reduced distance compared to setup 1 (W).
The visible percentage reduction of fine particles becomes particularly obvious at the third inclined borehole of setup 2 (S). In reverse, a reduced part of soil particles with low/ fine diameters retained during sieving is visible which, in turn, is attributable to acting suffosion during drillings.

Referring to presented figures in Appendix C1, the percentage differences in sieve result retaining of soil in connection to the undisturbed gradation curve of sample 1 shows that the soil is affected by the drilling process of setup 2 (S). The rearrangement and wash out of fines affects the grain-size distribution and especially the range between coarse sand (1.0 – 2.0 mm) and the content of fines (0.063 mm). A comparable high average percentage difference in sieve retaining (+ 2.6 %) on mesh size 1 mm is attributable to filter sand residuals as a result of the filter pipe installation in the first borehole of setup 2 (S). Particularly obvious is a significant proportional increase of grains with a size of 31.5 and 20 mm compared to sample 1 respectively values of setup 1 (W). This is once again attributable to crushing of existing large stones up 100 mm in the soil as well as rearrangement of the soil structure towards the borehole. Normally more excessive vibrations of the drill string, by reactions between the drill bit and drill pipes, in combination with flushing water and a larger drilling diameter cause more soil disturbance, leading to a higher permeability and increased influenced zone compared to setup 1 (W).
8.2.3 Setup 3 (OD)

Drillings with the third drill setup, OD, also cause an increased average soil resistance over a depth of 3 m with an increased distance from the borehole axis. However, it becomes apparent that even the average WST value in a distance of 0.2 m to the first vertical borehole of setup 3 (OD) is significantly higher (38.5 %) compared to drillings with setup 2 (S) and slightly increased (5.4 %) related to setup 1 (W). Generally, calculated average WST values for a distance of 0.20 m (85.3 ht/0.20 m) and 0.50 m (116.3 ht/0.20 m) next to the first vertical borehole of setup 3 (OD) indicate an increase in average resistance of 26.7 % over a distance of 0.30 m. The second vertical borehole shows an average WST value of 99.8 ht/0.20 m in a distance of 0.35 m to the borehole axis. This value is 11.4 % higher than setup 1 (W) and increased by 26.3 % compared with setup 2 (S) in relation to each second vertical borehole.

The average WST of 84.8 ht/0.20 m in connection to the third inclined borehole of setup 3 (OD) indicates, however, the highest reduction (32.5 %) of average soil resistance compared to the initial average WST value of F1. Details are shown in Appendix B3 to B5. The described high average WST values in connection with both vertical boreholes of setup 3 (OD), which in turn means that soil adjacent to the borehole is considerably less influenced by suffosion, is attributable to the absence of an continuously water flow during drilling and an efficient handling of the casing. The not expected low value of average WST besides the inclined borehole of setup 3 (OD) is attributable to comparatively high values of down-thrust and the effect of a vibrating and rotating casing. Thus, the larger stones within the soil are ripped during the feeding operation of the casing/- casing bit and deposited elsewhere. During this process, fine (1 mm) to very fine (0.063 mm) grain-sizes are moving due to acting vibrations towards the casing wall and refill the created empty spaces. As a result these soil particles are missing within the grain-size distribution in the area of the implemented weight sounding. Hence, it can be assumed that the porosity has increased, whereby the decreased average value of WST over a depth of 3 m could be determined. Nevertheless, the density index “very dense” for all average WST values in connection with setup 3 (OD) could be derived, as previously determined for setup 1 (W).
Setup 3 (OD) exhibits exact the same zone of influence by drilling of 0.49 m as setup 1 (W) around the borehole axis. Based on analysis of the results of grain-size distribution tests the average distribution curve of setup 3 (OD) as well as the average single curves per borehole lie above the initial gradation curve of sample 1.

This calculated percentage increase of passing fine particle becomes particularly obvious at both vertical boreholes of setup 3 (OD). The increased part of soil particles with low to fine sizes retained during sieving is visible which, in turn, is attributable to the previously described assumption of particle movement towards the casing wall and the refill of small cavities. On the other hand, the drill process with the casing bit in combination with high values of down-thrust acts rather like a grinding and displacement process. Thus, fine cuttings are pressed into directly adjacent soil. Both possible effects in turn influence the grain-size distribution of analysed samples next to each borehole of setup 3 (OD). In this connection, a rearrangement of fine to very fine soil contents is visible due to percentage differences in sieve retaining of soil in connection to the undisturbed gradation curve of sample 1.

Unlike setup 1 (W) and setup 2 (S), a considerably high positive difference in sieve results is visible in the mesh size range of 1 to 0.063 mm, which supports previously discussed assumptions. Furthermore, it can be assumed that the percentage difference in sieve retaining (+1.8%) on mesh size 1 mm is influenced by filter sand residuals as a result of the filter pipe installation at the first borehole of setup 3 (OD). The part of filter sand, however, is considered to be very low. A relatively low percentage difference in sieve retaining of grain-sizes in the mesh size range of 31.5 to 20 mm is due to the fact of less destabilised soil without cavities adjacent to the borehole. Generally it should be mentioned that the grain-size distribution of the used soil in Test Field 1 can be influenced by the construction and compacting process as well as by natural variations due to the mining process and therefore some spread in content is expected. Furthermore, the significance of laboratory results of sample 1 (before drilling) is limited because only one sample was taken from one spot in the middle of the constructed embankment. The comparatively high percentage difference of accumulated weight of particles in the mesh size range from 11.2 to 5.6 mm compared to sample 1 might be attributable to a general change in soil gradation within this drilling area.
Based on results of a general investigation study on drilling technologies in connection with embankment dams, after Perman (2011), the findings of this study clarify the influence of different drilling technologies on soil used in embankment dams. Figure 97 summarizes each analysed drill setup and the corresponding differences between visible effects on adjacent soil after excavation in relation to the radius of influence based on weight sounding and grain-size distribution analyses.

In addition, the grey lines with a curve shape towards the borehole indicate the rearrangement of soil particles due to down-thrust forces, possible drill fluids and vibrations acting on the soil adjacent to the drill pipe. Generally, it can be assumed that the process of rearrangement and the radius of influence along the borehole are depending on the pressure conditions (seepage line) within the embankment structure, as previously indicated in relation with inclined test drillings. Thus, an evenly area of influence adjacent to the borehole axis is depending on a balanced distribution of pressure. Nevertheless, each calculated zone of influence per drilling method for soil in Test Field I, considered as a sensitive soil distribution, may support the process of finding an appropriate distance to sensitive elements of embankment dams.

Figure 97: Visualization of each drill setup and the corresponding visible (blue dotted line) and calculated (red dashed line) radius of influence.
9 Conclusion

The aim of this study was to analyse the impact of three types of drilling methods and their corresponding setups on soil. The methods/setups were: water powered DTH hammer (setup 1), top hammer with a casing drilling system (setup 2) as well as a top hammer with a pre-drilling protective casing (setup 3). Drilling was carried out in a 3 m high embankment consisting of compacted gravelly sand. Complementing to this study, recorded information of drilling one 30 m deep borehole with setup 1 in a tailings dam in Malmberget, Sweden, was evaluated. The main conclusions from this research project are summarised below.

The analysis of average values of soil resistance indicates an average radius of influence from the borehole per drill setup of 0.49 m in connection with setup 1 and setup 3 and of 0.72 m for setup 2. There is a clear distinction between the estimated zones of soil disturbance observed (by eye) on-site and the larger calculated radius of influence.

The existing radius of soil disturbance for setup 3 is, despite the use of a protective casing in order to prevent a hydraulic connection to adjacent soil, attributable to the highest values of down-thrust and casing vibration. Laboratory analyses show that setup 3 registers the highest loss of larger particles and an increased proportion of low to fine grain-sizes. Setup 1 and setup 2 show an opposite trend in a comparable pattern.

Setup 1 exhibits performance advantages in terms of highest average penetration rates in connection with lowest values of down-thrust and lower flow rates. The drilling with setup 1 in the tailings dam shows an improved performance as a result of apparently well-coordinated values of down-thrust. The water powered DTH hammer offers significant advantages compared with top hammer setups: controlled and environmental friendly drillings with reduced complexity in combination with highest penetration rates in soft and hard formations reduced the total drilling time and provide financial advantages.

In summary, the rearrangement of soil particles and the radius of influence along the borehole are, respectively, dependant on the interaction between applied down-thrust, the vibration of the drill string, the amount of drained flushing water towards the adjacent soil and existing pressure conditions within the embankment structure. Apart from differences in drilling technologies it is obvious that education, experience and the reliability of the drilling operator as well as knowledge concerning the subsurface significantly affect borehole safety and the resulting influence on geotechnical structures.
10 Recommended Future Studies

A continuous evaluation of existing drilling requirements and further development of existing drilling technologies is an important safety aspect in connection with drilling applications in sensitive geotechnical structures. An extract of possible and recommended future studies are listed below.

- Drilling investigations in other soil types e.g. moraine respectively formations with different water contents.
- Drilling in a laboratory scale in correlation with theoretical simulations (finite element method).
- Measurement of the existing water pressure around the drill bit.
- Further investigations of the effect of applied down-thrust forces.
- Use of CFD (computational fluid dynamics) software for analysing the fluid flow in connection with different drill bits.
- Further development of drill bit designs.
- Using geotechnical or geophysical methods to detect structural changes in the soil profile around the borehole.
References


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References


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## Appendix B: Drilling and Measured Parameters

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Appendix B: Drilling and Measured Parameters

Appendix B1: Test Field I, comparison of MWD parameters

**Average: Rate of penetration (ROP) per borehole [m/min]**

![Graph showing average ROP per borehole](image1)

**Overall average: Rate of penetration (ROP) per setup [m/min]**

![Graph showing overall average ROP per setup](image2)

**Average: Down-thrust per borehole [kN]**

![Graph showing average down-thrust per borehole](image3)

**Overall average: Down-thrust per setup [kN]**

![Graph showing overall average down-thrust per setup](image4)

**Average: Revolution per minute (RPM) per borehole [1/min]**

![Graph showing average RPM per borehole](image5)

**Overall average: Revolution per minute (RPM) per setup [1/min]**

![Graph showing overall average RPM per setup](image6)
Appendix B: Drilling and Measured Parameters

Average: Flow rate per borehole [l/min]

Overall average: Flow rate per setup [l/min]
Appendix B: Drilling and Measured Parameters

Appendix B2: Test Field I, process of MWD parameters

Overall average: Rate of penetration (ROP) per setup [mm/min]

Overall average: Down-thrust per setup [kN]

Overall average: Revolution per minute (RPM) per setup [1/min]
Overall average: Flow rate per setup [l/min]
Appendix B3: Test Field I, setup 1 (W), 1. borehole

MWD information

1. weight sounding after drilling (distance 0.20 m)

Weight sounding:
Average soil resistance over a depth of 3 m.

2. weight sounding after drilling (distance 0.50 m)

Weight sounding:
Average soil resistance with averaged WST values.
Appendix B: Drilling and Measured Parameters

Appendix B3: Test Field I, setup 1 (W), 2. borehole

MWD information

Weight sounding after drilling (middle distance 0.35 m)

Weight sounding:
Average soil resistance over a depth of 3 m.

Appendix B3: Test Field I, setup 1 (W), 3. borehole, inclination of 18°

MWD information

Weight sounding after drilling (middle distance 0.35 m)

Weight sounding:
Average soil resistance over a depth of 3 m.
Appendix B: Drilling and Measured Parameters

Appendix B4: Test Field I, setup 2 (S), 1. borehole

MWD information

1. weight sounding after drilling (distance 0.20 m)

Weight sounding:
Average soil resistance over a depth of 3 m.

2. weight sounding after drilling (distance 0.50 m)

Weight sounding:
Average soil resistance with averaged WST values.
Appendix B: Drilling and Measured Parameters

Appendix B4: Test Field I, setup 2 (S), 2. borehole

MWD information

Weight sounding after drilling (middle distance 0.35 m)

Weight sounding:
Average soil resistance over a depth of 3 m.

---

Appendix B4: Test Field I, setup 2 (S), 3. borehole, inclination of 18°

MWD information

Weight sounding after drilling (middle distance 0.35 m)

Weight sounding:
Average soil resistance over a depth of 3 m.
Appendix B: Drilling and Measured Parameters

Appendix B5: Test Field I, setup 3 (OD), 1. borehole

MWD information

1. weight sounding after drilling (distance 0.20 m)

Weight sounding:
Average soil resistance over a depth of 3 m.

2. weight sounding after drilling (distance 0.50 m)

Weight sounding:
Average soil resistance with averaged WST values.
Appendix B5: Test Field I, setup 3 (OD), 2. borehole

**MWD information**

Weight sounding after drilling (middle distance 0.35 m)

Weight sounding:
Average soil resistance over a depth of 3 m.

---

Appendix B5: Test Field I, setup 3 (OD), 3. borehole, inclination of 18°

**MWD information**

Weight sounding after drilling (middle distance 0.35 m)

Weight sounding:
Average soil resistance over a depth of 3 m.
Appendix B6: Test Field II, setup 1 (W), borehole DH1

MWD information

Performance comparison: DH1 vs. avg values of drilling with setup 1 (W 35) at Test Field II

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<th>Down-thrust</th>
<th>Revolution per minute</th>
<th>Flow rate</th>
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<td>DH1</td>
<td>Avg W Field I</td>
<td>DH1</td>
<td>Avg W Field I</td>
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Appendix B: Drilling and Measured Parameters

Appendix B6: Test Field II, setup 1 (W), borehole DH1, process of MWD parameters

Average values per meter of drilling: Rate of penetration (ROP) per setup [mm/min]

Average values per meter of drilling: Down-thrust per setup [kN]

Average values per meter of drilling: Revolution per minute (RPM) per setup [1/min]
Average values per meter of drilling: Flow rate per setup [l/min]
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Average: Grain-size distribution per borehole

Overall average: Grain-size distribution per setup vs. sample 1

Sieving analyses: Percentage of retaining per setup vs. sample 1

Effects on soil by drilling: Δ percentage of retaining per setup

Average: $C_U$ per borehole; per setup vs. sample 1

Average: $C_C$ per borehole; per setup vs. sample 1

Single values: $C_U$ and $C_C$ per borehole and sample

<table>
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<td>W1-1</td>
<td>22.00</td>
<td>1.34</td>
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<td>W1-2</td>
<td>44.80</td>
<td>0.91</td>
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<td>W2-1</td>
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<td>W2-2</td>
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<tr>
<td>W2-3</td>
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<tr>
<td>W3-1</td>
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<tr>
<td>W3-2</td>
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<tr>
<td>W3-3</td>
<td>40.00</td>
<td>2.03</td>
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</table>
Appendix C: Laboratory Analyses

Appendix C1: Test Field I, setup 2 (S), 1. – 3. borehole

Average: Grain-size distribution per borehole

Overall average: Grain-size distribution per setup vs. sample 1

Sieving analyses: Percentage of retaining per setup vs. sample 1

Effects on soil by drilling: ∆ percentage of retaining per setup

Average: \( C_U \) per borehole; per setup vs. sample 1

Average: \( C_C \) per borehole; per setup vs. sample 1

Single values: \( C_U \) and \( C_C \) per borehole and sample

<table>
<thead>
<tr>
<th>Sample No. per borehole</th>
<th>Coefficient of uniformity, ( C_U )</th>
<th>Coefficient of curvature, ( C_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-1</td>
<td>80.00</td>
<td>0.64</td>
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<td>S1-2</td>
<td>31.75</td>
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<td>S1-3</td>
<td>16.00</td>
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<tr>
<td>S2-1</td>
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<td>S2-2</td>
<td>32.00</td>
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</tr>
<tr>
<td>S2-3</td>
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<td>0.78</td>
</tr>
<tr>
<td>S3-1</td>
<td>34.78</td>
<td>1.22</td>
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<tr>
<td>S3-2</td>
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<td>1.33</td>
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<tr>
<td>S3-3</td>
<td>46.67</td>
<td>0.47</td>
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</table>
Appendix C: Laboratory Analyses

Appendix C1: Test Field I, setup 3 (OD), 1. – 3. borehole

Average: Grain-size distribution per borehole

Overall average: Grain-size distribution per setup vs. sample 1

Sieving analyses: Percentage of retaining per setup vs. sample 1

Effects on soil by drilling: Δ percentage of retaining per setup

Average: $C_U$ per borehole; per setup vs. sample 1

Average: $C_C$ per borehole; per setup vs. sample 1

Single values: $C_U$ and $C_C$ per borehole and sample

<table>
<thead>
<tr>
<th>Sample No. per borehole</th>
<th>Coefficient of uniformity, $C_U$</th>
<th>Coefficient of curvature, $C_C$</th>
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</thead>
<tbody>
<tr>
<td>OD1-1</td>
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<td>OD1-2</td>
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<td>OD3-3</td>
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Appendix C: Density, porosity and specific gravity of Test Field I

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Depth [m]</th>
<th>( W_0 ) = Mass of empty, clean pycnometer [g] (W.ₜ)</th>
<th>( W_{m} ) = Mass of empty pycnometer + dry soil [g] (Wₜ)</th>
<th>( W_a ) = Mass of pycnometer + dry soil + water [g] (Wₜ)</th>
<th>( W_b ) = Mass of pycnometer + water [g] (Wₜ)</th>
<th>Specific gravity (GS) ( G = \frac{(W_2 - W_1)}{(W_4 - W_1) - (W_3 - W_2)} )</th>
<th>Temp. corrected Specific Gravity of soil [-]</th>
<th>Average Gs [-]</th>
<th>Bulk Density [g/cm³]</th>
<th>Dry Density [g/cm³]</th>
<th>Porosity [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 0.3</td>
<td>206.5</td>
<td>373.24</td>
<td>602.31</td>
<td>491.43</td>
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<td>2,985</td>
<td>2,978</td>
<td>2,978</td>
<td>2,013</td>
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<tr>
<td>B2 0.3</td>
<td>194.18</td>
<td>356.94</td>
<td>592.94</td>
<td>480.35</td>
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<td>3,244</td>
<td>3,119</td>
<td>3,119</td>
<td>2,219</td>
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<tr>
<td>B3 0.3</td>
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<td>378.32</td>
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<td>2,925</td>
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<td>2,290</td>
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</table>
Appendix D: Photos

Appendix D1: Test Field I, setup 1 (W), jointed photos of borehole 1 P. 143
Appendix D2: Test Field I, setup 2 (S), jointed photos of borehole 1 with filter pipe P. 144
Appendix D3: Test Field I, setup 3 (OD), jointed photos of borehole 1 with filter pipe P. 145
Appendix D1: Test Field I, setup 1 (W), jointed photos of borehole 1
Appendix D2: Test Field I, setup 2 (S), jointed photos of borehole 1 with filter pipe
Appendix D3: Test Field I, setup 3 (OD), jointed photos of borehole 1 with filter pipe