Geotechnical input in embankment dam construction and the influence of geosynthetics in the overall design

By

Ian Courtney Fraser-Jones

Thesis presented in fulfillment of the requirements for the degree of Masters Research in Engineering in the Faculty of Civil Engineering at Stellenbosch University

Supervisor: Dr. M de Wet

March 2017

Declaration

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the sole author thereof (unless to the extent explicitly otherwise stated), that reproduction and publication thereof by Stellenbosch University will not infringe any third party rights and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

March 2017

Ian Courtney Fraser -Jones

Acknowledgements

With all good things in life, you need a good support structure around you and thus it needs to mention that this thesis would not have been completed without the help of others. I therefore would like to express my deepest thanks and gratitude to the following people and places:

1. My supervisor, Dr Marius De Wet, for his guidance and input throughout this study.
2. To Colin, Gavin and Eric, the laboratory staff of the University of Stellenbosch Geotechnical laboratory, for their assistance, help, patience and the odd joke while conducting the various tests.
3. My parents and brother, for the constant support and financial assistance over my university career.
4. To the farm, Stone house Estate, for a place to conduct the various tests, collection of samples and for providing assistance in sampling of the material.
5. My friends that I have made over the years, for laughs and allowing me time off studies, when it was needed just to have a braai and relax.
6. And finally to the town of Stellenbosch and the University of Stellenbosch, for providing me an amazing platform and place to conduct my studies over the past 7 years.

Abstract

Dams have played an important part in history, with some of the earliest civilizations like the Egyptians and Mesopotamians, constructing some of the first ever. Early examples ranged from simplistic earth structures to dams being constructed from masonry and building ruble. Since then the field of dam construction has progressed to include a wide range of designs and uses varying from the norm of holding water for irrigation and human consumption. Earth dams have become one such wide spread design, as the reasoning behind its construction, lies in the abundance of material. And as such, it makes sense that earth embankments make up more than 60% of the worlds constructed dams. As the years have gone by, technology has advanced in many fields of today's world. This is true for one such field synonymous with earth embankments, which is namely geosynthetics, with the earliest inclusion of these products in 1970. From early inclusions as filters, geosynthetics have been changed earth embankment construction drastically, providing key help in solving difficult soil conditions. Ultimately, it has been the study of soil itself that has held the answer to solving difficult soil conditions and have aided in the advancement of geosynthetics technology. Geotechnics, the study and engineering understanding of soil interaction, has thus been an area well worth studying.

Using a farm near Stanford in the Western Cape, the influence of geotechnics and geosynthetics in embankment dams was investigated. This study was conducted to see how the field of earth embankment design, incorporates the information gained from the geotechnical understanding of soil and how geosynthetics have altered how we look at embankment problems. Areas of the design process where looked at where geotechnics was used before, during and after the construction of an embankment dam. These areas where further broken down as a desk study where elements such as various soil types, climate and the various parts of an embankment dam. The next step was assessing the onsite ground conditions and using the results that were gathered to design an appropriate dam. Although not built yet, geotechnical measuring instrumentation was looked at, to assess the design that was built, as well as the steps that had to be taken for site preparation.

From site-specific materials, it was determined that a 13.4 meter high earth clay core embankment dam would be constructed on the specific farm site. The water would be stored at a height of 10.4m and the slope gradients would be 1:2 for the downstream slope and 1:3 for the upstream slope. The core would have slope of 2:1 and a cut-off trench with slopes of 1:1. Geosynthetics would be used as upslope protection, for wave erosion.

Table of Contents

.

List of figures

List of Appendixes

Chapter 1: Introduction

1.1 Background

The farm Stone House Estate, which is located close to Stanford in the Western-Cape, is interested in upgrading its farm irrigation. With the upgrade in irrigation, which includes the installation of 35 hectares of new irrigation, they want to increase the farms water storage capacity by planning the design of a new earth embankment dam. The farm's current irrigation system is gravity fed and thus the system relies on water that is stored within the dam. With the proposed upgrade, in the designing and building of a new embankment dam, the plans are to have new previously un-irrigated fields, being fed by the new system. The new system will also be gravity fed and will be able to reach more fields.

The farm, which is 3.4 kilometers from the van Brakels Stoor fourway crossing, can be accessed via the R326 on the way to Stanford. This 245 hectare farm, which is situated on the banks of the Klein Rivier which splits the farm into two sections, has a majority of the farm along the northern slope of the Akkedisberg. Primarily a dairy farm, the income of the farm can be linked to the amount of milk produced by the farms cows, which is directly linked to the quality of the feed that the cows receive. The preferred feed is that of a mixture of Kikuyu, Rye and clovers which the farm has planted in various pastures. These pastures need water of about 5000m^3 per hectare per month, for the climatic region and are often irrigated twice a week. With each irrigation session the plan is to cover the pastures with about 25mm of water.

Currently water is pumped out of the Klein River, via a pump and water is transported to a 100 000m^3 earth embankment dam, which stores the water used for the pastures. The farm relies mainly in the summer months, November to April, for water out of the river as the region is situated in a winter rainfall area. During the last few months of summer, when the river slowly dries up, the water becomes brackish. When this happens, no water is pumped out of the river and this is normally during a critical time for the pastures as the region experiences high temperatures.

1.2 Problem Statement

Due to the need for more water, as a result of wanting to expand the pastures currently under irrigation, a proposed dam needs to be investigated for the farm Stone House Estate. Various sites on the farm will be investigated to see the viability of each site.

1.3 Motivation for research

The motivation for the research lies in looking at what the geotechnical input is in the construction process of an embankment dam and how the results from the various tests and factors influence the selection of the type of embankment for the proposed site. The research will cover the embankment design process from planning, design and construction phases and see where geotechnics is used to select a certain type feature.

1.4 Research aim

The aim of the research study is to look at how geotechnics aids in the designing of an embankment dam. This means to say that the research will cover all steps taken from the starting research phase of the dam, the second phase being the design to the third and final phase of construction. By looking at these phases, we will be able to see how geotechnics effects each of the phases. The research will also try and see how the development of geosynthetics has influenced the field of dam engineering.

1.5 Research objectives

The objectives of this research study is to see how and where during the design, feasibility and construction phases of an embankment dam, the areas that geotechnics influences the various parts of the above mentioned embankment dam. The research will also show how geosynthetics can positively influence the designing and construction of an embankment dam.

1.6 Limitations of research

The limitations of the study are that the research will mainly focus on one type of embankment dam, which will be embankments mainly constructed from earth. Initial definitions may be given regarding certain aspects where there are referred to more than one type, but will then focus on the main aspects of an earth embankment dam. Another limitation is that the soil assessed, will be only soil prevalent to the site that the farm is situated itself in. Some tests mentioned in the design of the dam, won't be done, due to limitations of equipment at the laboratory. Some of the tests not completed, will be accompanied with literature, to support them as tests for embankment dam design. This will be done due to limited tests being able to be completed in the Geotechnical laboratory of Stellenbosch University.

1.7 Report layout

This research report consists of the following 5 chapters and is divided up as follows:

Chapter 1: Introduction

The introductory chapter will include vital background information on the research study and also inform the reader about the aim and objectives of the research, any limitations to the research study as well as provide the motivation behind the research study.

Chapter 2: Literature review

The literature review will look at embankment dams in general and then focus on the types of earth embankment dams. It will cover the elements involved in a desk study for an embankment dam. The chapter will also look and define geosynthetics and see where in the dam construction process they can be used and utilized.

Chapter 3: Design of a dam

This chapter will cover the various investigations needed for the design of an embankment dam and assess how the geotechnical tests and other surveys help determine the various parts of an earth embankment dam.

Chapter 4: Construction of a dam

This chapter will look at how geotechnical information is utilized during the construction phase of an earth embankment dam and look at how the construction of the dam is monitored during and after construction.

Chapter 5: Conclusion

This chapter will sum up the research that was done and present the conclusions that can be made from the research.

Chapter 2: Literature Review

2.1 Classification of types of dams

"As per the definition of the International Committee of Large Dams, dams built of earth or rocks are called embankment dams." (Hagen, 2015)

Although the above statement gives us one way of classifying a dam, it also depends on the elements used to construct a dam. Dams are often thus classified in terms of the type of use intended for the dam, the type of materials used in constructing the dam or in terms of various Dam Safety regulations.

2.1.1 Dam type, determined by material used in construction

Due to the advances in engineering and the understanding of how certain materials work together, has resulted in various dams being constructed based on material type. Earth and concrete form the backbone of most designs and have the following types of dams associated with each material:

Earth as material:

Embankment Dams

Embankment dams can be considered as being dams built from natural materials, i.e. earthfill or rockfill. "A cross-section (or slice) through an embankment dam shows that it is shaped like a bank, or hill" (Anderson & Robert). As a result, the name of this dam is derived from the shape.

Concrete as material:

Arch Dams

Arch dams are concrete dams and so named after the shape that the dam forms, see Figure 2.1. The top of the arch, points generally in the direction of the water. "An arch is a strong shape for resisting the pushing force of water behind a dam" (Anderson & Robert). This is the main principle, around which this dam is designed. Fell, Macgregor, Stapeldon & Bell (2005) states that the arches forces, are transposed "into the abutment foundation by the arching action and generally impose higher loads on the foundations". Due to the arch shape and forces experienced, the dam is often constructed in "narrow, steep sided valleys" (Anderson & Robert).

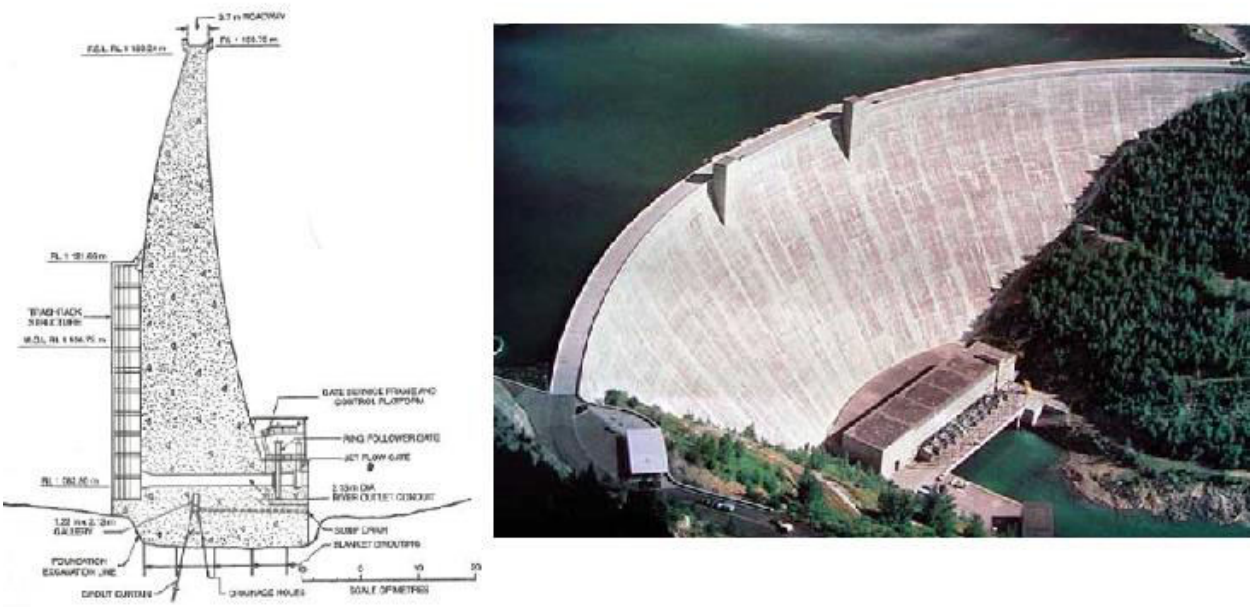


Figure 2.1 Diagram of typical arch dam (Fell, MacGregor, Stapeldon, & Bell, 2005) and (United States society of dams)

<u>Gravity Dams</u>

Gravity dams are so-named, due to the principle on which the dam supports itself; Figure 2.2 shows a typical gravity dam. The dam's mass provides an adequate downwards force due to gravity, which provides support and keeps the foundation down. As a result, the dam supports itself and "prevents loads (or forces) due to the pressure of the water in the reservoir from causing the dam to slide, overturn" (Anderson & Robert). A cross-sectional view provides a general, rough triangle shape. This dam is ideal for wide or narrow valleys, given that the bearing capacity of the foundation is acceptable.

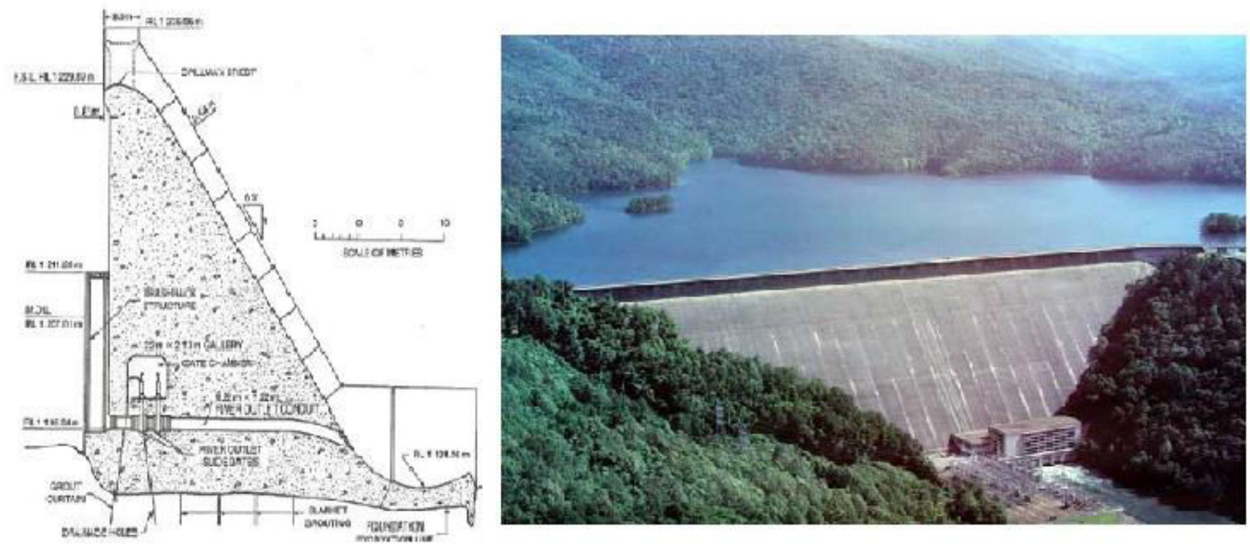


Figure 2.2 Diagram of typical gravity dam (Fell, MacGregor, Stapeldon, & Bell, 2005) and (United States society of dams)

<u>Buttress Dams</u>

Buttress dams are named after the various triangle shaped walls, called buttresses, of which the dam consists of. Refer to Figure 2.3 for reference on what a typical buttress dam looks like. This dam is also constructed from concrete. As with the arch dams, the buttresses provide a water tight boundary, with the triangles providing support to "resist the force of the reservoir water trying to push the dam over" (Anderson & Robert). The theory behind the development of the buttresses dam lies behind the same principles in the gravity dam design, in that the dams own mass supports in from sliding or moving due to water forces. A key advantage to that of the gravity dam, is that the buttresses dam has far less materials in construction, due to space between buttresses. This in return leads to that the rock situated below each buttress, must have suitable bearing capacity to be able to support the load from the buttress. This type of dam is often constructed in wide and narrow valleys.

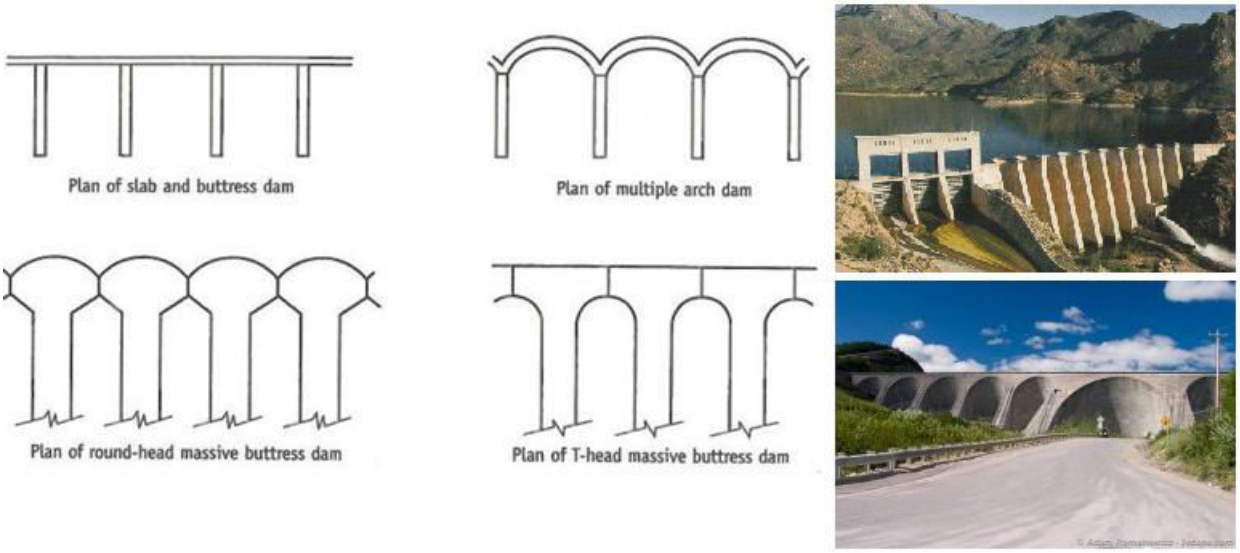


Figure 2.3 Diagram of typical Buttress dams (Fell, MacGregor, Stapeldon, & Bell, 2005) and (United States society of dams)

<u>Mixture of both:</u>

<u>Tailings Dams</u>

Tailings dams are dams that provide mines a disposable site, for generated waste, which has resulted from their mining activities. These dams can be considered in the concrete section, but due to the nature that it provides another type of function, tailings dams can be grouped in a different classification. Although this dam can be made up of just concrete, earth material is often used in-conjunction to add certain filter effects. These types of dams are often designed around specific legislations pertaining to the various hazardous materials that may be in the mining wastes.

2.1.2 Embankment dams

Most irrigation dams that occur worldwide are embankment dams. As defined above, embankment dams are dams which are built of natural material, i.e. earth or rock. The materials used each have unique properties associated with them. The materials used, “derive their strength from position, internal friction and mutual attraction of their particles” (Sowers, 1962).

Before we look at various types of embankments, let’s discuss reasons why we choose earth/rock embankments. There are various reasons why natural materials are considered to that of cement dams. The first would be, that natural materials can deform slightly so that the material follows the natural movement deflection of the foundation, without any failure occurring. Secondly, earth and rock material is a widely abundant resource, i.e. earth and rocks can easily be sourced to places compared to getting cement. Sowers (1962), also states that “with the steadily increasing knowledge of the mechanics of soil and rock even such materials which were once thought unsuitable can now be used”.

The third reason is that earth is easily handled. Onsite materials can be excavated with ease and materials can also be easily be transported to site, if need be. The use of machinery, to help with the process of excavation of materials, has added to ease of working with this material. Another telling reason is that earth/rock embankments are often able to be built, where concrete/cement dams aren’t. The secret of this lies in the density of the two masses. Per cubic foot of material, earth is less dense than concrete, thus “the intensity of the stress resulting from it will be less” (Sowers, 1962). Sliding of the dam is also reduced, due to the wide base of the embankment dam being able to distribute the horizontal forces evenly, across the base. The final and probably the most important reason for earth/rock embankments, is the costs involved. Earth and rock can be used free for on-site jobs, but if the earth is inadequate for embankment construction, cement has to be trucked in. The Katse dam in Lesotho had 687 thousand tons of fly ash cement, trucked into the site. Although this dam is very large, it also provides extra logistical issues.

In earth and rockfill dam engineering, there are disadvantages associated with earth and rockfill dams. On site material may not be suitable and thus may become an economical cost to bring in material”. Sowers(1962) adds that “greater maintenance is required for earth embankments than for good concrete ones”, he also states that “earth embankment usually cannot be used as a spillway”. Spillways are often constructed next to the abutments.

Due to the advantages, normally outweighing the disadvantages when it comes to choosing earth and rockfill dams, the next step is to consider the various types of embankments there are in this sub fields. These variations in type of embankments can be classified in the zonation of the materials used. All embankments are “zoned earthfill type with an impervious core” (Hagen, 2015). Zones of rockfill are also incorporated in some designs and in the case that impervious earth material can’t be found, a suitable geosynthetic can be substituted to provide the same goal. The effects of geosynthetics in dam design will be discussed at a later chapter.

Hagen (2015) summarizes the following as common embankment types:

- Earthfill
- Clay core Rockfill
- Concrete Faced Rockfill
- Asphalt Faced Rockfill

Figures 2.4a to 2.4e and Table 2.1, shows the general zones found in an embankment dam

Table 2.1 Key to the figures below (Fell, MacGregor, Stapeldon, & Bell, 2005)

Zone	Description	Function
1	Earthfill/"core"	Controls the seepage through the dam
2A	Fine filter/filter drain	(a)Controls the erosion of Zone 1 by seepage water,(b) controls erosion of dam foundation(when horizontal drains used),(c)controls buildup of pore pressure in downstream face when used as a vertical drain
2B	Coarse filter/filter drain	(a)Controls erosion of Zone 2A into rockfill,(b) discharge seepage water collected in vertical or horizontal drain
2C – (i)	Upstream filter	Controls erosion of Zone 1 into rockfill upstream of dam
2C-(ii)	Filter under rip rap	Controls erosion of Zone 1 through rip rap
2D	Fine cushion layer	Provides uniform support for concrete face; limit leakage in the event of the concrete face cracking or joints opening
2F	Coarse cushion layer	Provides uniform layer support for concrete face. Prevents erosion of Zone 2D into rockfill in the event of leakage in the face
1-3	Earth Rockfill	Provides stability and has some ability to control erosion
3A	Rockfill	Provides stability, commonly free drainage to allow discharge of seepage through and under the dam. Prevents erosion of Zone 2B into the coarse rockfill
3B	Coarse rockfill	Provides stability, commonly free drainage to allow discharge of seepage through and under the dam.
4	Rip rap	Controls erosion of the upstream face by wave action and may be used to control erosion of the downstream toe from backwater flows from spillways

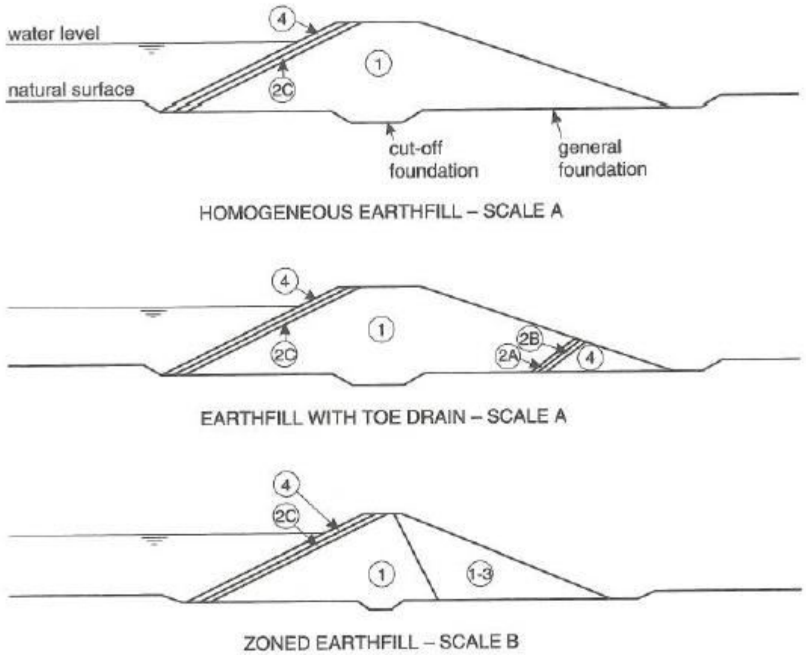


Figure 2.4a Diagram of typical embankment dams (Fell, MacGregor, Stapeldon, & Bell, 2005)

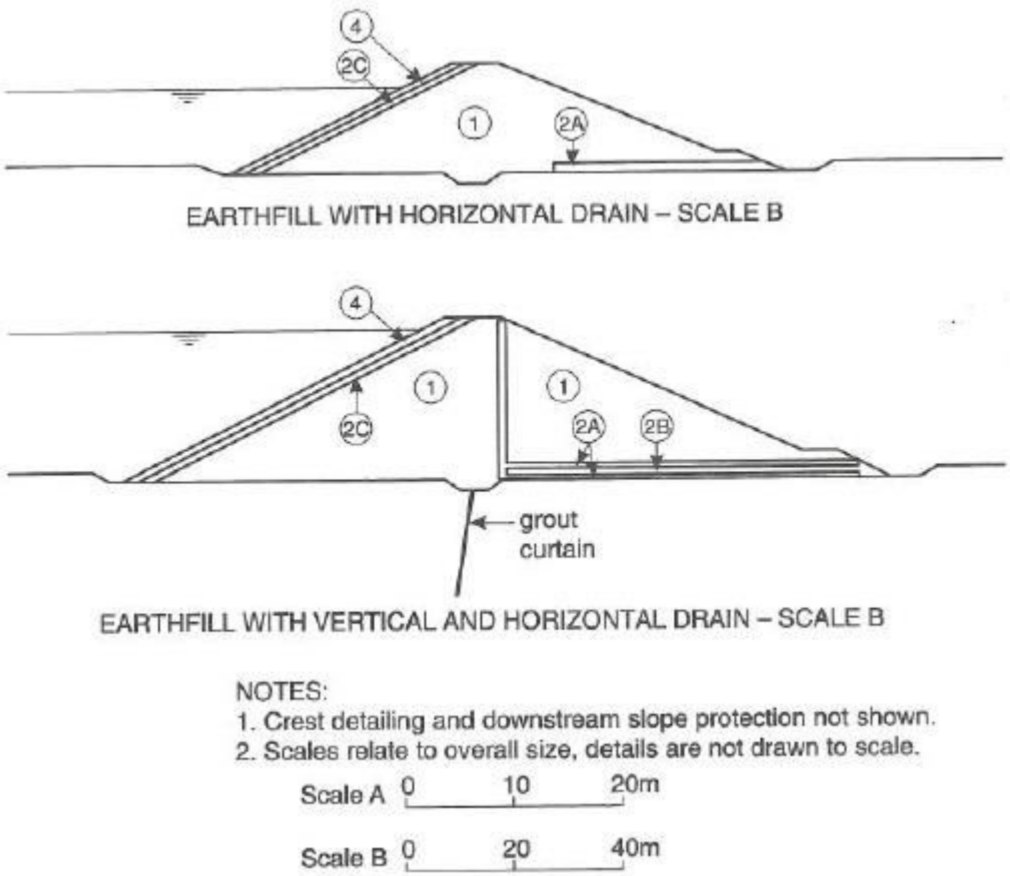


Figure 2.4b Diagram of typical embankment dams (Fell, MacGregor, Stapeldon, & Bell, 2005)

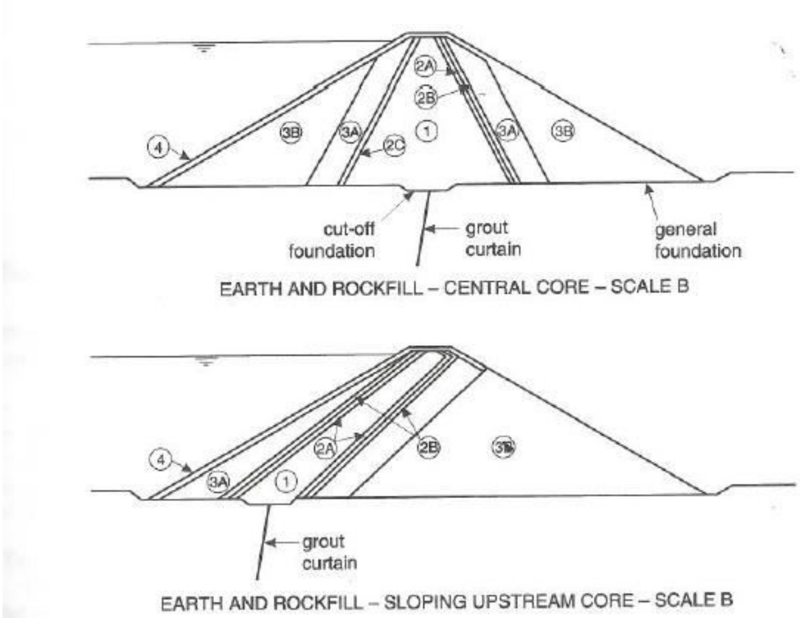


Figure 2.4c Diagram of typical embankment dams (Fell, MacGregor, Stapeldon, & Bell, 2005)

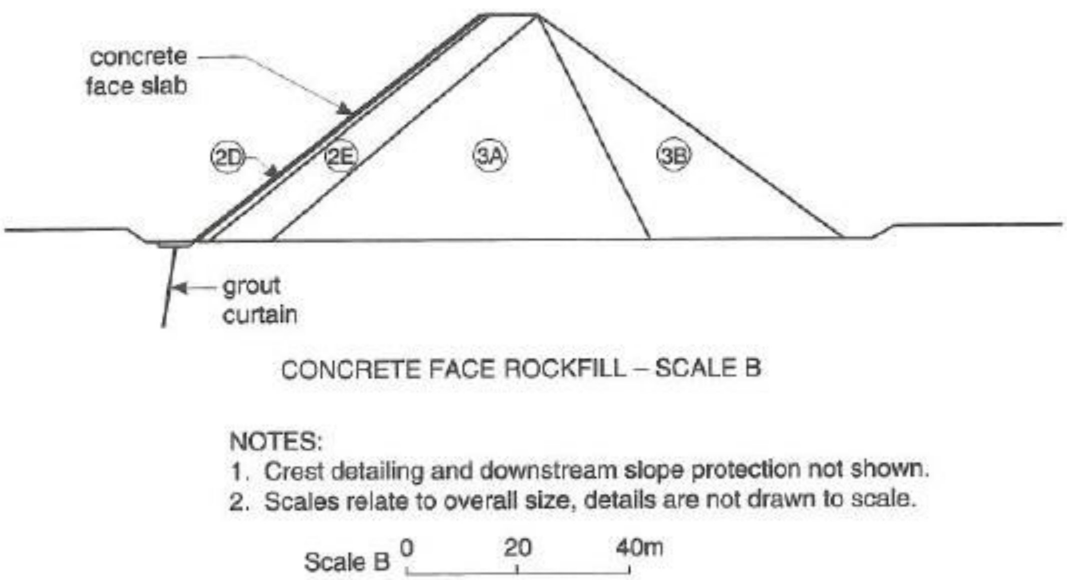


Figure 2.4d Diagram of typical embankment dams (Fell, MacGregor, Stapeldon, & Bell, 2005)

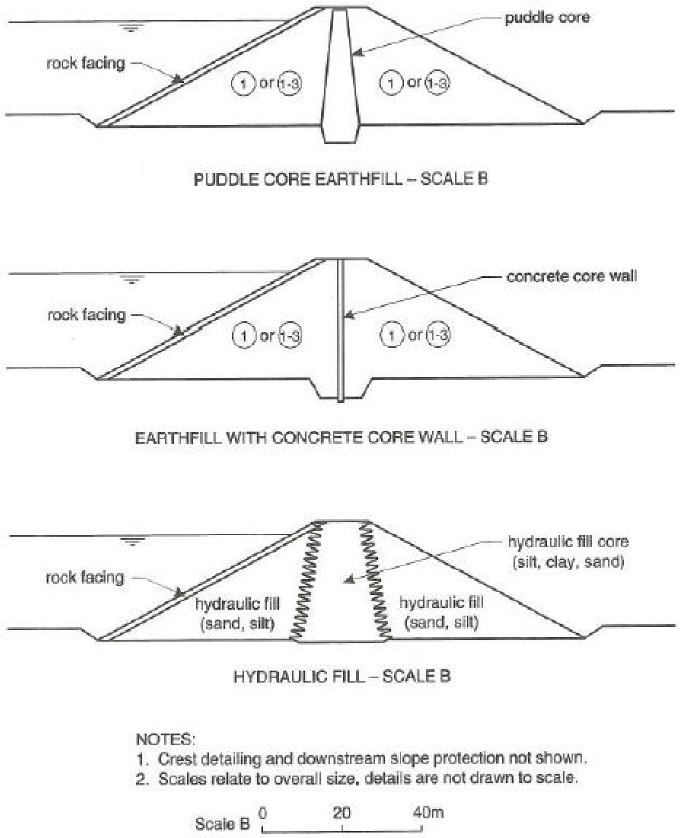


Figure 2.4e Diagram of typical embankment dams (Fell, MacGregor, Stapeldon, & Bell, 2005)

The above Figures show that a wide variety of embankment dams can be constructed, with them containing some or most of the elements mentioned in Table 2.1. Each of the designs above is dependent on various factors and the decision to choose a specific design, depends on the factors that will be discussed in the following section.

Now that we have some definitions of the different types of embankment dams, we have to look at how one goes about designing an earth embankment. Narita (2000) outlines that there are three important steps to consider when approaching a project like embankment dam design, which are namely "Investigation, Design and Construction". These three steps are crucial in the process of construction of an embankment, with each step assisting the next one. This is outlined by the flow diagram in Figure 2.5. Each step can be further expanded regarding the steps taken within each phase of the planning, but these will be discussed within the relevant sections.

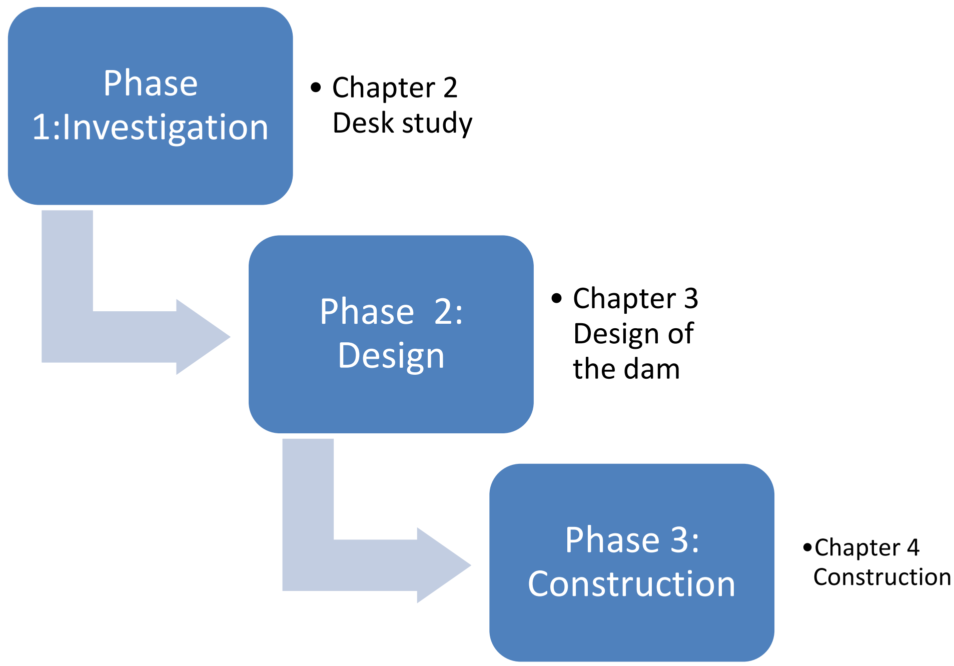


Figure 2.5 Planning an embankment dam

2.2 Desk Study

Before a dam is rushed and built, a series of factors need to be investigated. This investigation is termed the desk study and provides valuable information in deciding the design and building procedure. The process of the desk study is to aid in the selection of a certain type of dam, depending on various site factors. The information gathered can explain situations that could be experienced by onsite engineers. The background knowledge is used in conjunction with the site experiments and tests, to help determine the type and design of the embankment dam.

In a report written for the sixth conference of the British Dam Society (1990), it states that the desk study's aim is "to identify susceptible material" for usage as fill and to assess the foundation of the site. Another paper, written by Narita (2000), topographical, geological, hydrological and meteorological information, is of importance for a desk study. Hagen (2015) points out that spillway type and size, earthquakes and environmental factors are also points to look at. A countries law regarding water usage vary from place to place and therefore has to be considered when conducting a desk study.

The above mentioned factors , are summarized below:

- 2.2.1 Topography
- 2.2.2 Geology of dam site
- 2.2.3 Engineering properties of the geology
- 2.2.4 Founding conditions

- ❖ 2.2.5 Spillway size and location
- ❖ 2.2.6 Earthquake loading(seismic hazard)
- ❖ 2.2.7 Stability of design
- ❖ 2.2.8 Availability of construction materials
- ❖ 2.2.9 Climatic conditions
- ❖ 2.2.10 Environmental considerations
- ❖ 2.2.11 Water license application

2.2.1 Topography

Topography is often very important, as it is sometimes the first way of choosing the type of embankment dam used. Common ways to look at topography is to look at topographical maps, aerial photographs and satellite maps. By looking at the way the site is presented, one is able to consider certain problems that may arise by having the various earth or rockfill embankments. It is important though that you first look at bigger topographical scale maps of the region, as one is able to leave out key geology features, often not seen on the site specific map. By looking at topographical maps, you can often pre-select certain sites for the dam, based on what geological features you can see on the maps. The sizes/scales of the maps may vary, depending at what features you are looking on the maps. From the aerial photographs/maps you are able to see "relationships between the regional geology and landforms, drainage, soils, vegetation and land-use" (Fell, MacGregor, Stapeldon, & Bell, 2005). These relations often help with planning roads to be used on site, locations of possible construction material and assessing the dam's rate of siltation.

One such element can be assessed on maps, is to look out for lineaments. Lineaments as shown in Figure 2.6, as defined by Fell, MacGregor, Stapeldon & Bell (2005) are "linear features or linear arrangements of features that are visible on the photographs". The features mentioned here could indicate faults, valleys, rivers and saddles. An area that has dense vegetation could also be a sign of one of the above lineaments. On the other side the photos and maps, would also be able to show your folded rock group strata's and dipping strata's, which could be determined from the steepness of the sides of the slopes. Once some features have been identified, some similarities can be assessed between certain soils/areas of the site and these can possibly be grouped into various groups. The groups could help asses areas of similar construction materials.

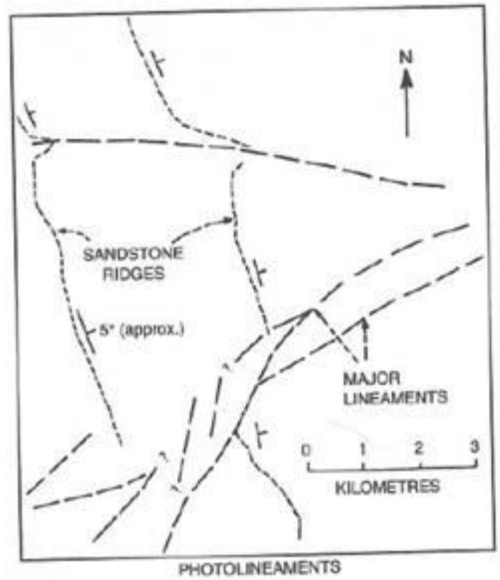


Figure 2.6 The photo on the left shows an aerial photograph and on the right is how these lines can be linked (Fell, MacGregor, Stapeldon, & Bell, 2005)

2.2.2 Geology of dam site

Synonymous with geotechnical engineering is the subject of Geology. Most soils are derived from a parent rock/soil and understanding the specific geology of a soil, will aid in various geotechnical parameters.

As stated, the farm is situated along the R326 and thus finds itself in the Western Cape of South Africa. The farms area falls in a section of the Cape Supergroup, as indicated in Figure 2.7. Figure 2.8 indicates that the present day distribution of the formations may include the Bokkeveld and Table Mountain Groups. These formations are the result of deposition of sediments that occurred in the Agulhas Sea 500 to 330 million years ago, after tension caused a rift to form along the Pan African belt. The first formation of the Cape Supergroup is that of the Table Mountain group. This formation, of which Table Mountain is apart of is, was mainly the result of a shallow marine deposition setting in which numerous rivers deposited sediments. Deltaic depositional features, like fan delta's on the edge of the sea, provides another setting of deposition for this formation.

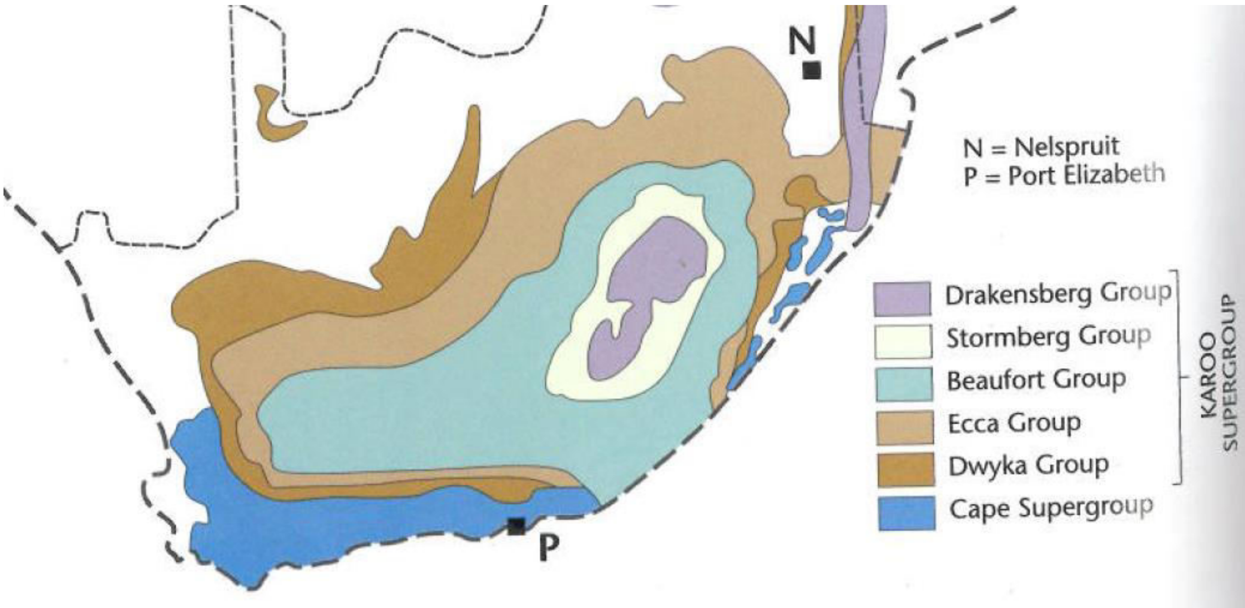


Figure 2.7 Diagram showing the spread of the Cape and Karoo Supergroup across South Africa (McCarthy & Rubidge, 2005)

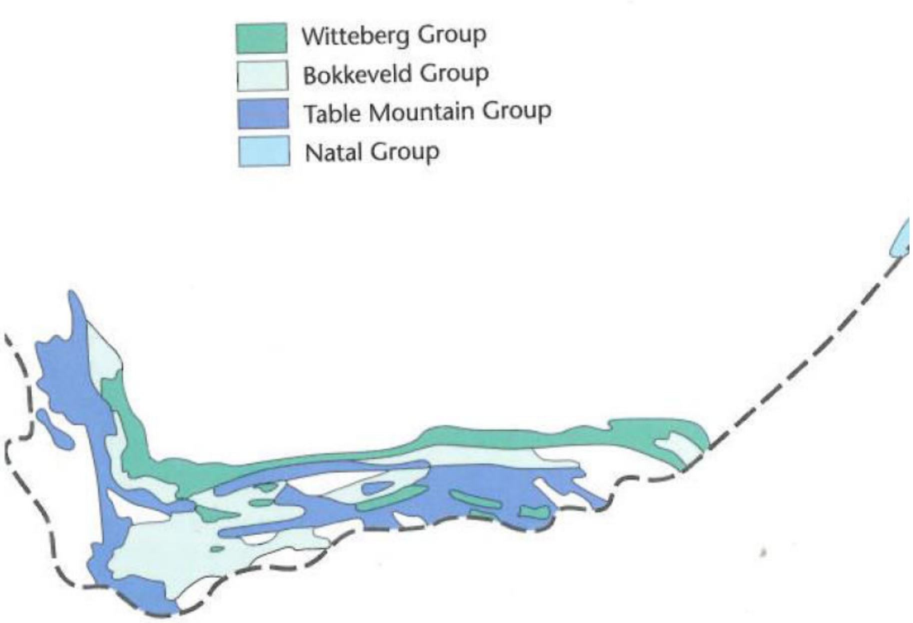


Figure 2.8 Diagram showing the spread of the various Groups within the Cape Supergroup (McCarthy & Rubidge, 2005)

In a thesis written by Fourie (2010), he states that the Table Mountain Group consists mainly of "supermature quartz-arenite". According to the Pettijohn classification of sandstones, "a quartz arenite consists of 95 % quartz, with a matrix of 15% or less", as noted by Nichols (2009). According to Nichols (2009,) "the matrix to sandstone will be silt and/or clay-sized sediment". Brink (1981) notes that the Table Mountain Group is "well jointed, thickly bedded and characteristically cross-bedded quartzitic sandstone with minor mudrock horizons".

During the period of around about 440 and 420 million years ago, the region underwent a glacial period, of which the Pakhuis formation was deposited. The Pakhuis formation consists mainly out of tillite. Tillite, as stated in by Nichols (2009), is deposits by ice that has become lithified. From McCarthy & Rubidge (2005), lithification is termed as involving the cementation of particles .Once the glaciers started to retreat, due to an increase in the earth's surface temperatures, it resulted in the deposition of fine muds in a shallow bay or glacial lake environment. This is known as the Cedarberg formation. Fourie (2010) states that Cedarberg formation consists mainly of "glacial rebound argillaceous (shale)". The term shale, as stated by Nichols (2009), can be applied to any mudrock which shows "fissility", "which is a strong tendency to break in one direction, parallel to the bedding".

Around 400 million years ago, another period of rifting and subsidence resulted in the deepening of the ocean floor. This resulted in the deposition of "deeper-water, fine-grained sediments" of the Bokkeveld formation" as noted by McCarthy and Rubidge (2005). These sediments resulted in mainly mudstones. A clear transition from coarse-grained, supermature quartz arenite moving to that of blue-black mudstone and shale, can be seen at the contact of the Table Mountain Group and the lower most unit of the Bokkeveld Group, was documented by Fourie (2010). According to McCarthy & Rubidge (2005), "these rocks weather more quickly than sandstones of the remainder of the Cape Supergroup and consequently form valleys rather than mountains".

Fourie (2010) states that the Bokkeveld Group has up to "5 upward-coarsening progradational deltaic sedimentary successions that each grade from mudstone and shale into siltstone and is finally capped by feldspathic wacke or immature sandstone". Brink (1981), makes reference to the fact that the sandstone contained in the Bokkeveld group "is softer and less resistant to weathering", than the other sandstones in the Cape Supergroup. In Nichols (2009) upward-coarsening refers the coarsest bed occurring at the top and beds with finer material at the bottom. See figure 2.9 below, for a visual representation of what coursing up may mean .Nichols (2009) states that the general term mudrock refers "any indurated sediment made up of silt and/or clay". Nichols (2009) go's further to defining mudstones to be made out of mixtures of more than one-third each of clay and silt. See Figure 2.10 for a compositional make up of various clay containing rocks.

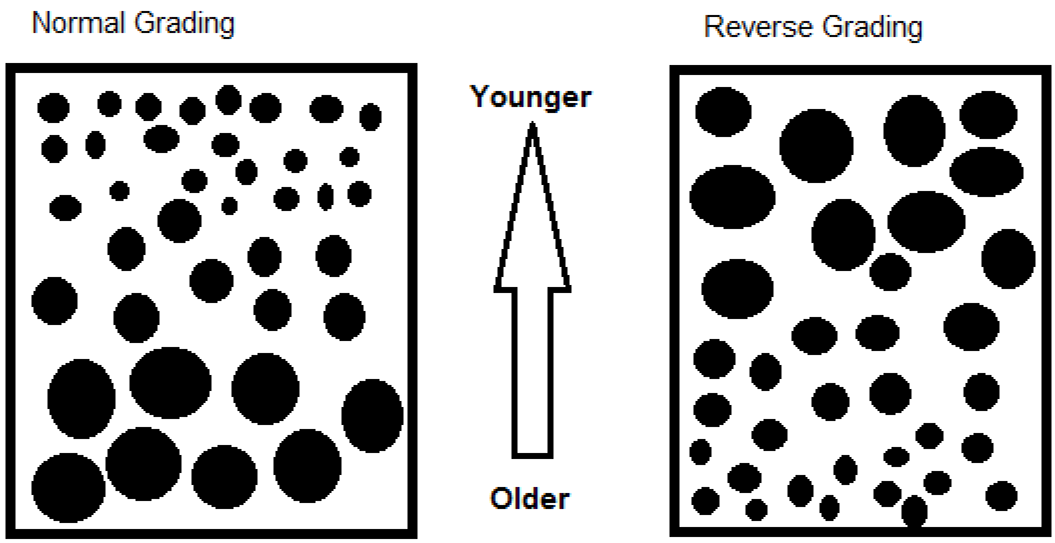


Figure 2.9 Diagram showing grading and layer fining

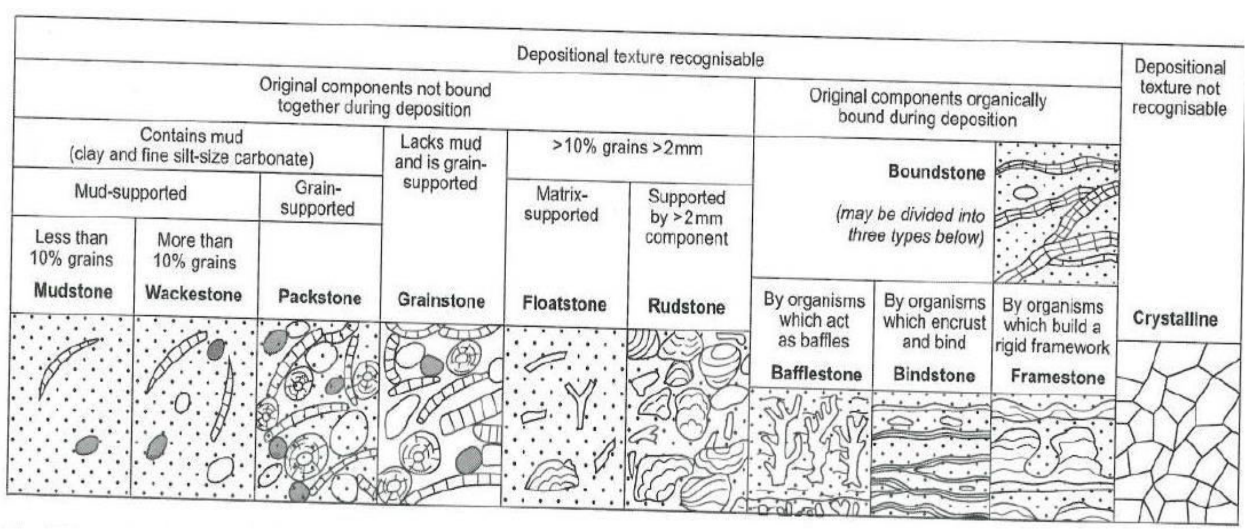


Figure 2.10 Diagram showing compositional makeup of various clay rocks (Nichols, 2009)

Around about 250 million years after the last sediments where deposited in the Cape Supergroup, major tectonic shifts where experienced. Stresses from South and West in the continent, caused a change in the "reversal of direction of sedimentation" as noted by Brink (1981). Due to the stress increase, the various strata layers buckled due to the stresses experienced and were eventually folded and upturned due to faulting. Brink (1981) notes that the Bokkeveld mudrocks in the Cape Fold belt "exhibit intensive slaty cleavage as a result of later folding". As a result of all the folding , two general mountain ranges formed, each with their own strikes. The one mountain range stretches from Villiersdorp all along the west coast, up towards Clanwilliam. This mountain range has a general strike of N-S .The second, the one of which is of importance to us, ranges from Caledon and goes all the way to Port Elizabeth. This mountain range has a general strike of E-W. Brink (1981) notes that "very large strike faults" occur in the region.

Although not indicated as a significant layer in various resources, surface limestone also occurs on the farm's upper most area. "Calcium carbonate ($CaCO_3$) forms the principle compound to Limestone", as stated by Nichols (2009).

For a general summary of the rocks occurring in the Cape Supergroup, Figure 2.13 gives the breakdown of the Group names, formations, thickness of formation and the dominant lithology in the formation. Figures 2.11 and 2.12 just confirm again what parts of the Cape Supergroup , we are dealing with.

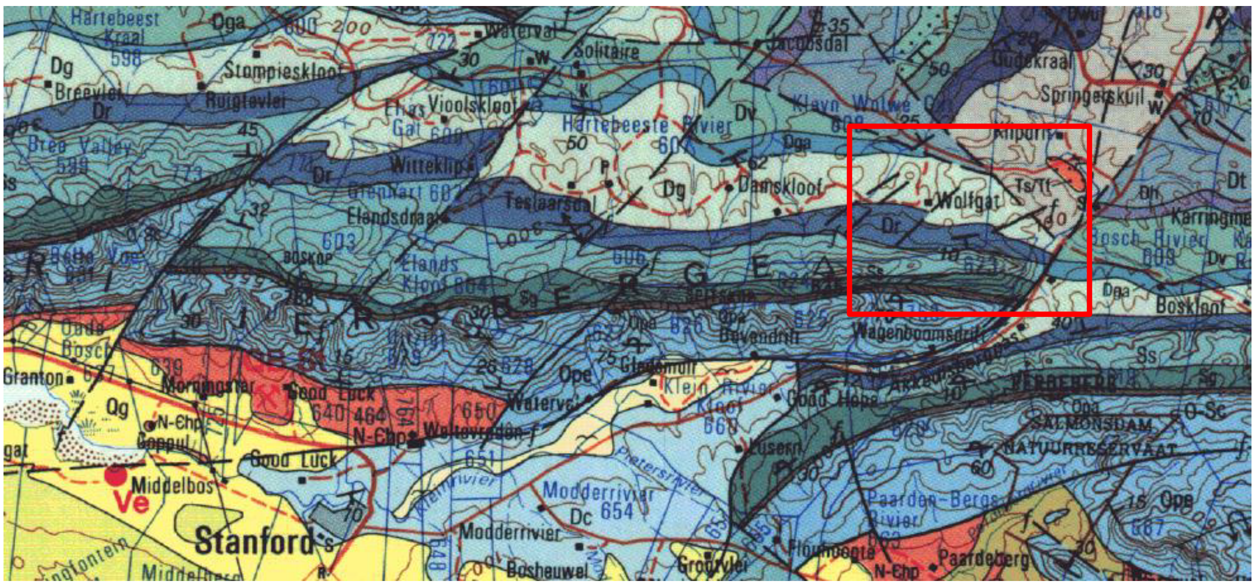


Figure 2.11 Zoomed in area of embankment sites geology (Council for Geoscience)

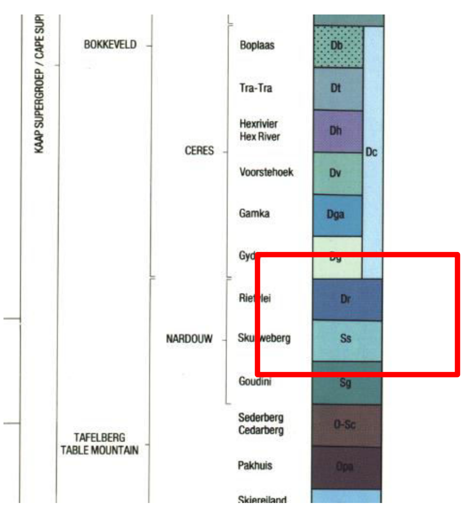


Figure2.12 Key to reading figure 2.11 numbers (Council for Geoscience)

Groups	Sub-groups	Formation	Thickness (m)	Dominant lithology†
Witteberg	Lake Mentz	Waaipoort Shale*	?	Shale (grey).
		Floriskraal	60	Shale, quartizitic sand-stone, reddish shale.
		Kweekvlei Shale	130	Shale.
		Witpoort Sandstone	310	Sandstone.
	Weltevrede	Swartruggens	450	Shale, sandstone.
		Blinkberg Sandstone	80	Sandstone (resistant, white).
		Wagen Drift	70	Shale, sandstone.
Bokkeveld	Bidouw	Karoopoort Shale	50	Shale,
		Osberg Sandstone	55	Sandstone.
		Klipbokkop Shale	170	Shale.
		Wuppertal Sandstone	65	Sandstone.
		Waboomberg	200	Shale.
	Ceres	Boplaas Sandstone	30	Sandstone.
		Tra-Tra Shale	85	Shale.
		Hex River Sandstone	100	Sandstone.
		Voorstehoek Shale	115	Shale.
		Gamka Sandstone	135	Sandstone.
		Gydo Shale	160	Shale.
Table Mountain		Nardouw Sandstone	500	Sandstone.
		Cedarberg Shale	120	Shale.
		Pakhuis	40	Sandstone, conglomerate, diamictite.
		Peninsula Sandstone	1 550	Sandstone (coarse, thick-bedded)
		Graafwater Sandstone	440	Sandstone (thin-bedded), shale.
		Piekenierskloof	800	Conglomerate, sandstone.

* Formerly known as the Lower Dwyka Shales

Figure 2.13 Summary of Cape Supergroup lithology's (Brink A. , Engineering Gelogy of Southern Africa Volume 2 : Rocks of 2000 to 300 million years in age, 1981)

2.2.3 Engineering properties of the geology and Founding conditions

Below, we take a more in depth look at each rock type and specific characteristics associated with each type of rock. The rocks selected, are those selected in the above section of the Geology of dam site.

2.2.3.1 Mudrocks

In this classification, we have the mudstones and shale, as mentioned in the geology of the region.

2.2.3.1.1 Engineering properties of mudrocks

According to Fell, Macgregor, Stapeldon and Bell (2005), "most mudrocks when fresh lie in the weak to extremely weak range" as defined in Table 2.2. The strength in mudrocks lies in the rocks cementation by the minerals calcite and silica. Due to their high concentration composition consisting of high clays, mudrocks have high porosity and water absorption properties. Due to the expansive nature of clays, mudrocks develop small cracks due to periods of wetting and drying and with further cracking and swelling, results in the disintegration of the rock back to clay. The mechanism behind this rapid disintegration is as follows:

- Water is absorbed rapidly into cracks by soil capillary suction
- Air is compressed by water in the cracks
- Adjacent rock swells slightly
- As a result, cracks widen and propagate. Eventually leading to breakage of the clay minerals

Fell, MacGregor, Stapeldon and Bell (2005) notes that under "constant humidity environments such as in rockfills or earthfills, such rocks have been found to have suffered little or no breakdown over periods of up to 90 years".

Table 2.2 Engineering characteristics associated with rock strength (Fell, MacGregor, Stapeldon, & Bell, 2005)

Rock strength class	Symbol	Point load strength index($I_{s((50)}$)	Approximate unconfined compressive strength-Qu(MPa)
Extremely weak	EW	0.04	1
Very weak	VW	0.2	5
Weak	W	1	25
Medium strong	MS	2	50
Strong	S	4	100
Very strong	VS	10	250
Extremely strong	EH		

2.2.3.1.2 Bedding surface faults in mudrocks

Fell, MacGregor, Stapeldon and Bell (2005) noted that in mudrock sequences, which are trapped between "interbedded stiffer rocks (e.g. sandstones or limestones)" undergo folding, tilting or stress relief movements, thin seams of crushed rock maybe develop at these boundaries. The cause of these crushed seams, are due to interbed slips. In mudrock, these planes of slips are termed bedding-surface faults. The planes normally contain clay planar layers, with the layers having slickensided surfaces on the planar edges, as well as within in the structure. See Figure 2.14 for an indication of bedding surface faults. Figure 2.15 shows a few interlayered sandstone and mudstone layers, with bedding surface faults. Fell, Macgregor, Stapeldon and Bell (2005) conducted tests in a laboratory and established that the residual effective shear strength of these planes ranged “from 7 to 12 degrees, with no cohesion”.

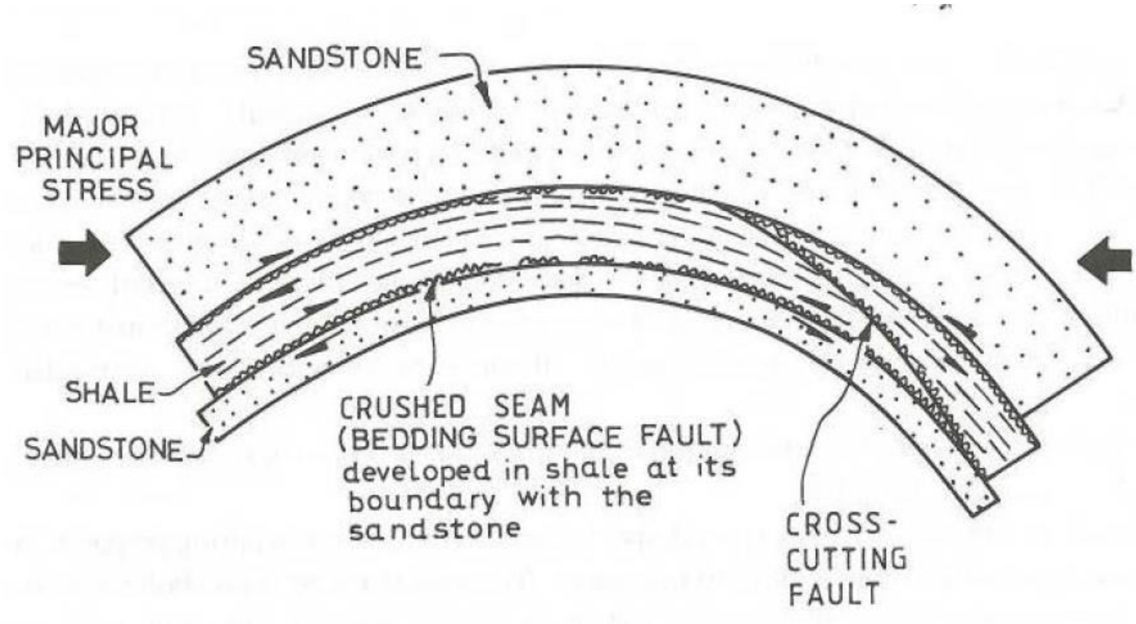


Figure 2.14 Bedding surface fault (Fell, MacGregor, Stapeldon, & Bell, 2005)

2.2.3.1.3 Slickensided joints or fissures

In some mudrocks, irregular sets of intersecting, slickensided joints may occur and are indicated in Figure 2.14. Fell, MacGregor, Stapeldon and Bell (2005) state that these are the result of the following in the list below:

- Synerisis
- Shrink and swell movements
- Different shear movements during consolidation
- Large lateral stresses

The causes are considered to have happened during the formation of the rock, when it was still clay soil. These joint sets however, have much less shear strength, than that of the already low shear strength of intact mudrock. When considering the strength parameters for design usage, Fell, MacGregor, Stapeldon and Bell (2005) make reference that “the strength of both the intact surface substance and the joints, and the spacing, orientations and the continuity of the joints, need to be taken into account”.

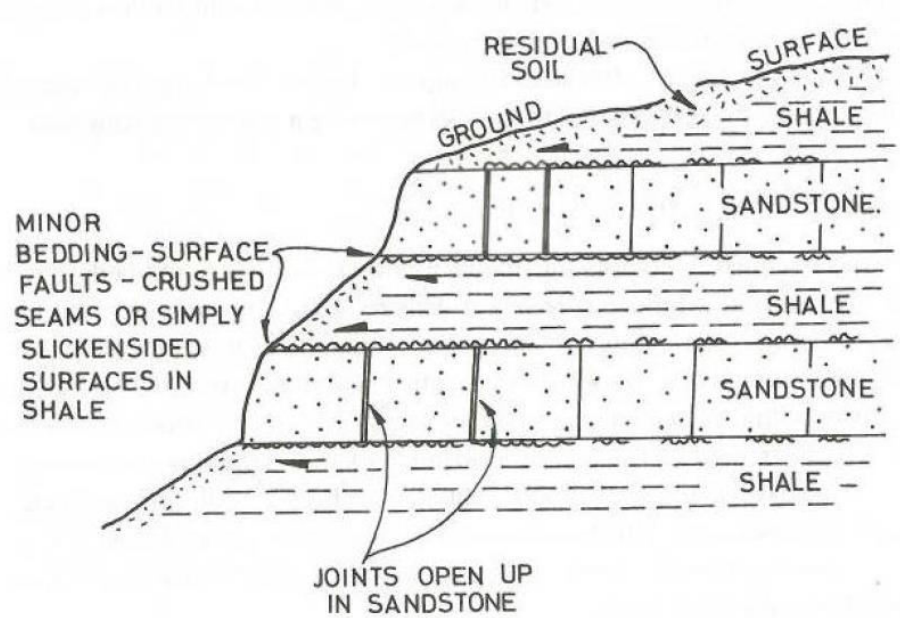


Figure 2.15 Interlayered beds of sandstone and shale, with bedding surface faults and slickensided surfaces (Fell, MacGregor, & Stapledon, Geotechnical engineering of embankment dams, 1992)

2.2.3.1.4 Stability of slopes underlain by mudrocks

Slopes that are underlain by mudrocks are commonly unstable, even at slope angles as small as 10 to 15 degrees. This can clearly be seen with the above mentioned material make-up that results in lower strength in the rock. Fell, MacGregor, Stapeldon and Bell (2005), points out that weathered shale produces a zone layer of low-permeability near the surface and is often underlain by "jointed, less weathered shale, which is more permeable". This situation becomes a problem when groundwater infiltrates the lower zone or along sandstone beds, which lead to excessive pore pressures in the near-surface, more weathered shale zone.

2.2.3.1.5 Suitability of mudrocks for use as construction material

Due to its characteristics of having low strength and slacking properties, mudrocks are often not considered for the usage as a construction material in concrete or filters. However, Fell, MacGregor, Stapeldon and Bell (2005) states that "random fills, earthfills and cores for embankment dams have been built successfully using mudrocks in various conditions". Sowers (1962), points out that "shale foundations are usually water-tight but may be structurally weak because of their tendency to slide along bedding planes". Shale that has been fully weathered can be considered as a clay source for fills, provided that the plasticity isn't to high. Partially weathered shale is not considered as fill, as Sowers (1962) points out the shale "tends to weather at an accelarated rate after being incorpaorated in an embankment dam".

2.2.3.1.6 Checklist in mudrocks

From the above sections on mudrocks, the following list summarizes the main issues when considering mudrocks:

- Slaking or disintegration on exposure environmental factors
- Swelling exposure of the mudrocks
- Soluble minerals in veins or beds
- Slickensided fissures within the beds
- Bedding surface faults or shears within the beds
- Unstable slopes (shallow, in weathered materials)
- Unstable slopes (deep-seated, if bedding in folded rocks daylights
- Possibility of high pore pressures, in layered sequences
- Suitability for rockfill, random fill, earthfill and haul roads

2.2.3.2 Sandstone and other related sedimentary rocks

As defined in the previous section on the sites geology, the make-up of sandstone and other related sedimentary rocks often consist of the same mineral make-up, but differ in the amount of cement/matrix. For the engineering properties, the make-up of these rocks will be that in Table 2.3.

Table 2.3 Sedimentary rocks and make-up (Fell, MacGregor, Stapeldon, & Bell, 2005)

Rock name	**Particle shapes, grading**	**Minerals**	
		Most grains	**Common matix/cements**
Sandstone	Usually rounded, one-size grains and less than 15% matrix or cement	Quartz, fragments of older rock	Silica, clay, iron oxides, calcite, gypsum
Arkose	Sub-angular, often well graded, little matrix	Quartz, plus at least 25% feldspar; some mica	Clay, iron oxides, silica
Greywacke	Angular, well graded down to clay matrix which is usually >15% of volume	Feldspar, quartz , hornblende, micas, rock fragments, iron oxides	Clay and same as grains

2.2.3.2.1 Properties of rock substances

The types of grains and cement/matrix combined with that of the depositional environment of the sandstones all have an effect on the sandstones strength and durability. Quartz sandstones are by Fell, MacGregor, Stapeldon and Bell (2005) "sandstones often have significant porosity (5 to 20%) and might be slightly permeable". Due to angularity and grading of particles, greywackes are often stronger than sandstones. Fell, MacGregor, Stapeldon and Bell (2005) state that “Silica cement usually occurs in strong, durable rocks and at the other extreme, rocks cemented by clay or gypsum are usually weak and non-durable”. Special tests need to be conducted on these cements, as they could contain gypsum or anhydrite, as these substances can cause faults and easy erosion of the dam foundation.

2.2.3.2.2 Suitability for use as construction materials

Rocks which form part of the strong to extremely strong group of the sandstone group of rocks are used as rockfill or rip-rap in dams. These rocks are also used in concrete, as aggregates, due to their strength which is associated to the hardness of quartz. See Figure 2.16, which shows that the hardness of quartz lies at 7. Although as a negative aspect, this leads to high costs associated with quarrying and the handling of the material, which is associated with its high abrasiveness. Some of the weaker rocks in the sandstone group of rocks, tend to be “more porous and also lose significant strength on saturation” according to Fell, MacGregor, Stapeldon and Bell (2005).

Figure 2.16 Mohs scale of hardness (Geology IN:The Mohs scale of mineral Hardness)

2.2.3.2.3 Weathering products

Chemical weathering has a major effect on sandstones, due to the interaction with the matrix or cement. Interactions of these chemicals with the cement, leads to the breakdown of chemical bonds in the rock structure, which results in the removal of cement/matrix. This weakens the whole structure of the rock. Fell, MacGregor, Stapeldon and Bell (2005) states that “rocks with silica or iron oxide cements are the most resistant”. This is as a result of the hardness of the minerals, as given in Figure 2.16 above. The interaction of the chemicals leads to the types of rocks in the sandstone group, to be broken down into their make-up. Quartzites will break into clean quartz sands, arkose and greywackes into silty or clayey sands and sandstones and conglomerates leads to sands or gravels.

2.2.3.2.4 Weathered profiles and stability of slopes

Gradational boundaries can be seen in the weathering profiles of the sandstone group rocks, in strengths of both weak porous rocks and that of the “stronger, more durable rocks when they are closely jointed" as noted by Fell, MacGregor, Stapeldon and Bell (2005).Reference by Fell, MacGregor, Stapeldon, Bell (2005) that "where large contrasts occur between the resistance to weathering of interbedded rocks, sharp but irregular sawtooth shaped boundaries may occur".

As mentioned in the above section on mudrocks, crushed seams (bedding surface faults) may occur in interbedded layers of shale and sandstone, along bedding boundaries. If these interbedded layers where near horizontal and had formed cliffs, then steeply dipping joints would form in the sandstone, perpendicular to the bedding surface faults in the shale layer ,as in Figure 2.17. If these joints only occur close to the edge of valley sides, the formation as shown by Fell, MacGregor, Stapeldon and Bell (2005) is due to "interbedded stress relief of the shale moving further out from the slope than the sandstone". Pressures from water in the joints and earthquake forces, may also effect the movement of the sandstone blocks. The opening of joints, leads to higher permeability in these exposed sandstone layers, near the surface.

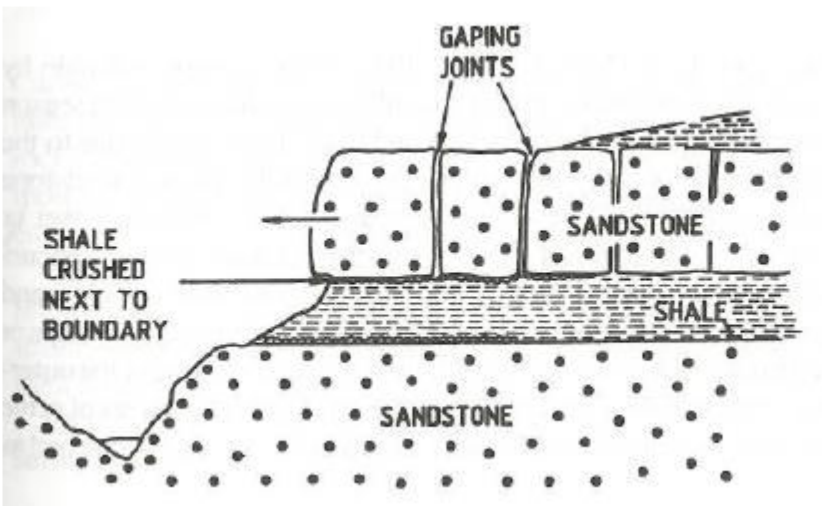


Figure 2.17 Crushed shale layer and bedding joints (Fell, MacGregor, Stapeldon, & Bell, 2005)

When hillsides consist mainly out of rocks from the sandstone group, steep slopes or cliffs are formed, like in Figure 2.18. Fell, MacGregor, Stapeldon and Bell (2005) describes that "slope failures are rare and usually by rockfalls or toppling from cliff portions". When these hillsides consist of interbedded layers of shale and sandstone on the other hand, weathering extends deeper due to the shale component and can lead to landslides being more common in the area. In cases where the hillside mentioned above, is in an area with a high water table or situated in a high rainfall area, chemical weathering of both rock groups may be experienced. Due to the "relatively free draining nature of sandstone beds", as noted by Fell, MacGregor, Stapeldon and Bell (2005), springs may occur at the bases of the sandstone beds, which also leaves the underlying shale either saturated or between phases of wet and dry. When this happens, both sandstone and shale undergo chemical weathering, but can be seen more clearly in the shale beds. Some of the shale has weathered done to clay and "contains clay-coated joints or fissures, often slickensided", which was noted by Fell, MacGregor, Stapeldon and Bell (2005). When the bearing capacity of the shale is exceeded or slumping in the layer occurs, large movements and collapse of the outer sandstone blocks may occur.

If the above mentioned process is continued, scree and colluvium may develop on the slope. Fell, MacGregor,Stapeldon and Bell (2005) make note that landsliding has been observed , where sandstone has been covered by scree and colluvium. When this happens, drainage from the sandstone layers are restricited(refer to layer on diagram), resulting in a confined aquifer forming with its own piezometric pressure level. Sliding occurs along the collovium-weathered shale contact, due to pore pressure build up. As in indicated in figure 2.19, sliding can occur further inside the slope, due to various geological reasons. Fell, MacGregor, Stapeldon and Bell (2005) lists "bedding surface crushed seems along the shale-sandstone boundaries", high water pressures in sandstones 2 and 3 and high water levels in the affected mass". Small movements may then occur and as a result of all these above mentioned geological processes, will lead to the permeability increasing deeperin the slope. This will result in further weathering deeper in the slope. When folded interbeds of sandstone and shale/siltstone occur on steep slopes, landslides are common, where the dipping beds daylight on the slope.

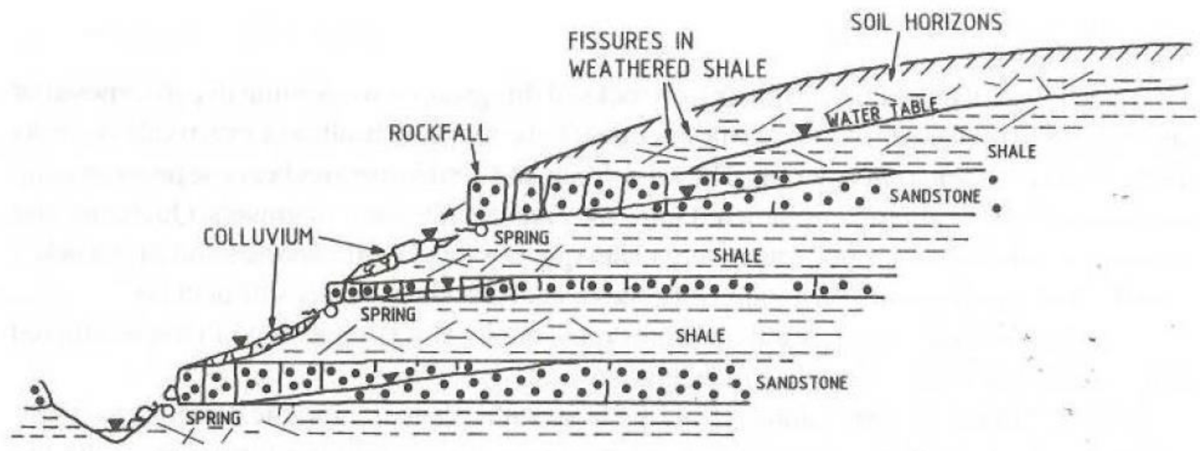


Figure 2.18 Steep slopes of interlayered sandstone and shales (Fell, MacGregor, Stapeldon, & Bell, 2005)

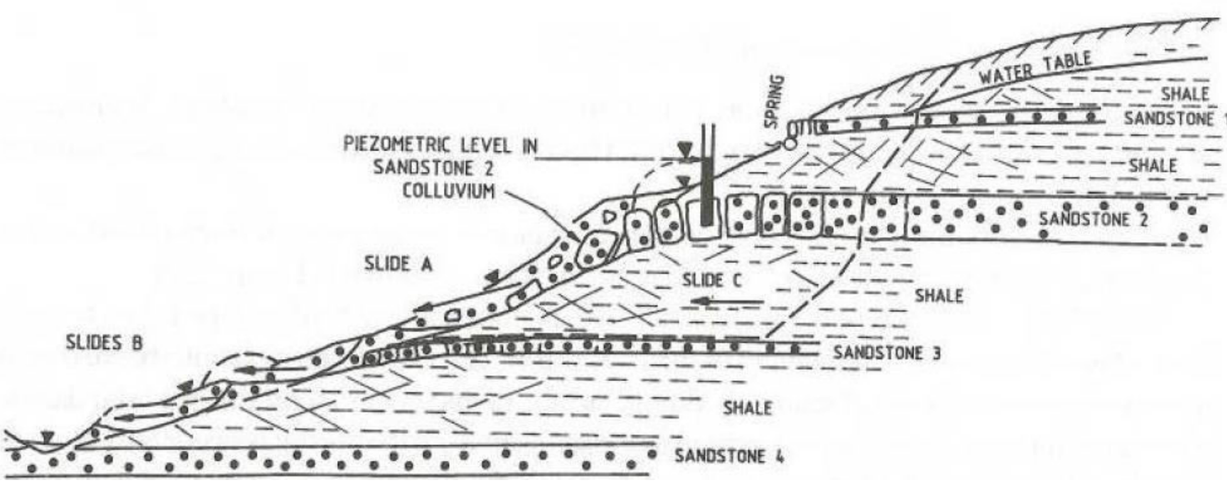


Figure 2.19 Occurrence of sliding along the same slope in figure 2.17 (Fell, MacGregor, Stapeldon, & Bell, 2005)

Past landslides can cause trouble if no knowledge of site information is discovered and one such dam payed the price. The St. Francis dam in America, was completed in 1926 , but the dam didn't function for a long period of time after construction. Shorty after completion, cracks started to occur in the dams foundation and as eartly as 1928 the dam failed. The cause of the failure, was investigated to caused by the embankment being buit on top one of these weak layers , caused by a past landslides.

2.2.3.2.5 Sandstone checklist

From the above sections on sandstones, the following list summarizes the main issues when considering sandstones:

- Gypsum or anhydrite present as cement
- Rocks of medium or lower strength may not produce free-draining rockfill
- Interbeds of shale or claystone
- Bedding-surface faults at bed boundaries
- Horizontal beds: Open joints and bedding surface crushed seams near surface of horizontal beds
- Horizontal beds with shale interbeds: Collapse due to removal of support by weathering shale
- Landslidding in colluvium developed on weathering sandstone/shale slopes

2.2.3.3 Carbonate rocks

Fell, MacGregor, Stapeldon and Bell (2005) define these rocks as being rocks "which contain significant amounts of soluble minerals calcite, aragonite or dolomite in their substance fabrics". Figure 2.20 shows the composition of the above mentioned rocks. Carbonate rocks can be divided into two groups, dependent on their age of formation:

Geologically young carbonate rocks (Category Y) (Fell, MacGregor, Stapeldon, & Bell, 2005)

- Age : Tertiary to younger
- Composition/Structure: Loosely packed, weakly cemented shell fragments
- Engineering properties : Porous and weak to very weak in strength
- Carbonate minerals present: aragonite and high magnesian calcite
- Mineral reactions: high magnesian calcite is more susceptible to dissolution and cementation, than aragonite and calcite.
- Depositional Environment: marine setting
- Reactions: Exposure to fresh water, compaction and recrystallisation both aragonite and high-magnesian calcite eventually revert to calcite and the rock becomes *Category O*
- Exception: Calcrete- highly variable strength rock, occurs in arid regions

Geologically old carbonate rocks (Category O) (Fell, MacGregor, Stapeldon, & Bell, 2005)

- Age : Mesozoic and older
- Composition/Structure: Dense
- Engineering properties : non-porous, range from strong to extremely strong
- Carbonate minerals present: calcite or dolomite
- Exception: Marble is included here- noted to be dense, non-porous and strong to very strong. Calc-Silicate rocks- combination of both carbonates and silicates like olivine, diopside and garnet

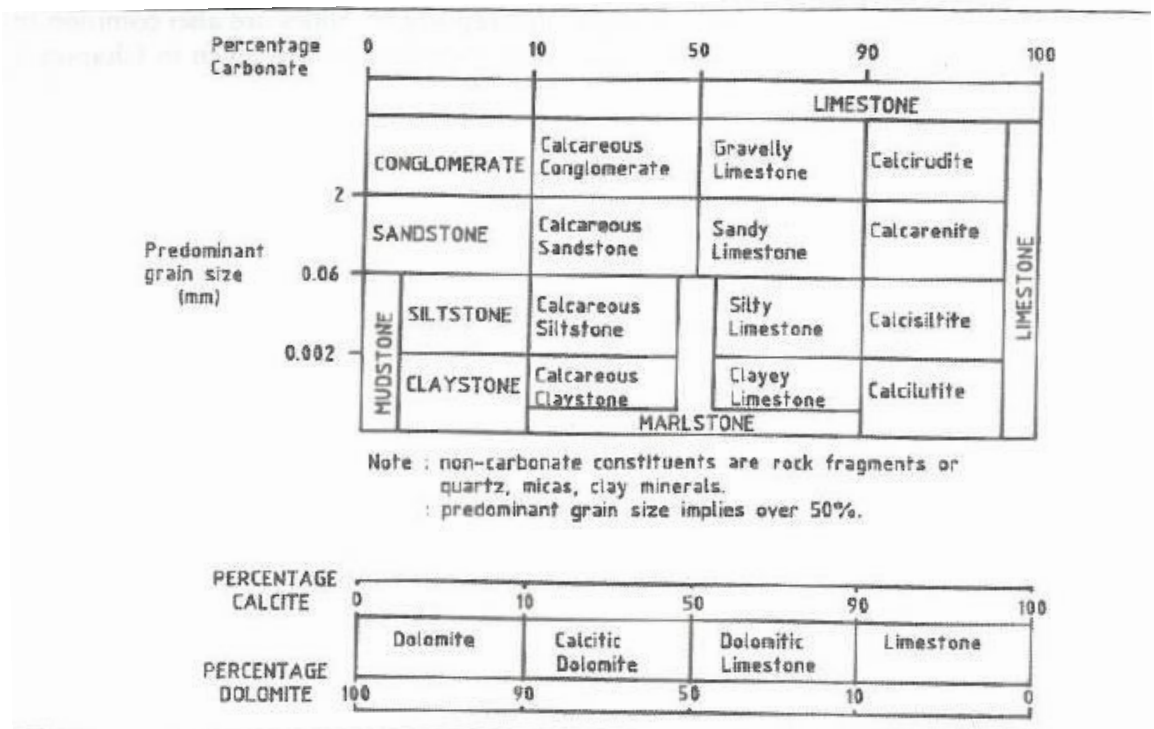


Figure 2.20 Composition of Carbonate rocks (Fell, MacGregor, Stapeldon, & Bell, 2005)

2.2.3.3.1 Effects of solution

Chemical weathering, in the form of solution, affects most rocks in some way or another. Due to their reactivity in more acidic solutions, cavities are formed in carbonate rocks, due to calcite and dolomite being highly soluble. Saline solutions allow for more solubility than fresh water . The effects of solution have a varying effect on the type of carbonate rocks, dependent on their percentage carbonate mineral build-up. Fell, MacGregor, Stapeldon and Bell (2005) list the following three groups and explains the solutions effects on them.

1. Rock masses composed of dense, fine grained rock substances comprising more than 90% of carbonate (usually Category O)

These rocks, when fresh and intact, have very low porosities and “their substance permeabilities are effectively zero” as noted by Fell, MacGregor, Stapeldon and Bell (2005). Flow of groundwater, in these types of carbonate rocks, is restricted along joints and other cracks. These spaces are often increased due to solution, causing the formation of cavities and shafts within the rock. These formations are often referred to as karst landscape. Karst landscapes can be defined as landscape that " is largely shaped by their dissolving action of water on carbonate bedrock (usually limestone, dolomite, or marble)" (Karst- What is a karst made of (2016)).

Fell, MacGregor, Stapeldon, & Bell (2005) make note on certain important features that may occur within this type of carbonate system. On some upper rock surfaces, both in outcrops and below the surface, cavtities and deep slots might separate pinnacle outcrops. These slots and cavities are either empty or might be soil filled. Refer to Figure 2.21 of the type of landscape. When no outcrops are present , dissolution may have resulted in this relatively pure carbonate being fully eroded away, leaving about 10% insolubles left. The insolubles form residual soils and fill in some cavities. Clay and iron-oxides form a major part of these soils and can be characterised as being fissured in nature. Sinkholes, which refers to shaft-like cavities, occur on surface and is often clearly exposed or often covered by residual soils. Natural or man-made activities may lead to the formation of new sinkholes, on this type of landscape.

Figure 2.21 Karst landscape (Hassellf)

2. Rock masses composed of dense, fine grained rock substances containing 10% to 90% of carbonate (usually Category O)

The types of rocks in this classification, also experience the same formation of cavities like the 90% carbonates, but exposed rock surfaces in cavities have a weathered nature to them. Rocks, with fresh carbonate percentages close to the lower boundary of the 10% carbonate composition mark, have higher exposed weathered parts compared to cavities. Due to this, "the higher the ratio of infilled cavities to open cavities" there is in the landscape, as documented by Fell, MacGregor, Stapeldon and Bell(2005). This weathered fraction of the rock is weaker than surrounding rock and as result of the solution interaction, less dense too. Fell, MacGregor, Stapeldon and Bell (2005) also take note to mention, that the proportion and propertities of the weathered rocks depend on percentages of insoluble minerals in the fresh rock. The type of non-soluble cements, which held the minerals together, was also key to the fraction of unweathered rock.

3. *Rock masses composed of porous, low density carbonate rock substance (usually Category Y)*

The types of rocks in this classification are considered weak calcarenite, with the carbonate recognizable as shell fragments. The rock beds occur mainly without joints or other characteristic tectonic defects. Gentle dipping or horizontal beds often characterize the orientation of these carbonate beds. The beds have high permeability and when water enters the beds and move downwards, "solution and redepostion effects" are visible, as noted by Fell, MacGregor, Stapeldon and Bell (2005). The above-mentioned source also makes reference that when this solution of calcarenite moves downwards in the beds, "redepostion of calcite derived from that solution" occurs and results in the strengthening of the bottom calcarenite. This strengthening of the bottom beds, results in the formation of tubular/ vertical pipe systems in the soil. Fell, MacGregor, Stapeldon and Bell (2005) note that very little cavitities form in the low density, porous carbonates compared to that of the jointed, dense carbonate. The authors do however point out that sinkholes are present in this carbonate rock groups, which form along "localized calcarenite zones rendered more permeable by small movements along faults in the bedrock".

2.2.3.3.2 Watertightness of dam foundation

Fell, MacGregor, Stapeldon & Bell (2005) take note to point out that sites that have been underlain by Catergory O carbonate rocks, have had succesful dam construction , even though solution cavities where present on site. Due to the irregular nature of the cavity formation in the rocks, this rock substrate provides a difficult construction surface for embankment dam foundations, as the foundation surface needs to "provide stable,non-erodible surfaces for placement of embankment dam materials", as stated by Fell, MacGregor, Stapeldon and Bell (2005). Possible solution have been achieved by either cement grouting or selective mining and then backfilling of the cavities. Sometimes the cavities are larger and bigger, at wich each situation has to be investigated on its own merits. Cement grouting is often then not considered by itself to form the cut-off and another action is considered. Backfilling of individual cavities and digging wall slots in cavernous rock and then fill with concrete to form cement walls, are two options which are considered. In their book, Geotechnical Engineering of Dams, the authors Fell, MacGregor, Stapeldon and Bell (2005) also make the solutions of "Diamprahm walls comprising overlapping boreholes backfiled with concrete" and "closely spaced drilled holes, washed out with compressed air and water backfilled with high-slump mortar,poured in and needle vibrated" as another two viable ones.

2.2.3.3.3 Potential of sinkhole formation below dam

The formation of sinkholes is commomly associated with areas with carbonate rocks beneath it.In areas of rocks, which have experienced collapse in past, it is not un-common for any future new sinkholes forming. However, the frequency and size in sinkholes is often increased, if there is any man-made activity on the area. Fell, MacGregor, Stapeldon and Bell (2005) list the following mechanisms as possible sinkhole formations:

- Dewatering: This process can cause sinkholes in many ways. Rocks may lose buoyant support from surrouding rock, due to loss of water. The drying out of soil, can lead to the shrinkage of the soil and can result in failure. The loss of water, may also increase the hydraulic gradient in the soil, which may lead to erosion and collopse of soil into the cavity.
- Inundation: Inundation can be defined as "to flood an area with water" (Cambridge Dictionary).This process can cause sinkholes in two ways. The first being, that dry soil may "lose apparent cohesion" and result in collopse in the soil. The second way being, that in wet soils, a gradient increase may result in erosion into a cavity
- Vibrations: The usage of machinery on top of the soil or possible blasting in surrounding areas, may result in creating a sinkhole in an already unstable rockmass.

2.2.3.3.4 Potential for continiung dissolution of jointed carbonate rock in dam foundation

Dissolution is the process where by carbonate rocks slowly dissloves. The dissolving occurs as a reuslt of percipation containing dissolved Carbon dioxide(CO_2) and this solution forms a mild acidic mixture. This acidic mixture , when in contact with carbonite rocks, slowly dissolves away the carbonate rocks resulting in open cavitites and cracks. This can lead to water entering and can cause possible fam failure, due to tunnel errosion.

2.2.3.3.5 Potential problems with filters composed of carbonate rocks

"Most filters are well graded sands or gravels with few or no fines" as stated by Fell, MacGregor, Stapeldon and Bell (2005). These are often compacted to density ratios between 60% to 70%. These filters often experience various moisture environments, specifically in chimney zones. The filters may experience periods of either unsaturated, basically dry conditions, to periods of inundation to large/small flow rate through the filter. These periods of moisture change, plus the development and movement of carbon dioxide into the filter, may have an effect on carbonates, if used in a filter. The prescence of carbon dioxide can either be explained by CO_2 dissolved in rainwater,from the rotting of organic matter or from the oxidation of sulphite minerals. If carbon dioxide had to enter a carbonate filter, Fell, MacGregor, Stapeldon and Bell (2005) describe that it may have the following effects:

i. Change in grading, due to dissolutoin
ii. Partial dissolution and recementation
iii. Interlocking of grains due to pressue solution

Category O carbonates:

As predefined for this category, these carbonates are very dense and strong to very strong. When crushed, this type of rock forms angular, but fine material that can be used in a filter. The authors have found no report of a dam incident or failure due to malfunction of these filters. On the other hand, the authors Fell, MacGregor, Stapeldon & Bell (2005) do make note that "not all dams have instruments sufficient to monitor the performance of all of their filters". Due to this statement, it is key to discuss possibly how the above mentioned filter affects problems, may occur for category O carbonates.

i) Change in grading due to dissolution: Critical filter zones are often much smaller than 2.8 m and using that information in conjunction with table 2.3 we can discuss a scenario. Using the scenario discussed by Fell, MacGregor, Stapeldon and Bell (2005), "if pure water was to flow at a rate of 331024m/s through such a zone formed by 50mm diameter calcite particles", then the water present would not become fully saturated in carbon. This would lead to particle size reduction by the process of dissolution. If this size reduction is fueled by acidic water, it may further lead to a reduction in particle and ultimately create problems in the future of the dam's life.

ii) Partial dissolution and recementation: Acid formation during the process of sulphide oxidation may lead to finely crushed carbonate recementation, due to particle breakdown and later cementation.

iii) Interlocking of grains due to pressure solution: For this mechanism of filter problem change, another scenario is by Fell, MacGregor, Stapeldon and Bell (2005) give the following scenario; given than magnesium sulphate and gypsum are highly soluble and if a "chimney zone filter has become cemented by either of these during a period of very low seepage", how does this filter behave when water enters? The authors later concluded after a few tests, that after the carbonates became "saturated and soft to the touch". This was thought to be due to the interlocking particles adding strength, which was created by the pressure-solution. Figure 2.22 shows the process of how this happens.

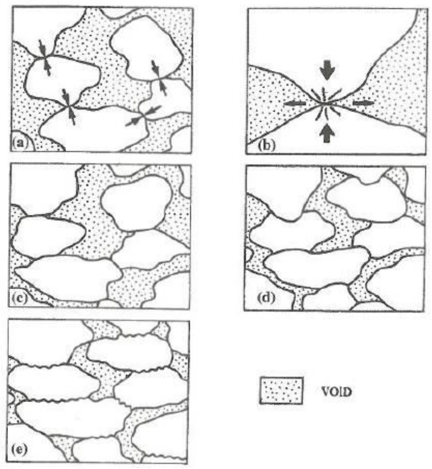


Figure 2.22 Pressure solution shaping grain interlocking (Fell, MacGregor, Stapeldon, & Bell, 2005)

Category Y carbonates:

In the above definition of category Y carbonates, they have been defined to have high fractions of magnesian calcite and are much weaker than the category O carbonates. As also discussed for the category O carbonates, the effects of change in grading due solution, partial dissolution and recementation and interlocking of grains due to pressure solution, will be looked at for the usage of category Y carbonate materials in filters.

i. Change in grading due to dissolution: Magnesian calcite is more soluble than calcite and aragonite, the key component of category O carbonates. Knowing this fact and from past tests done by Fell, MacGregor , Stapeldon & Bell (2005) , they have concluded that if category Y carbonates had to be used as filter material , “more rapid dissolution” will occur than if similar category O carbonates had to be used.
ii. Partial dissolution and recementation: CBR tests that have been conducted on certain road sections, which very made up of calcretes , have shown results in doubling values after “a few cycles of wetting and drying” (Fell, MacGregor, Stapeldon, & Bell, 2005)
iii. Interlocking of grains due to pressure solution: As discussed in Geotechnical Engineering of dams, the authors Fell, MacGregor, Stapeldon, & Bell (2005) decribe tests that where done by the inclusion of coral sands in road mixes, which saw “significent strength gains”. These gains however lost some of its apparent strength after soaking, probably due to the dissolution of gypsum out of the rock structure. A small bit of remaining strength was noted after soaking and was considered to be caused by the interlocking of grains due to pressure solution.

Fell, MacGregor, Stapeldon and Bell (2005) have drawn some conclusion on category Y carbonate materials:

1) Conclusion 1: Some category Y carbonates had a strength increase after a conjunction of compaction and a period of wetting followed by drying occurred. This was caused by solution and recementation. The materials where fine in nature and the apparent strength was noted to be lost after a period of re-soaking.
2) Conclusion 2: Exposed carbonate surfaces, with properties of being very weak and porous, where noted to strengthen due solution and redepostion of carbonate minerals.
3) Conclusion 3: Gypsum was considered to be the mineral, behind the strengthening.

To conclude their final findings on carbonate usage in filters, it was noted by Fell, MacGregor, Stapeldon and Bell (2005) that due to the three methods of change in carbonates, it could all lead to these filters becoming “cohesive” and thus “ineffective”. The authors also note that they won’t advise the usage of carbonate as filters, unless the long term effects of dissolution on carbonates have been concluded.

2.2.3.3.6 Suitability of carbonate materials as embankment materials

Leading off from the usage of carbonates in filters, we can discuss the usage of carbonates in embankment materials. Category O carbonates has been used in rip-rap, rockfill and random fill. Considering the effects of dissolution, discussed in filters, we must consider the effects of suphuric acid on these carbonates. The way suphuric acid may become detrimental to the carbonates, would be if there are sulphide containing minerals in or around the dam site. The sulphide can either be contained in the carbonate rock structure, within other embankment materials or within the foundation or area of the dam. If the sulphide had to undergo oxidation and form sulphuric acid, the sulphuric acid could attack the carbonate rocks and be later deposited in filter zones via solution. The authors Fell, Macgregor, Stapeldon & Bell (2005) makes a key statement in that “carbonate rocks commonly occur in association with mudrocks containing sulphide mineral”. This is a very key statement, as previously mentioned in our geology of the dam site, mudrocks also occur on site. Due to unknown nature of rock types and varying compositions, it must be notably considered not to use carbonates as embankment materials, as one doesn’t always know what minerals all the rocks are made of and a general assumption is normally concluded.

2.2.3.3.7 Stability of slopes underlain by carbonates

Landslides of underlain carbonate area are very uncommon, which can be linked to that redepostion of calcite into joints and other skarn features have stabilized the possible landslide cause. It is also considered, that due to “inherently high frictional strength of joints” (Fell, MacGregor, Stapeldon, & Bell, 2005), that less landslides occur.

2.2.3.3.8 Dewatering of excavations in carbonate rocks

Due to the irregular nature and occurrence of cavities in carbonate rocks, a lot of pumping will have to occur on an excavation site. This coupled with the fact that carbonates are soluble, may lead to more water over time at the excavation site. Thus key bore-hole placing is considered, to limit water in the excavation site.

2.2.3.3.9 Carbonate checklist

From the above sections on carbonates, the following list summarizes the main issues when considering carbonates:

- Category O or Y
- Cavities, air-filled ,water-filled or soil-filled
- Sharp boundary between residual soils and fresh rock
- Strong rock around solution tubes and cavities in weak, porous rocks
- Extremely high permeabilities
- Possible deep, major leakage paths out of reservoir
- Presence of sinkholes
- Composition and pH of groundwater and reservoir water
- Potential for dangerous ongoing solution in the dam foundation
- Suitability for use for embankment materials
- Unstable slopes, where interbeds of mudrocks are present

2.2.3.4 Colluvial Soils

Due to the fact that a portion of the farm is situated along a steep mountain slope, the possibility for us to find colluvial soils is quite high. Fell, MacGregor, Stapeldon & Bell (2005) defines colluvial as "soils which have been eroded and deposited under gravity forces, often with the aid of water". The soils range from containing large boulders to that of finer, high plasticity, clayey soil. The key to this type of soil and the similarity between colluvium, is that the soils consist of a range of a mixture of different particles.

2.2.3.4.1 Scree and talus

These types of soils form at the bottom of steep slopes. When rock fragments become loose from cliff faces, they roll down the slope due to gravity. The larger the rock fragments, the more momentum the rocks have and the further they are able to transport themselves down the slope. This type of colluvium is often poorly graded, highly permeable and compressible, due to the varying types of rocks and soil in the make-up. Fell, MacGregor, Stapeldon and Bell (2005) take note that this type of soil often occurs at the natural slope angle and thus if any excavation is done on the slope, "raveling failures extending upwards" will occur.

2.2.3.4.2 Colluvium Checklist

From the above sections on carbonates, the following list summarizes the main issues when considering carbonates:

- High permeability and compressibility
- Timber debris, rotted or preserved
- Potential for instability or debris-flow

2.2.4 Founding conditions

A dam's foundation plays a major part in deciding which type of embankment dam to choose. Parameters like foundation strength, compressibility and permeability are all factors which need to be considered. If founding conditions are on soil with a low strength, flat embankment slopes are considered. Fell, MacGregor, Stapeldon and Bell (2005) state that this is "likely to favour the construction of earthfill dams, i.e. earthfill with horizontal and vertical drains, rather than earth and rockfill". In soils, which are considered permeable, foundations are often exposed to leakage and erosion and requires a cutoff and filter drain to be constructed under the downstream slope of the embankment.

The presence of a low permeable, but strong rock foundation, would provide the possibility for various dam type constructions, but would favour gravity, arch and concrete dams as the foundation would be able to transpose the forces generated by the weight of the dam's material. In earthquake prone areas saturated sands with a loose to dense makeup, could cause a problem, as liquefaction can take place. According to Rafferty (2016), liquefaction can be defined as "loss of strength that causes otherwise solid soil to behave temporarily as a viscous liquid". Ways of solving this problem would be either to compact the sands or might result in the removal of this problem soil from site. Foundations built on limestone karsts, would need to be grouted at a stage, to control the leakage in the foundation. The Teton dam failure of June 1976, as noted by Sharma and Kumar (2013), was investigated to have failed due to deficiencies in the grouting and sealing of the rock foundation. As a result, internal erosion took place and caused the dam to fail. Fell, MacGregor, Stapeldon and Bell (2005) consider a design plan, "which allows for grouting to continue during construction or after it is completed". Concrete face rockfill dams or earth and rockfill dams, with a sloping upstream core, are dams often considered.

In alluvium soils, foundations may have to battle with the various rates of settling in the alluvium and thus may develop cracking and experience differential movements. Filters play an important part in these embankments, as they would control erosion and seepage experienced internally. The combination of deeply weathered rocks and lateritic soil profiles, may give rise to foundations founded on highly permeable soils. Fell, MacGregor, Stapeldon and Bell (2005) state that these conditions favor embankments "with flatter slopes and good under drainage, e.g. earthfill with vertical and horizontal drain". Folding and faulting of interbedded weak mudstone and claystone and strong sandstones, may result in the formation of low effective friction angles. The above mentioned authors, also go on to mention that "flat slopes may be required on the embankment, favouring earthfill with vertical and horizontal drains, or earth and rockfill with random rockfill zones".

Sowers (1962) states "when the foundation is so weak that it affects the safety factor materially, corrective measures are necessary". This can be corrected in various ways. Removal of weak material, widening of base of the dam, having foundation drainage, preconsolidation of clay soils and densification of cohesionless soils are a few examples of corrective measures.

2.2.5 Spillway size and location

As per the definition of spillways, they are "structures constructed to provide safe release of flood waters from a dam" (aboutcivil.org). The spillways role is to provide a safe route that water can be diverted to, once the full supply limit of the dam is reached. The crest level or top of the dam is always higher than the full reservoir line of the dam. Once the water exceeds the full reservoir line, the excess water then enters the designed spillway for a controlled release. This provides the dam to maintain its shape, as water is prevented from overtopping the crest and creating structural problems on the downslope. Chadwick, Morfett, and Borthwick (2013) point out that the spillways should be designed "to accommodate the "largest" flood discharge (the probable maximum flood or 1 in 10 000 flood) likely to occur in the life of the dam".

As with the fact that there are various types of embankment dams, so are there various types of spillways. The following are factors, that the website civil.org (aboutcivil.org) point out, which should go in the designing of spillways:

- The inflow design flood hydro-graph
- The type of spillway to be provided and its capacity
- The hydraulic and structural design of various components and
- The energy dissipation downstream of the spillway.

The website also points out that "topography, hydrology, hydraulics, geology and economic considerations all have a bearing on these decisions".

Sentürk (1994) lists a few topics, which determines his placement and selection of certain spillways depending on an factor. When looking at the specific dams, focusing on embakments, spillways "cannot be placed on the body of an earth fill dam". The spillway can cause unstability and a point of dam failure. When looking at the factor of geology of the site, the stability of the foundation is important.

A wide variety of spillways have been designed over the years and all have the unique adaptability to why they are being used in conjunction with a certain type of embankment dam. Below follows a list of spillways:

2.2.5.1 Gravity (Ogee) Spillways

This type of spillway is the most common, mainly due to the ease of construction and also due to the various conditions under which this spillway can be constructed. Tancev (2005) states that this spillways has 4 components, namely “approach channel; spillway of frontal type; outlet or diversion part and the terminal part.”. Figure 2.23 shows the schematic diagram of an ogee in embankment cross-section and placement in terms of embankment slope. The approach channel is opening at the top of the spillway, which dictates the amount of water that can enter the spillway. The approach channel is often curved in plan view, which is linked for a greater surface area to transport water, down and away from the dam. The spillway is the overflow passage that takes the water over the embankment. The outlet part is an extension of the spillway, which forms a downward directing chute, which takes the water further to the terminal. The terminal is the end of the spillway, where a toe feature is formed. The toe is curved upwards to decrease the speed of the water. The toe completes this spillway and the water is able to drain safely. The sides and bottom of the spillway, is constructed from concrete, to sustain the force of the water. Chadwick, Morfett and Borthwick (2013) describe that the spillway is often applied to concrete or masonry dams, due to “sufficient crest length to obtain the required discharge”.

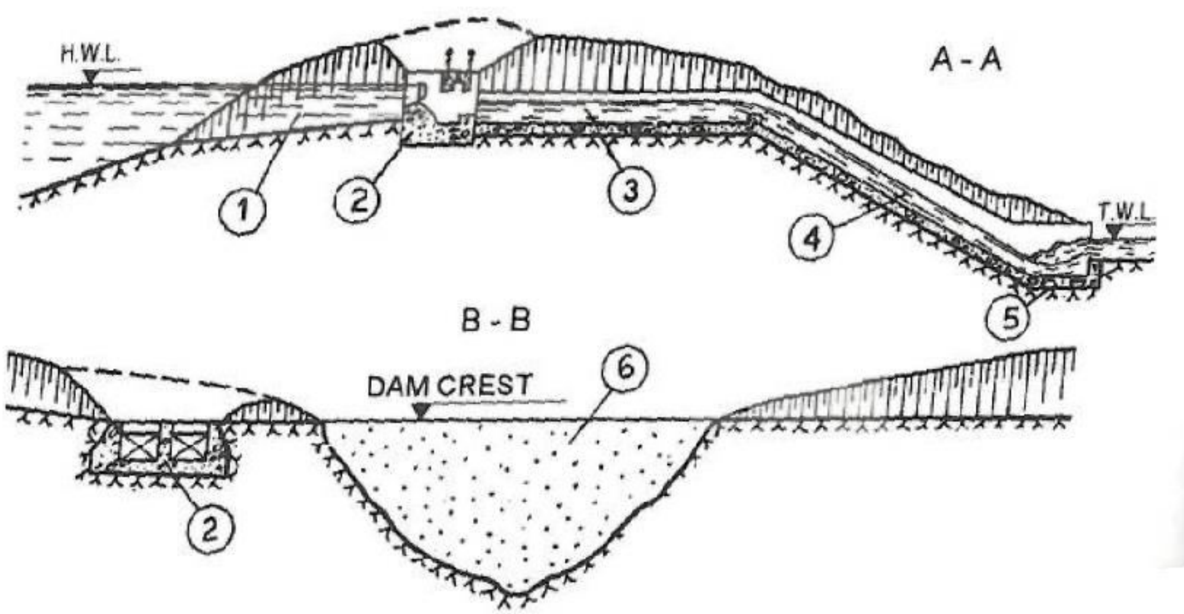


Figure 2.23 Ogee spillway in cross-section of embankment (Tancev, 2005)

2.2.5.2 Spillway chute

Tancev (2005) describes the spillway chute to be “a channel with a longitudinal gradient greater than the critical one”. The size of the chutes entrance and terminal ends might either be straight or variable, with the later taking two approaches of either increasing or decreasing towards the terminal end. The increase or decrease in the chute width may not happen along the whole chute and may occur only along a certain section of the chute. The change in width, is very much dependent on the energy dissipation required at the terminal end. Refer to Figure 2.24 which shows increasing and decreasing of the spillway chutes, along the length. Chadwick, Morfett & Borthwick (2013) points out that spillway chutes "are often used on earthfill dams". As previously mentioned, topography and geology has an effect on spillway design and with spillway chutes, it affects the ratio of the width versus that of height.

Tancev (2005) states that the spillway is designed for the most severe condition, these being "for a full channel, without the action of lateral earth pressure, and for an empty channel, with the action of lateral earth pressure". The spillway chute is ideally designed in a straight line, but due to changing ground conditions, may lead to the design being curved. Many other features are often included in the design, including low sills which decrease velocity of the water and longitudinal parting walls which divert the water in a controlled way. In some cases a cascade offtake of water is considered, when "significant longitudinal inclination of the ground, for instance greater than 25%, and relatively small specific discharge (up to $15m^2/s$)" (Tancev, 2005). The design is based on a step system, with each step slowing down the water a bit. Figure 2.25 shows the design of a cascade spillway.

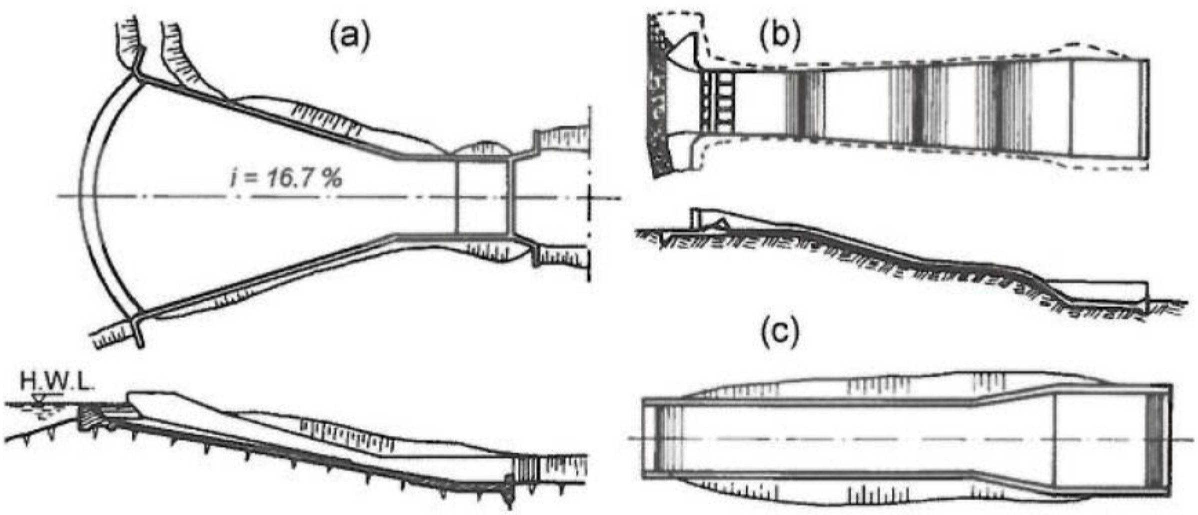


Figure 2.24 Spillway chute (Tancev, 2005)

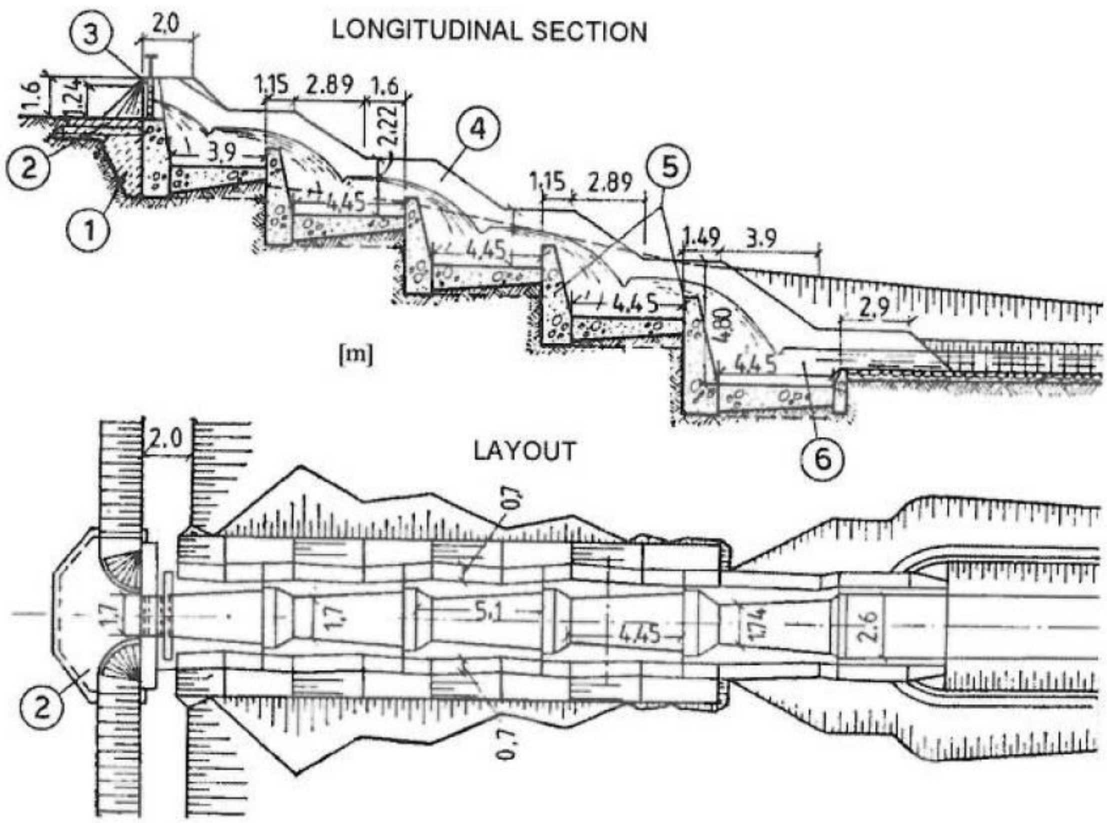


Figure 2.25 Cascading spillway (Tancev, 2005)

2.2.5.3 Side-Channel spillway

In the manual General Design and Construction Considerations for Earth and Rock-Fill Dams (2004), "the size, type and restrictions on location of the spillway" are factors to consider for the choice/type of dam. If soil from spillway excavations can be used as material in earth-fill embankments, an earth or rockfill dam might be the best option. As previously stated, by having embankment dams made from earthfill, the spillway can't be situated on the bank, unless a part of the face has been concrete or rockfill face, with two flanking earth abutments. Onsite factors like topography and natural restrictions will also have to be looked at when considering the location of the spillway. See figure 2.26 for the schematic diagram and placement of a side channel spillway.

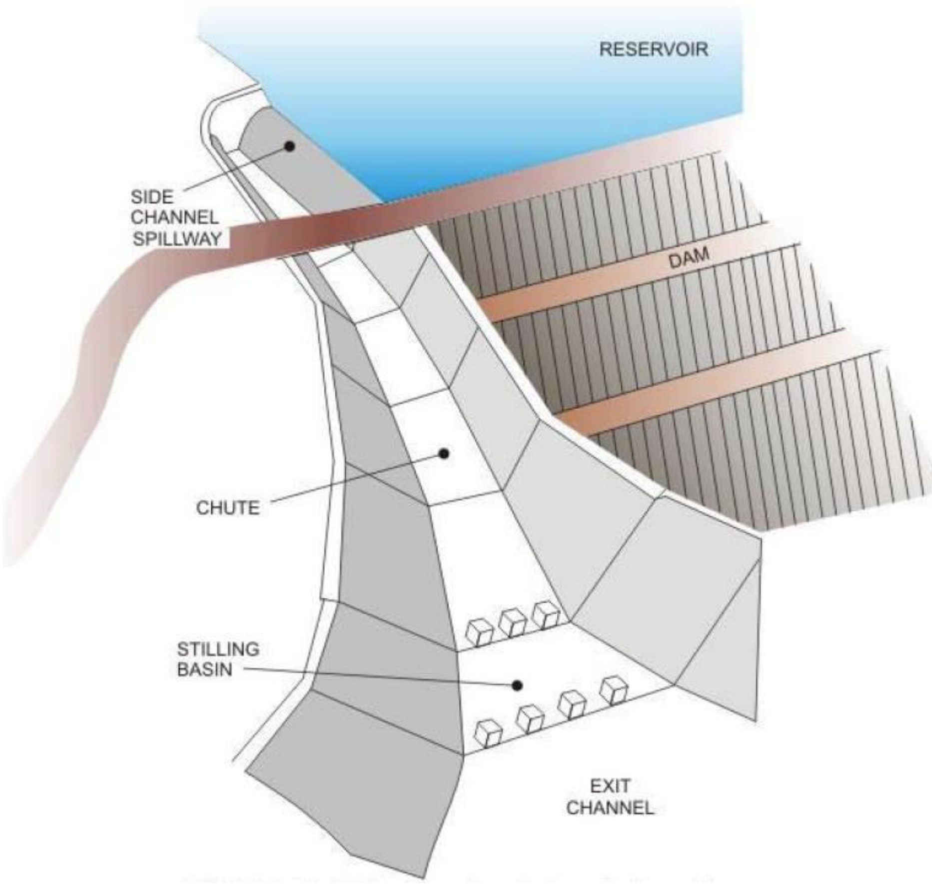


Figure 2.26 Side channel spillway (Solanki, 2014)

2.2.6 Earthquake loading (seismic hazard)

The Western Cape and especially certain parts of the Cape Supergroup have various numbers of faults that occur in the region. Although not as active and dangerous as some fault areas in the world, planning and design as if earthquakes can occur. One only has to look at what happened in Tulbagh, in 1969, where earthquake reaching 6.3 on the Richter scale was recorded (Meso, 2011). The problem that arises with earthquakes is that it adds extra loads in both the vertical and horizontal directions.

Fell, MacGregor, Stapeldon and Bell (2005) lists the following as effects that earthquakes cause on embankments:

- Settlement and cracking of the embankment, particularly near the crest
- Causes instability in the upstream and downstream slopes
- Reduction in freeboard due to settlement or instability (may cause overtopping)
- Differential movements between the embankment, abutments and spillway structures, which ultimately leads to cracks

- Internal erosion and piping developing in cracks, may occur
- Differential movements on faults passing through the dam foundation
- Liquefaction or loss of shear strength due to increase in pore pressure, caused by the earthquake
- Outlet works may be damaged and as a result may lead to erosion of embankments ,from internal leakage

A “defensive design” approach, is often taken when designing for the effects of earthquakes. Engineers base most designs on experience and make certain addition and premeasures to design, that take into account the possibility of an earthquake happening, over the embankments design life. Fell, MacGregor, Stapeldon and Bell (2005) state “these measures are at least as important (probably more so) as attempting to accurately the stability during earthquake”. The following can be considered as important measures to include in the planning and design:

- Having ample freeboard. This allows for cases where the crest may be displaced by the movement of the earthquake.
- Have correctly designed and constructed filters in the downstream slope. This will help control erosion, in the event that the core or face cracks. Fell, MacGregor, Stapeldon and Bell (2005) states that “filters should be taken up to the dam crest level, so they will be effective in the event of large crest settlements”.
- Increasing the amount of drainage features in the dam’s design. If any discharge would happen, due to leaks, water would be able to drain easily through system. Fell, MacGregor, Stapeldon and Bell (2005) gives the order in decreasing resistance of the dams are “concrete face rockfill, sloping upstream core earth and rockfill, central core earth and rock fill, earthfill with chimney and horizontal drains, zoned earth-earth rockfill and homogeneous earthfill”.
- Materials that can liquefy, due to the earthquake, should either be removed or not considered in the embankment material design or foundation. Densification of the soils is often considered, as the removal of the soil may add to the economic costs of the dam. Vibro-flotation, dynamic consolidation, vibro piles or stone columns are ways of densifying the soil. Fell, MacGregor, Stapeldon and Bell (2005) state that the stone columns “have the added role of draining excess pore pressures developed by the cyclic loading”. It is important to compact granular material contained in filters or as rockfill. If this compacted material becomes wet, the material should not liquefy.
- Sharp changes in the shape of the cores foundation, should be tried not to occur in the design. Fell, MacGregor, Stapeldon & Bell (2005) point out that if there are sharp changes, it makes the “core more susceptible to cracking, due to differential settlement under earthquake (and normal) loading”.
- Founding the dam on a rock foundation is often preferred to that of a soil foundation.
- Surrounding slopes that occur on the reservoirs edges should be stabilized, to prevent slides into the reservoir. This is also important of any dam features like spillways.

2.2.7 Stability of design

Although not as important in the desk study phase of the investigation of planning and designing an embankment, it is important that once the various properties of the material and design has been finalized , that one looks at various ways that your design can possibly fail. This is important, as one can safely change dimensions or materials, which from your original design might cause failure. The below mentioned points, may include some previously mentioned stability issues, but are summarized below for convenience. There are mainly three types of failures that may occur, failures caused by hydraulic, seepage and structural causes.

1.) Hydraulic causes:
    - i) Overtopping of embankment: Once an embankment dam is constructed, is often too late in certain cases to change certain parts of the design, possibly due to a lack of information or a result of natural disasters. One of the mayor design incorporations is predicting a worst case scenario for rain in your catchment, providing assurance for the possibility that if that design flood occurs that the embankment design can handle the occurrence. If this design criterion is not met, either by an adequate freeboard or sufficient spillway, the dam level will raise and result in the overtopping of the dam. As a result, erosion of the outer layer of the embankment will occur, which ultimately could lead to whole embankment failure. See Figure 2.27 label E, for this type of failure.
    - ii) Wave erosion: Inadequate protection of the upstream slope and a possible too low freeboard, may lead to erosion of material due to the interaction of surface waves with the upstream slope.
    - iii) Toe erosion: If water movement through the embankment is not controlled or managed, erosion can occur at the toe of the dam and as a result lead to systematic failure of the dam.
    - iv) Erosion of downstream: On the downstream face, inadequate protection from erosion caused by rainwater can cause instability in the embankment. Cracks may form on the surface of the downstream face and as a result can lead an imbalance in the soil.

2.) Seepage causes:
    - i) Seepage through the dam: This mechanism of failure, involves the movement of water through your designed structure, which slowly erodes the material away that your have placed on the embankment slopes. As Statler (2015) mentions in a presentation, this leads to the material moving downstream and “eventually breaches the dam”. This process is known as piping. Seepage can also occur through the foundation of the embankment, which also leads to the same result of dam breaching. The process of seepage not only involves water, but the water can also displace the material into the soil of the surrounding foundation. Embankment failure occurs as a result in the instability of the designed forces. Solutions to this, lies in controlling the movement of water, without have any material movement. See Figure 2.27 label A and G, for this type of failure.

ii) Downstream slope instability: If the seepage lines end up exiting above the toe, a small slide can occur in this wet area, due to the difference between the wet and dry parts of the downstream slope.

3.) Structural causes:

i) Earthquake in area: As previously mentioned, the occurrence of an earthquake in the vicinity of the embankment, can lead to dam failure. As mentioned in the previous topic before, the earthquake may cause a loss of strength in the soil, which ultimately "leads to instability and failure by dam overtopping" (Stateler, 2015). The failure mechanism, liquefaction, results in the soil in the embankment losing shear strength and result in the displacement of this soil due to the force of the water behind it. Another result of failure, as a result of earthquakes, is the formation of cracks along the embankment. Seepage could then occur and as mentioned above, will lead to dam failure. See Figure 2.27 label C, for this type of failure.

ii) Pore water pressure: If proper drainage doesn't occur, pore pressure can lead to water stresses not being distributed properly, which can lead to a reduction in shear strength.

iii) Downstream failure: This slope is vulnerable at full dam capacity, as sliding can occur due to an increase in seepage force.

iv) Foundation sliding: Foundation instability can occur if the soil which the embankment is situated on is soft or is made up of fine material. See Figure 2.27 label H, for this type of failure.

v) Leaching: Soluble salts can leach out of certain soils, which cause an imbalance in the soils make up and as a result can cause failure.

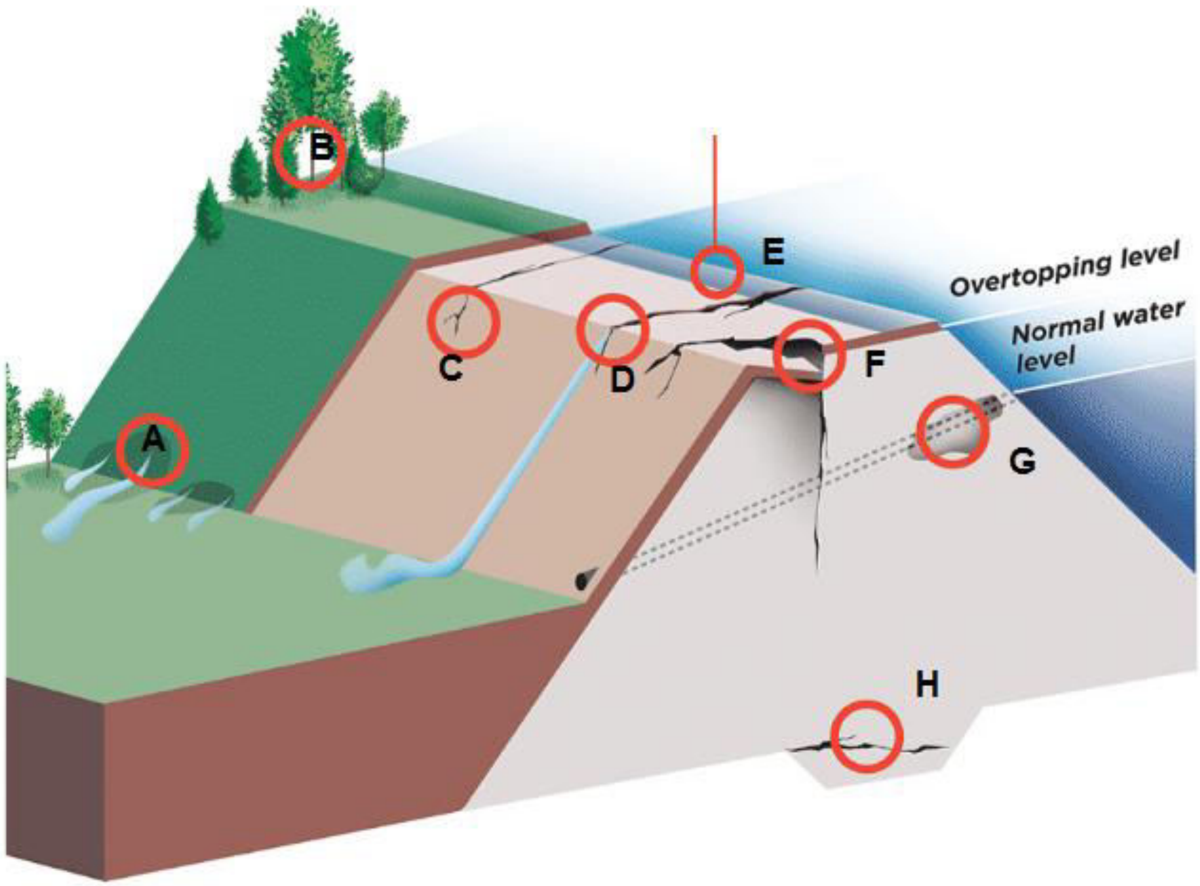


Figure 2.27 Figure showing typical embankment failures (Getschow, 2015)

2.2.8 Availability of construction materials

The less expensive solution are normally the solutions adopted to solve certain engineering problems. In terms of construction material, it is the material that is the easiest accessed. According to the General Design and Construction Considerations for Earth and Rock-Fill Dams (2004), "the most economical dam will often be one of which materials can be found within a reasonable haul distance from the site". Therefore as mentioned above, consideration will be taken to incorporate as much of the natural occurring material in the embankment construction, as long as that the materials adheres to certain properties.

2.2.9 Climatic conditions

The farm is situated in the Western Cape of South-Africa and falls in an area which has a Mediterranean climate over it. The area receives most of its rain in the winter months and this can be confirmed by looking at Figure 2.28 below, which shows the average rainfall (in mm) per month. January and February are the hottest months and these months also correlate to the months which have the least amount of rainfall. It's also these months, which the new proposed dam, will attend to alleviate the water scarcity for the farm. The three yearly rainfall Figures 2.29 -2.31, shows that the rainfall reaches a maximum of 1000-2000mm per annum in 2013, to a minimum of 200-300mm per annum for 2015.

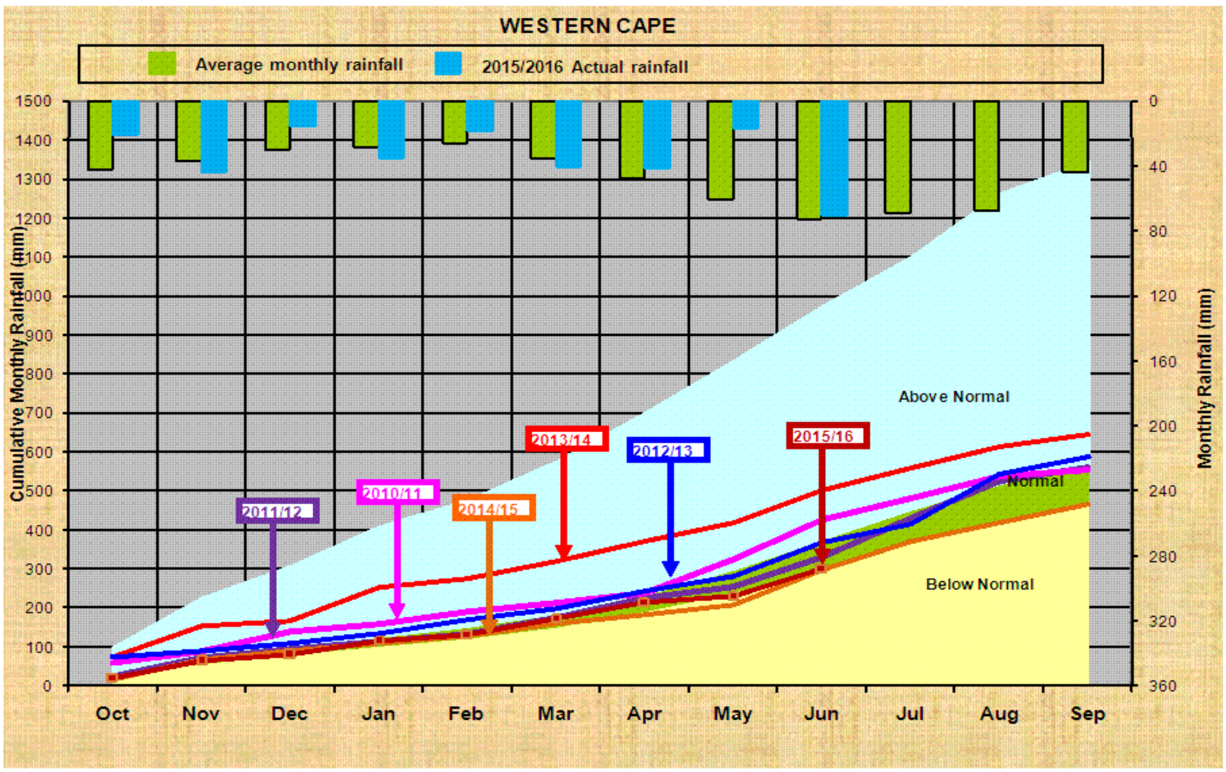


Figure 2.28 Rainfall data over a few years (Department of Water and sanitiation- Republic of South-Africa, 2008)

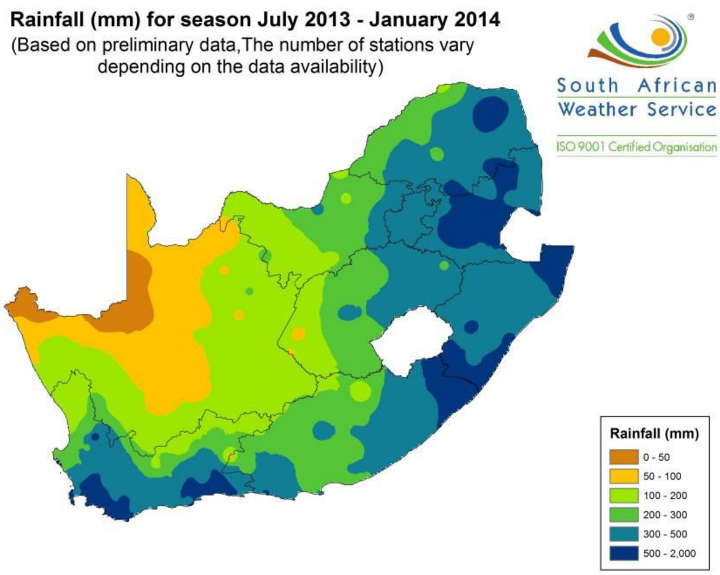


Figure 2.29 Rainfall data for 2013-2014 (South African Weather Service- Historical rain maps)

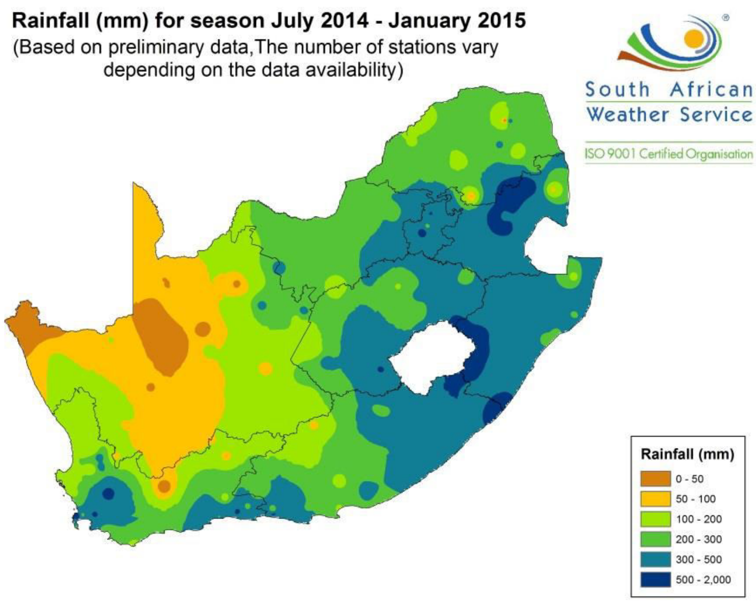


Figure 2.30 Rainfall data for 2014-2015 (South African Weather Service- Historical rain maps)

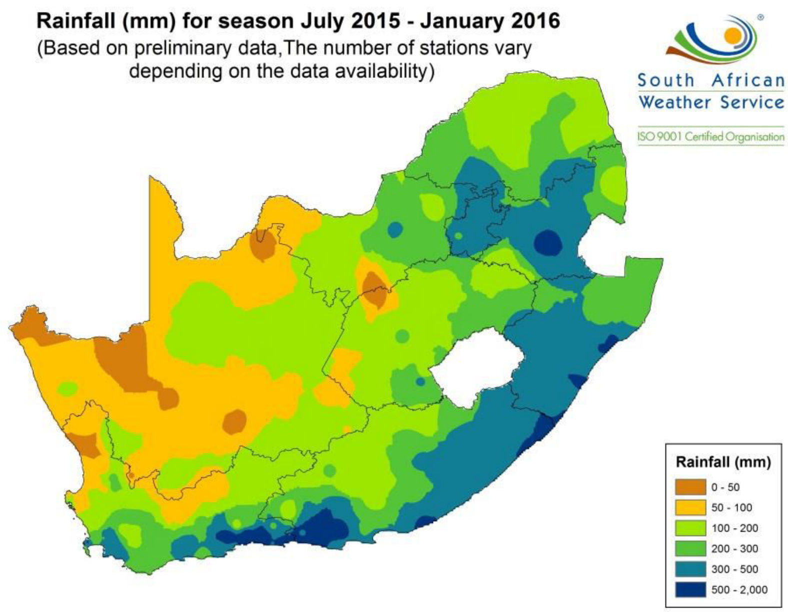


Figure 2.31 Rainfall data for 2015-2016 (South African Weather Service- Historical rain maps)

2.2.10 Environmental considerations

Dams provide one basic need that we need for survival and that is to have available drinking water. In terms of the farms context, it provides available water to irrigate crops with, as well as provide drinking water for the cows. Therefore with any new construction, there are always benefits which are the reason behind the construction, but one also has to look at the negative aspects that could be associated with your proposed plan. When considering any type of construction, whether it planning a new dam, road or building, it's important to know what occupies the current land that you want to construct on. More often than not, it's utilizing un-used land or rural parts of an area. These areas are often unoccupied and an uninterrupted ecosystem has been established on the proposed construction site. It is therefore important that certain studies take place, to assess the impact the proposed new construction might have on environment.

Before we start to discuss the negative and positive factors a new dam might have, Tahmiscioglu, Anul, Ekmekci and Durmus (2007) points that it is important to note that "Wherever the location of a dam is, its ecological results are the same".

Possible negative effects of having a dam constructed

- Sediments that may have entered the river system, now gets trapped in the dams catchment area.
- Changes in nutrient content and oxygen of the water may change, as well as changes in the temperature of the water

- The dam acts as a barrier and may disrupt any movement of organisms in the area
- Water quality will change in the area due to storage of some of the run-off
- Evaporation will be prevalent , due to the surface exposure to sun
- The interaction of soil-water-nutrient combination, will be effected downstream

Possible positives effects of having a dam constructed

- The dam acts as a flood control
- Dam will provide water for irrigation, which will allow for land that was previously not irrigated , to be utilized for better farming purposes
- Dam provides drinking water, to people and animals alike

2.2.11 Water license application

Water is one of those resources, that even though it forms one of the basic needs that everyone should have available to them, it is a very controlled resource in terms that there are various rules and regulations that controls the use of it. These rules and regulations have been put in place so that no one over uses water, in terms of illegally blocking/diverting a water supply or pumping extra water out of sources than needed. South Africa's water usage is governed by the Department of Water Affairs (DWA), who controls and regulates the various forms and people that make use of water resources of South Africa. This regulation is carried out by National Water Act (Act 36 of 1998), which regulates who uses and how much water can be used, dependent on the activity or purpose.

In terms of farmers, especially those considering constructing a new dam, would have to apply to the DWA for a water use application. The list below contains a few forms that will have to be completed in terms of a new water use application and also the reasoning behind each form:

- **DWA756 NWA individual form :** This form contains all the applicants personal details.
- **DWA762 NWA Section 21b :** This form pertains to any dam that can store more than 50 000 cubic meters of water and has a dam wall higher than 5 meters, a dam which can be a safety risk pertaining to the law of section 118(2) of the NWA or if the dam in question has been deemed significant by the relevant DWA office.
- **DW793cls form :** This form contains the information regarding any danger to existing structures, in the scenario of a dam failure.
- **DW901 property form :** This form contains the information regarding the property, where the water use application is taking place.
- **DWA787irg form :** This form contains information, regarding the type of crops that might make use of the water out of a dam, for irrigation purposes.

- **DWA784pmp form :** This form contains information regarding pump information, from which water might be used and pumped out of a water source

The above forms can be downloaded from the Department of Water and Sanitation website (Department of Water and Sanitation of the Republic of South Africa) and also has a clear step-by-step guideline on which forms to fill in. It is also important to note that no construction can take place without also filling in the **DW692E form**. Appendix A contains the general forms mentioned above.

2.3 Geosynthetics

2.3.1 Geosynthetics - what are they and their uses

Geosynthetics are designed to aid soil with filtration, reinforcement, separation, drainage or to provide a moisture barrier. Since the creation of geosynthetics, the use of the material has revolutionized the design in various fields. The main feature has come in the construction of buildings, walls and bridges, that all have a soil/ground part to the design. The civil engineering field has greatly benefited, especially the geotechnical section.

All though all termed in one bracket, geosynthetics come in many types. The various types are as follows:

- Geotextiles: As the name suggests, these are textiles designed for the use in soils. The textiles are made out of synthetic fibers and the product is eventually “made into a flexible, porous fabric” as discussed by Koerner (1986). The major usage of geotextiles in your design, is that the textile allows water flow through the material, but varies the flow rate depending on your usage. Geotextiles are made in various ways, in terms of the way the fabric is binded together. The two ways are either as a woven fabric or non-woven fabric. Both ways effect the above mentioned uses in the own way. Figure 2.32 label b, shows the two forms of geotextiles.
- Geogrids: These geosynthetics are grid like in appearance and made out of hard plastics. Depending on the type, geogrids “are stretched in one or two directions for improved physical properties” as defined by Koerner (1986). This principle of stretching, depending on which direction is stretched, allows for forces and loads to be applied in the sometimes elongated directions, but not the other non-stretched direction. Due to its rigid design, geogrids are often used for soil reinforcement and separation. Figure 2.32 label d, shows the various forms of geogrids available.
- Geomembranes: These types of geosynthetics are impervious layers that act as a liquid barriers. The geosynthetic is normally a thin layer of rubber and plastic. Figure 2.32 label a, shows the variable forms of geomembranes.

- Geocomposites: These geosynthetics, as the names suggest, are combinations of many of the above geosynthetics, into one product. Thus by combining two or more of the above geosynthetics, you are able to do various functions, according to the properties of each type of geosynthetic used. Figure 2.32 label c, shows the variable forms of geocomposites.

Figure 2.32 Different types of geosynthetics (Geotextiles in Embankment Dams-Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation, 2008)

All geosynthetics are used to aid with various forms of constructions and normally come into play when there is a lack of materials available and costs to bring in material would be too expensive. It is then that geosynthetics are used, to make up for the missing materials and fulfills the same job as the missing materials. On the other hand it can be seen that jobs can be adjusted to suite site specifications, where enough material may be available, but the use of geosynthetics would be better suited for a smaller design.

.

2.3.2 Individual types of geosynthetics and their application to dams

2.3.2.1 Geotextiles

As defined in the above section, geotextiles are "permeable textile material (usually synthetic) used with soil, rock, or any other geotechnical –related material to enhance the performance or cost of a human-made product, structure, or system" (Koerner, Designing with geosynthetics, 1986).

2.3.2.1.1 Geotextile Properties

There are various factors that go into designing the specific requirements for a geotextile. One not only has to look at the costs and specific use of the geotextile, but also the properties of the geotextile.

2.3.2.1.1.1 Physical properties

Properties of the material, given from the manufactured product

- <u>Specific gravity:</u> In terms of geotextiles, the specific gravity is considered to be "the polymeric feed stock" (Koerner, Designing with geosynthetics, 1986). Koerner (1986) adds that "specific gravity is defined as the ratio of the substance's unit volume weight to that of water at 4°C".
- <u>Mass per Unit Area (Weight):</u> Information regarding the "weight" of the fabric, normally measured in grams per square meter (g/m^2). The mass of the fabric, which is related to the polymer used, has a direct link to the cost of the product.
- <u>Thickness:</u> The distance measured from the upper surface to the lower surface

2.3.2.1.1.2 Mechanical Properties

Properties of the materials resistance to mechanical stresses mobilized from applied loads or installation conditions.

- <u>Compressibility:</u> The change in thickness of the fabric, due to varying normal pressures. The manufacturing way of producing the type of fabric/material, has an effect on amount the material can compress. Figure 2.33 shows the varying compressibility, regarding the different manufacturing styles.
- <u>Tensile Strength:</u> One of the properties that constitutes the usage of a geotextile in design. The property, measures the amount of tensile stress (amount of force over a distance, measured in KN/m) a material can undergo. The Strain of the material, at the different tensile stresses, is also compared of the material. According to Koerner (1986) this is "the single most important geotextile property". From the varying tests, maximum tensile stress, strain at failure, toughness

and modulus of stiffness, are measured per material. Figure 2.33, shows the tensile stress vs strain, regarding the different manufacturing styles.

- Fatigue strength: Ability of an material to undergo various loadings before failure of the material

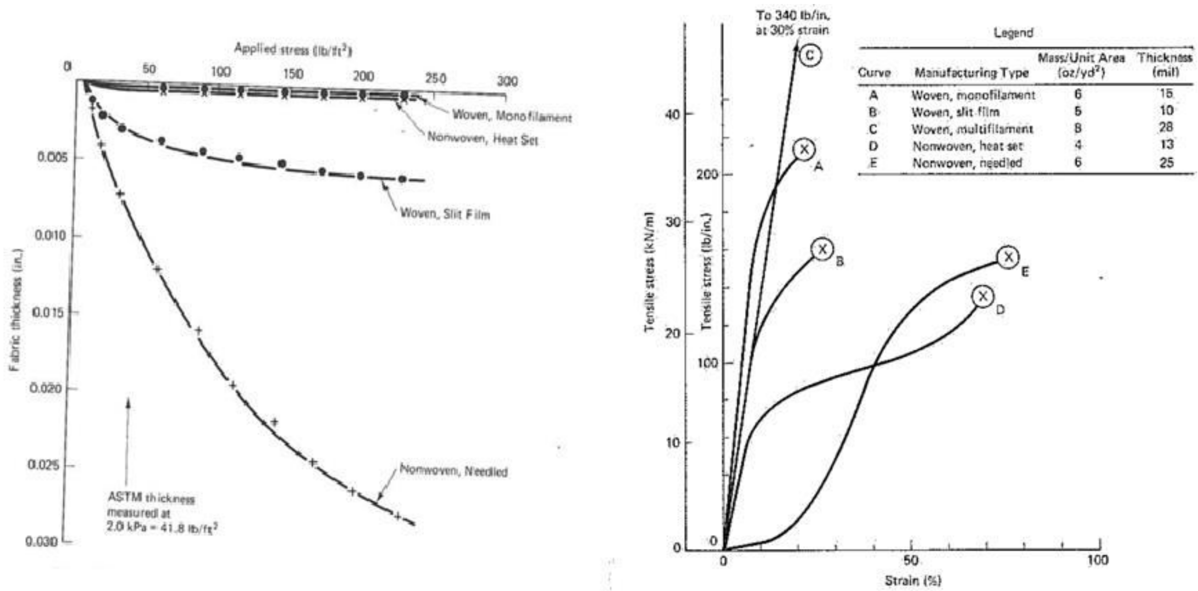


Figure 2.33 Graph on left is geotextile compressibility and graph on right is geotextile strain vs stress relation (Koerner, Designing with geosynthetics, 1986)

2.3.2.1.1.3 Hydraulic properties

Properties of the material, with regards to water application or water interaction.

- Porosity: Ratio of void volume to total volume. So the space that will allow water to flow through. The porosity for geotextiles can be calculated using equation 2.1 from Designing with Geosynthetics (1986).

$$n = 1 - \frac{m}{pt} \qquad (2.1)$$

Where n = porosity

m = mass per unit area

p = overall fabric density

t = fabric thickness

- Percent open area(POA): Property mainly for monofilament woven fabrics, this property compares the "total open area (the void spaces between adjacent fibers) to the total specimen area" (Koerner, Designing with geosynthetics, 1986).
- Apparent opening size(AOS): Property referring to the sizes of the voids in material/fabric. Relates to the sizes of soil particles which may pass through the fabric.

- Permittivity (Cross-Plane Permeability): Refers to the geotextile being able to allow water to flow through, without impeding the flow through. Due to the effects of forces changing the compressibility of the material, a new term is made, permittivity (Ψ). Permittivity can be calculated using equation 2.2 from Designing with Geosynthetics (1986).

$$\Psi = \frac{k_n}{t} \qquad (2.2)$$

Where Ψ = permittivity

k_n = permeability coefficient (hydraulic conductivity) normal to the fabric

t = Thickness of the fabric

This permittivity term is used in conjunction with the Darcy's Law formula (2.3) to create the following equation 2.4 from Designing with Geosynthetics (1986):

$$q = kiA \qquad (2.3)$$

$$= k_n \frac{\Delta h}{t} A$$

$$\frac{k_n}{t} = \Psi = \frac{q}{(\Delta h)(A)} \qquad (2.4)$$

Where q = flow rate

Δh= head lost (change in height)

A = area of fabric under test

Various flow rates (q) are measured, by changing the Δh. When these values are "plotted (i.e., ΔhA versus q) the slope of the resulting straight line yields the desired value of permittivity (Ψ)" (Koerner, Designing with geosynthetics, 1986).

- Transmissivity (In-Plane permeability): Property that relates the flow through of water in the plane of the fabric/geotextile. The compressibility of the fabric/geotextile also has an effect here and another term is created, transmissivity (θ). The transmissivity property is determined as follows in equation 2.5 from Designing with Geosynthetics (1986):

$$q = k_p i A \qquad (2.3)$$

$$= k_p \frac{\Delta h}{L} (W)(t)$$

$$k_p t = \theta = \frac{(q)(L)}{(\Delta h)(W)} \qquad (2.5)$$

Where θ = transmissivity

k_p = permeability coefficient (hydraulic conductivity) in the plane of the fabric

t = thickness of the fabric

q = flow rate

L = length of the fabric

Δh = head lost (change in height)

W = width of the fabric

2.3.2.1.1.4 Environmental properties

Properties of the material regarding there interaction with environmental factors, which could cause usage limitations

- Resistance to chemicals: Depending on the type of material used in the making of the geotextile, chemicals may have an effect on the above mentioned properties. These chemicals may then lower some of the above mentioned properties.
- Resistance to temperature: Also linked to the type of material used in the making of the geotextile, temperature could have an effect on the certain geotextile
- Resistance to elements: With this property, the effect of the weather, as well as sunlight may have an effect on the function of a geotextile. Sunlight exposure is tried to be kept at a minimum, thus geotextiles should be installed and covered immediately.
- Resistance to bacteria: Interaction of micro-organisms/bacteria may have an effect on the geotextile, as a favorable environment can be created, with a moisture source. Bacteria may decompose the geotextile fabric over time.
- Resistance to Burial deterioration: The effects of being buried in the soil and the way the various properties, both mechanical and hydraulic, are functioning over the period since it has been installed. A loss in mechanical and hydraulic properties can be expected over a long period of

time, but this not a too great a loss. Koerner (1986) states "maximum losses of 30%" of properties where experienced, for a geotextile that had been installed for over 12 years.

2.3.2.1.2 Geotextile Functions

As mentioned in the definition of geotextiles in the opening of Chapter 5, geotextiles have the following functions:

- 5.2.1.2.1) Separation
- 5.2.1.2.2) Filtration
- 5.2.1.2.3) Reinforcement
- 5.2.1.2.4) Drainage

Each function will be discussed in terms of how a geotextile aids in the various functions and then in the following section, we will look at how a geotextile can aid embankment dam construction.

2.3.2.1.2.1 Separation

The use of geotextile, which functions as a separator in-between two different materials, allows for both materials to do their intended function without disrupting the integrity and structure of the two different materials. Take for example , we have a coarse rock aggregate that gets placed on top of a fine sandy material The geotextile here intends to prevent two possibilities from occurring, the first being the sandy from entering the coarse rocks and the other possibility is that of the coarse aggregate entering the sand. If the fine sands enter the coarse aggregates voids, then the "drainage capability" (Koerner, Designing with geosynthetics, 1986) of the coarse aggregate can change. On the other hand, if the coarse aggregate enters the fine material, the aggregate loses its intended strength. Figure 2.34 explains the two situations above and how a geotextile can help.

In another book, Koerner (1984) states the following added advantages from using geotextiles as means of a separator of two soils:

1. Simplicity in construction
2. Less excavation
3. Less weight placed on underlying soils
4. Less time required for construction
5. Lower project costs
6. Less chance for problems and/or errors to arise

Figure 2.34 clearly shows how geotextiles can be included as alternatives to filter soils, in the designing of zoned earth embankments.

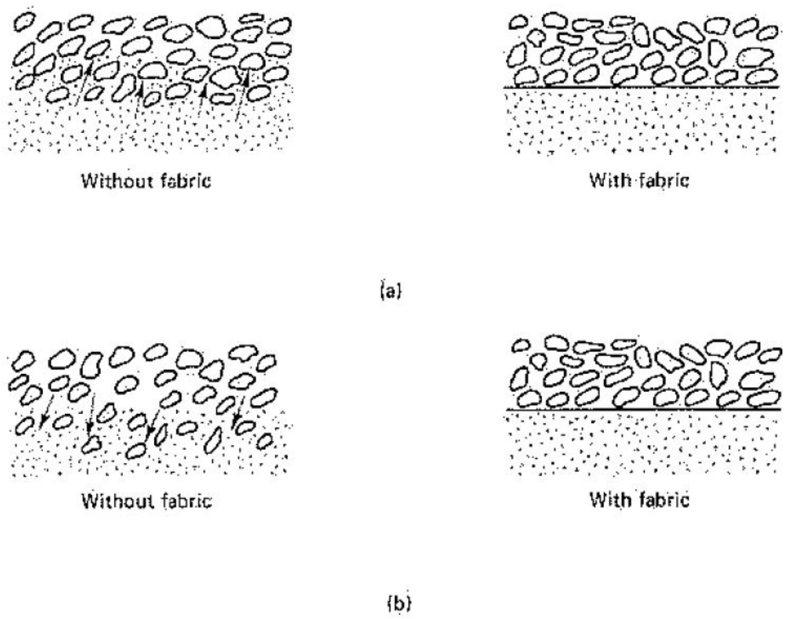


Figure 2.34 Diagram showing soil structure with and without geotextile (Koerner, Designing with geosynthetics, 1986)

2.3.2.1.2.2 Filtration

The use of geotextile, which functions as a filtration device, allows for the movement of water to pass through the manufacturing plane while maintaining the soil structure on either side of the fabric. For the geotextile to do the above mentioned function, it needs “both adequate permeability (requiring an open fabric structure) and soil retention (requiring a tight fabric structure)”, as noted by Koerner (1986). The above mentioned properties are linked to the geotextiles permittivity and soil retention. The permittivity property, is mentioned in the above section on hydraulic properties, but is clearly governed by the thickness of the geotextile layer. The soil retention mentioned, is linked to the geotextiles apparent opening size (AOP).

Depending on the material that the geotextile must retain, the opening size will be smaller than that of the material being retained. If this doesn’t happen and the AOP is bigger than some of the finer materials, it leads to “soil piping, where the finer soil particles are carried through the fabric, leaving large soil voids behind”, as investigated by Koerner (1986). Over time, water velocities can increase the removal of fines, ultimately leading to the collapse of the soil .The ability of the geotextile not to clog will also play a role in its functionality of a filtration device. Koerner (1986) points out that the following situations should be avoided, as it can lead to clogging:

1. Cohesionless sands and silts
2. Gap-graded particle size distributions
3. High hydraulic gradients

Figure 2.35 shows a few examples of ways the geotextile can clog.

Koerner (1984) states the following added advantages from using geotextiles as means of a filtration barrier, as well as the drainage functionality:

1. Less excavation required
2. Less soil to dispose of
3. Faster installation
4. Greater system stability
5. Tensile strength of the geotextile added to the system
6. Lighter load placed on the subsoil's
7. Less technical detail in planning and construction
8. Generally lower costs

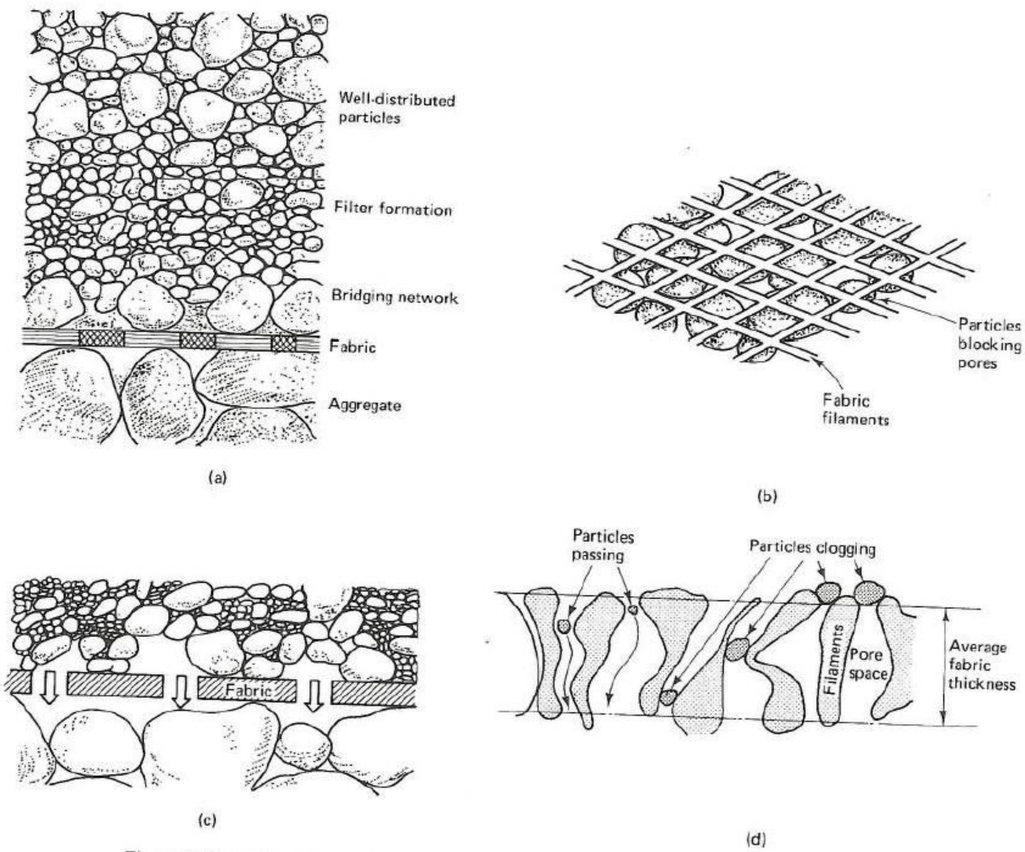


Figure 2.35 Problems which can occur along geotextile face (Koerner, Designing with geosynthetics, 1986)

2.3.2.1.2.3 Reinforcement

Geotextiles can be used to strengthen weak tensile soils, as the material fiber has a high tensile strength property. The following Figure 2.36 shows 4 curves, with each curve being linked to set of geotextiles placed in a sandy soil sample. The figure indicates that by placing geotextiles between one another, you increase the force that is needed, to lead to shear failure. Koerner (1986) points out that there "are three mechanisms" that leads to the type of reinforcement that can be given, namely "membrane type, shear type and anchorage type".

Membrane reinforcement "occurs when a vertical load is applied to a geotextile on a deformable soil". The force that is experienced by the geotextile can be expressed by the following equation 2.6 from Designing with Geosynthetics (1986):

$$\sigma_h = \frac{P}{2\pi z^2}[3sin^2\theta cos^3\theta - \frac{(1-2\mu)cos^2\theta}{1\ +\cos\theta} \qquad (2.6)$$

Where σ_h = horizontal stress at depth z and angle θ

P = applied vertical load

z = depth beneath surface where σ_h is being calculated

μ = Poisson's ratio

θ = angle from vertical beneath surface load P

Beneath the load, where θ = 0°, the σ_h can be determined using equation 2.7 from Designing with Geosynthetics (1986).

$$\sigma_h = -\frac{P}{\pi z^2}\left(\frac{1}{2} - \mu\right) \qquad (2.7)$$

Using the above equation and with the knowledge that the Poisson's ratio (μ) is lower than 0.5, we get a negative σ_h forming. This negative σ_h, results in a tensional force that is experienced in a horizontal plane, beneath the force. It is important to note, that here "tension results in the geotextile, which is precisely the objective" (Koerner, Designing with geosynthetics, 1986).

Shear reinforcement can be seen in Table 2.4. Using the various configurations of geotextiles shown in figure 2.36, shear box tests can be conducted to determine the effect of the geotextiles, on the cohesion and friction angle.

Anchorage reinforcement is similar to the above mentioned reinforcement, "but now the soil acts on both sides of the geotextile as a tensile force tends to pull it out of the soil" (Koerner, Designing with geosynthetics, 1986). Thus the anchorage of the geotextile is compared under two forces that act in different direction on the fabric.

Table 2.4 Soil to fabric friction angles and efficiencies (Koerner, Designing with geosynthetics, 1986)

Geotextile type	Manufacturer's designation	Concrete sand, $\phi = 30°$	Rounded sand, $\phi = 28°$	Sandy silt, $\phi = 26°$
Woven, monofilament	Polyfilter X	26° (87%)	—	—
Woven, slit film	500X	24° (80%)	24° (86%)	23° (88%)
Nonwoven, heat set	3401	26° (87%)	—	—
Nonwoven, needled	CZ600	30° (100%)	26° (93%)	25° (96%)

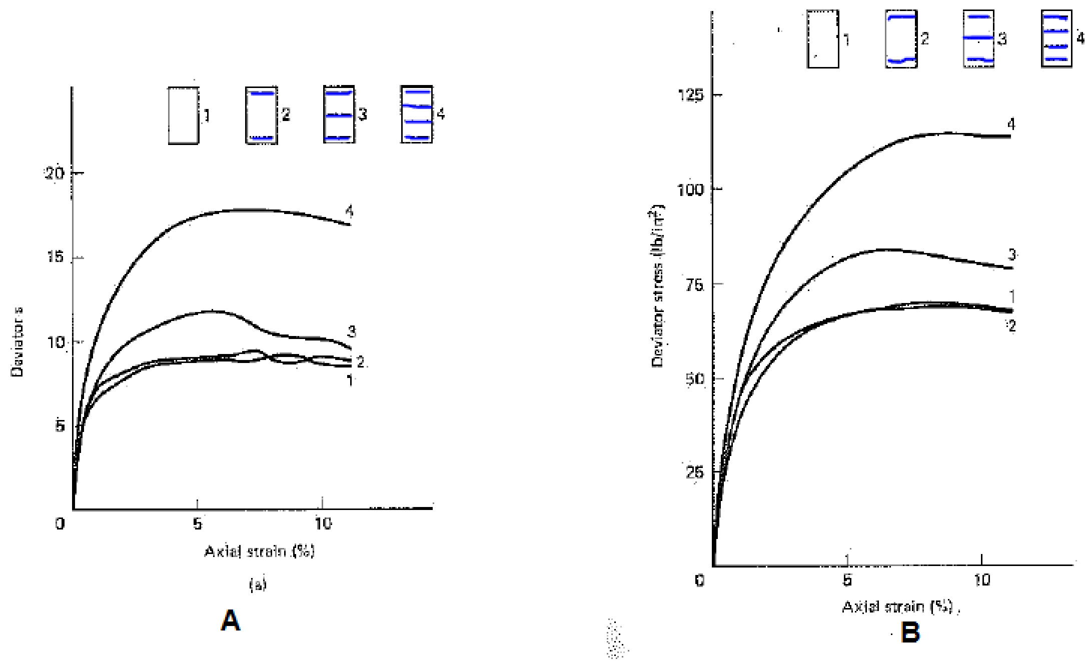


Figure 2.36 Triaxial results showing influence of geotextiles (Koerner, Designing with geosynthetics, 1986)

2.3.2.1.2.4 Drainage

Geotextiles also have the added function of drainage that it can be utilized for. The drainage of the geotextile can be linked to the geotextiles permeability, its soil retention and long term compatibility. For the permeability, we are referring to the drainage of water in the plane of the fabric. The permeability is linked to the thickness of the geotextile at a given time and we refer to it here as transmissivity, as mentioned in the hydraulic properties in the previous section. Equation 2.5 is given below again from Designing with Geosynthetics (1986).

$$k_p t = \theta$$

Where θ = transmissivity

k_p = permeability coefficient (hydraulic conductivity) in the plane of the fabric

t = thickness of the fabric

The soil retention, that the drainage relies on, is linked to the opening spaces of the fabric. The long term capability refers to the life functioning of the product, which is the point where the geotextile will stop performing its intended function. Figure 2.35, shows the ways that might affect the functioning of the geotextile.

2.3.2.2 Geogrids

As previously mentioned, geogrids are "relatively stiff, netlike materials with large open spaces between the ribs that make up the structure" as stated by Koerner (1986). You get 3 types of geogrids, with each type having its own unique properties.

2.3.2.2.1 Geogrid Properties

When looking at geogrid properties, one has to look at the different types of geogrids that you have. The following are all geogrids in figure 2.37 but have varying properties:

- Nondeformed grids, label A in figure 2.37
- Deformed grids, label B in figure 2.37
- Grids, made from joined polymeric strips, label C in figure 2.37

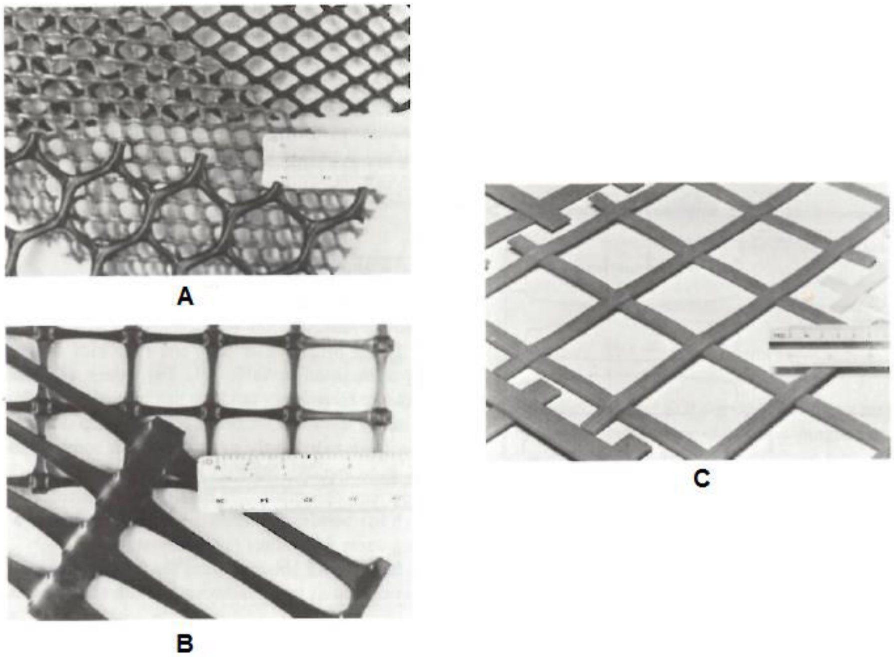


Figure 2.37 Various Geogrids (Koerner, Designing with geosynthetics, 1986)

Physical and mechanical properties

The major property that geogrids poses, is their ability to provide strength either in one or many directions. This is achieved by the way the geogrids are made. For example, a bi-axial geogrid is made by punching holes in polymer sheet and then drawing/pulling the ends in two directions. The geogrids also get strength from how the ribs are joined together. These ribs joints, allow the geogrid, to transfer its stress from transverse direction to the longitudinal direction. Geogrids also provide shear interaction with soil particles, dependent on the size of the grid openings, which adds to stability in soil.

Endurance and environmental properties

Normally made out of high density polymers, geogrids are thus "extremely resistant to chemicals, bacteria, and general aging processes." (Koerner, Designing with geosynthetics, 1986). There UV resistance is also high, which can be due to the addition of carbon black material, in the polymer formula.

2.3.2.2.2 Geogrid Functions

Reinforcement

Geogrids can be used to strengthen weak tensile soils, as the material fiber has a high tensile strength property; this is much the same as that of the geotextile

Drainage and filtration

Due to their large openings, geogrids aren't ideal for the usage in filtration or to act as a moisture barrier.

2.3.2.3 Geomembranes

As previous mentioned, geomembranes are "impermeable flexible barriers usually made from sheets of plastic or rubber, but can also be made from the impregnation of geotextiles with asphalt or elastomer sprays." (Koerner, Designing with geosynthetics, 1986).The primary function of the geomembrane is that of being a fluid barrier.

2.3.2.3.1 Geomembrane properties

There are various factors that go into designing the specific requirements for a geomembrane. One not only has to look at the costs and specific use of the geomembrane, but also the properties of the geomembrane.

2.3.2.3.1.1 Physical properties

Properties of the material, given from the manufactured product

- <u>Specific gravity:</u> In terms of geotextiles, the specific gravity is considered to be "the polymeric feed stock" (Koerner, Designing with geosynthetics, 1986). Koerner (1986) adds that "specific gravity is defined as the ratio of the substance's unit volume weight to that of water at 4°C".
- <u>Mass per Unit Area (Weight):</u> Information regarding the "weight" of the fabric, normally measured in grams per square meter (g/m²). The mass of the fabric, which is related to the polymer used, has a direct link to the cost of the product.
- <u>Thickness:</u> The distance measured from the upper surface to the lower surface
- <u>Water Vapor Transmission:</u> No material can claim that they are 100% waterproof, thus this property is to relate how much water can escape through the material.

2.3.2.3.1.2 Mechanical properties

Properties of the materials resistance to mechanical stresses mobilized from applied loads or installation conditions. (refer to scrim reinforcement)

- Tensile strength at Yield: The property, measures the amount of tensile stress (amount of force over a distance, measured in kg/cm). It is important to note "that with unreinforced geomembranes yield and break (or failure) are approximately the same, whereas for reinforced geomembranes, yield is always significantly higher (and much earlier) than break", as nted in tests by Koerner (1986). See figure 2.38 for examples of tensile strengths.
- Tensile strength at break: The tensile strength that the geomembrane experiences to cause failure.
- Elongation at yield: This is the elongation of the material, when it reaches the yield capacity of the material. This property "is usually expressed as a percent strain and is calculated on the basis of engineering strain, which is the incremental change in specimen length divided by the original length" as stated by Koerner (1986). The incremental change refers to the increase in length. From the figure 2.38 , two situations can be seen, the first being that reinforced geomembranes have high strength but low elongation , whereas the second situation is that un-reinforced geomembranes have low strength but have a much higher elongation. This is a result of scrim reinforcement. Koerner (1986) notes that the "mechanical behavior is markedly changed in that the presence of the scrim increases the strength and decrease the elongation at yield".
- Elongation at break: : The elongation that the geomembrane experiences to cause failure. When looking at Figure2.38, it is good to take note of the 36 mil CPE reinforced membrane. Once it reaches a maximum tensile yield, it takes a time before it eventually. The elongation takes a similar path and it doesn't break when it is at 10 % elongation, but only at 180%. Koerner (1986) notes that this is due to "the fibers of the scrim failed one by one (note the "sawtooth" pattern on the downside of the curves)".
- Modulus of elasticity:
- Tear resistance: This is the property of the geomembrane, which governs at what force the geomembrane might tear at. Due to the low tear resistance of unreinforced geomembranes, Koerner (1986) notes that "extreme care must be exercised during construction", as well as times when the geomembrane is being handled. Scrim reinforcement can increase the resistance of the geomembrane.
- Impact resistance: This property is linked to the geomembrane, being able to withstand objects falling on top of it and tearing into the material. Various angles of object penetration into geomembrane may have an effect on the forces that need to be applied, to result in a tear.
- Puncture resistance: Ability of the material to withstand puncturing, during or after a load has been placed on top of it.
- Soil-to-liner friction: A test similar to a shear box test is done here, to determine the friction angle that can be generated between geomembrane and soil.

- Seam strength: This property refers, to the strength of the seam, which joins the various geomembranes together.

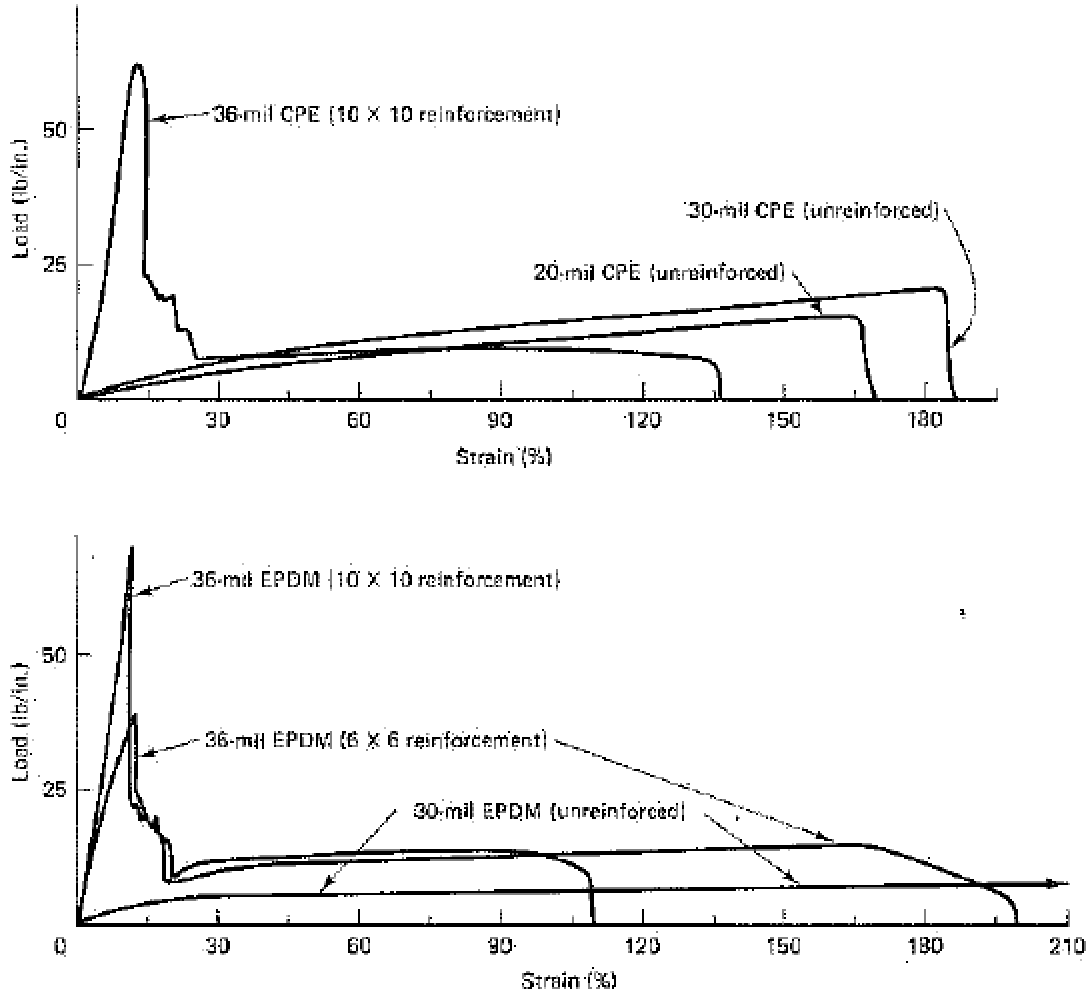


Figure 2.38 Tensile test behavior (Koerner, Designing with geosynthetics, 1986)

2.3.2.3.1.3 Chemical properties

Properties of the material regarding there interaction with chemical factors, which could cause usage limitations.

- Ozone resistance: The resistance of the geomembrane to the ozone, after it has been installed and a piece of the material becomes exposed
- Ultraviolet-light resistance: The resistance of the geomembrane to ultraviolet light, after it has been installed and a piece of the material becomes exposed

- Chemical compatibility: Ability of the geomembrane to function after being exposed to chemicals , which may alter the various other properties of the original membrane

2.3.2.3.1.4 Environmental and life expectancy properties

Properties of the material regarding there interaction with environmental factors, which could cause usage limitations.

- Hot climates or conditions: Changes in the above mentioned properties may occur, if the geomembrane is exposed to heat. Koerner (1986) notes the changes to the "heat driving out volatiles such as moisture, solvents, or plasticizers". The change of the geomembrane is all determined by the exposure conditions.
- Cold climates or conditions: Similar to heat, freezing conditions also can have an effect on a geomembranes. The behavior of the material under cold conditions, effects properties such as flexibility and puncture and tearing possibilities when working with ice.
- Biological interactions: No noted changes of geomembranes have been observed, when in contact with microbes or other soil particles. Geomembranes in turn, aren't harmful to soil environments.
- Liquid absorption: Swelling of material over time, can lead to weakness in the geomembrane , during the geomembranes life span
- Aging: All materials can't last forever, so life span of the material comes into play.

2.3.2.3.2 Geomembrane functions

Due to its properties, of being a very good water barrier, geomembranes are very much often used as a water tight seal. One such application is to use geomembranes as liquid containment liners in small reservoirs. The membranes are often covered with a thin layer of soil to keep it in place, but small reservoirs have also been designed with the membrane exposed. The exposed membranes sometimes degrade, if exposed for long periods of sunlight exposure. Another use of geomembranes is using the material as a cover for reservoirs. There are many advantages to this 1) Prevents water loss to evaporation, 2) Reduces need to drain and clean,3)Protection against pollution and 4) Provides protection against drowning (Koerner, Designing with geosynthetics, 1986). Another usage of this material is to use it as a way to close off and seal the tops of landfills. Through this usage, water wouldn't be able to percolate through the landfill and thus prevent the spread of chemical pollution. Figure 2.39 shows different ways of capping off a landfill sites.

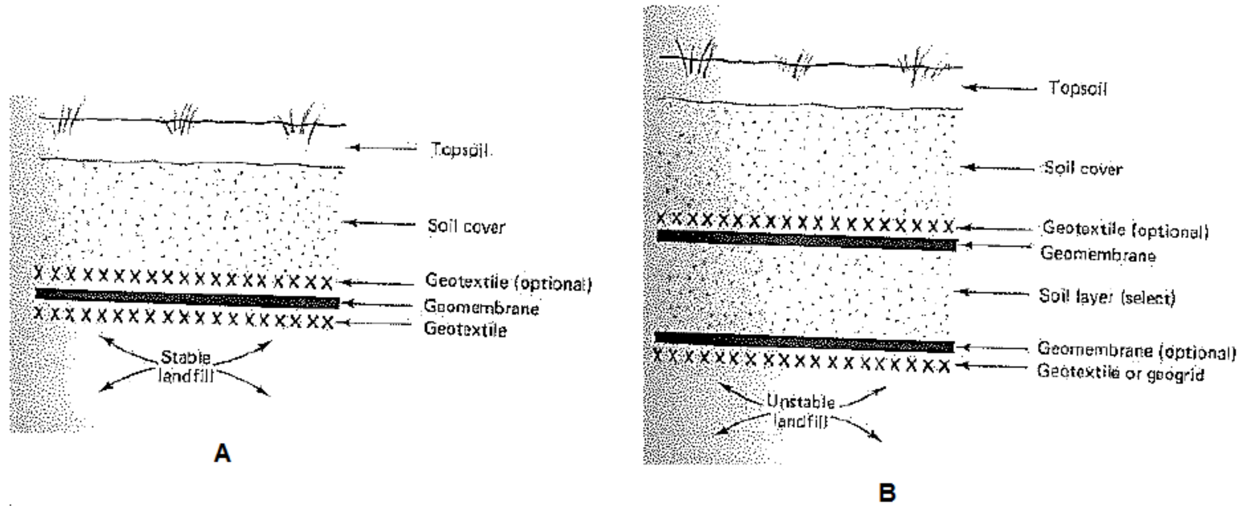


Figure 2.39 Capping of landfill sites (Koerner, Designing with geosynthetics, 1986)

2.3.3 Applications of geosynthetics in dam design

Regarding the information above of geosynthetics, further inset will be given, as to where in the dam design the product/products can be used. These areas are in:

- 2.3.3.1 Filtration and Drainage
- 2.3.3.2 Separation and protection
- 2.3.3.3 Embankment stabilization/Reinforcement

Although not discussed in the above section, geocomposites will be discussed in dam design, as they make use of single properties associated with one material and combine it with another.

2.3.3.1 Filtration/Drainage

In a paper written by Artières, Oberreiter, & Aschauer() reference is made “The first large earth dam using geosynthetic materials was built in 1970 in France”. The geosynthetic in question in this specific dam, was a geotextile, which acted as a filter in both the upper and downstream slope. Due to the properties of geotextiles, as mentioned, it allows for water to seep through, while maintaining the particle structure and design of the soil. Even though geotextiles have advanced in easy application of certain properties , it has been noted by several sources including Artières, Oberreiter, & Aschauer(), Fell, MacGregor, Stapeldon and Bell (2005) and in the status report on the usage of geosynthetics in embankment dams, that geotextiles shouldn’t be used as a critical filter design. “A critical filter is defined as a filter that serves as the sole defenese in protecting the embankment and foundation from internal erosion and piping failures” as documented in the status report on the use of geotextiles in embankment dam construction and rehabilitation.

As the statement mentions, a critical filter is the sole defence, thus if something happens to the geotextile it may lead to dam failure. The reasoning for not using geotextiles, as critical filters, lie in many flaws within the concept of geotextiles. These flaws lie in limitations to geotextile usage, as mentioned earlier, like chemical attack and ultra violet exposure. Other problems are things as product damage during installation and possible biological growth inside the geotextile, which may interfere with filtration. These factors may affect the product negatively and change the designed purpose of the product. As technology within this field advances, current engineers advise the usage of geotextiles to be used in areas of design, like a toe drain. In the following Figure 2.40 we can see where, with regards to filtration and drainage, we can use geotextiles in dam design. An advancement of geotextile technology has been the inclusion of instrumentation within the geotextile, which tests for localised failure (Geotextiles in Embankment Dams-Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation, 2008).

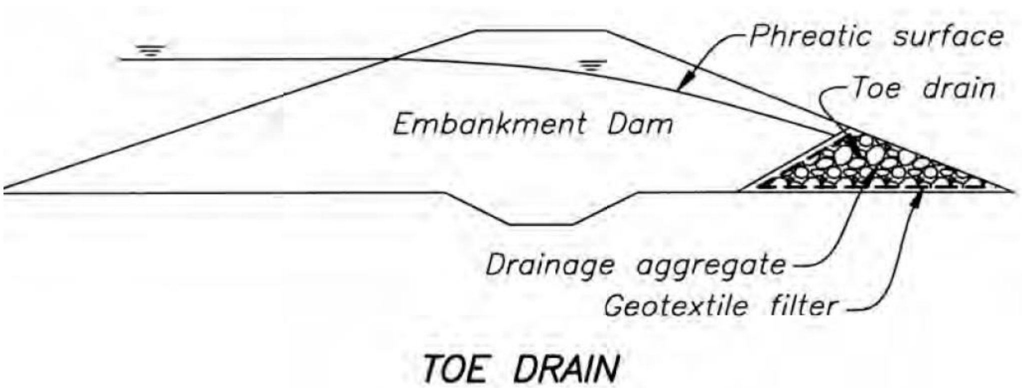


Figure 2.40 Geotextile filter in toe drain (Geotextiles in Embankment Dams-Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation, 2008)

The Valcros dam, was the first earth dam designed with a geosynthetic (a geotextile was used as a filter under the upstream slopes rip rap and at the downstream slopes toe drain filter), was evaluated 6 and 12 years after construction and the performance of the dams filters where “satisfactory”, as noted by Zornberg (2013). The Valcros dam, was built in 1970 in France and when tested sill provided adequate results in terms of the following actions:

- Clean water was observed at the toe filter
- The flowrate that was observed at the drain outlet, adhered to the properties of the geotextile used in the design
- No water was noted along the downstream slope

Drainage

To make use of a geotextile in dam design, a geotextile would also be used as an effective drainage layer in the embankment. As seen in Figure 2.41, a geotextile can be placed along the slope of the clay core, where it will act as a plane to drain water away. This will thus create a way for water to be diverted away from the downstream slope and in that way limit any erosion that may occur if water had free flow through the slope. According to Koerner (1986) that using the geometry available of the designed slope, “a required transmissivity can be calculated using Darcy’s formula”. From this calculation, you can then compare the various geotextile available and see which one yields an acceptable factor of safety. Koerner (1986) suggest “these values of factor of safety should be between 4 and 10”.

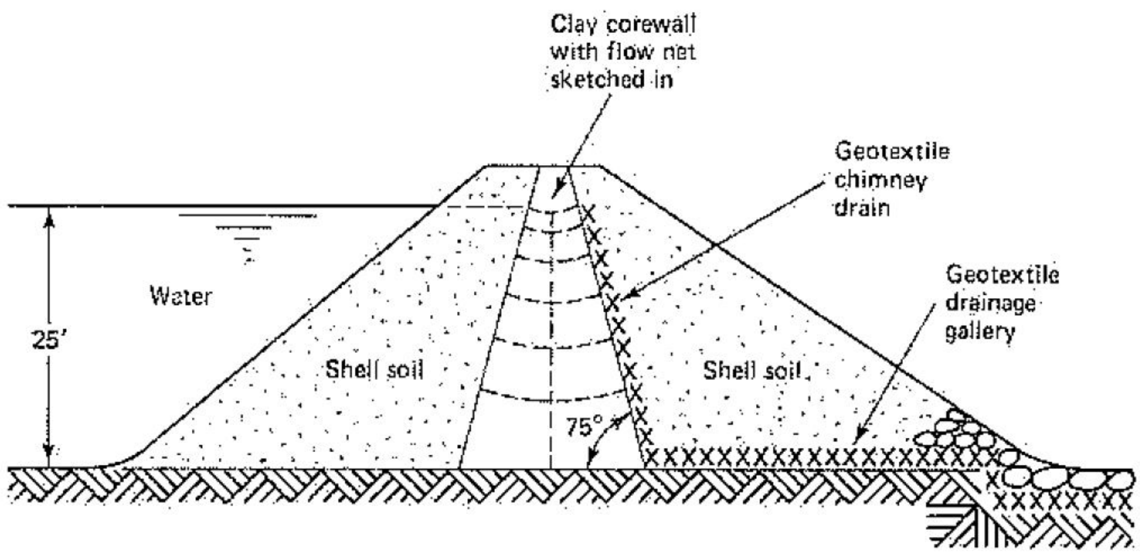


Figure 2.41 Geotextile behind clay core (Koerner, Designing with geosynthetics, 1986)

Another usage of geomembranes, as hydraulic barriers, is using the material as a possible substitute to the clay core in zoned embankments. Due to the material being of low-permeability, it is ideal to limit seepage, as noted by Koerner (1986). This usage of geomembranes instead of clay is ideal when there is limited or no clay available.Geomembranes aren’t fully watertight and infiltration can happen due to “flow through defects”, as noted by Zornberg (2005). Geocomposites, consisting of geomembranes and either geonets or geotextiles, are thus combined to drain off excess water. An example of this was a geonet being installed behind a geomembrane, in the Lost Creek Dam,as noted documented by Zornberg (2005). Another use of a geomembrane in dam design, that of using the geomembrane within the core of the dam as an impervious barrier. The geomembrane can either be used by itself as a substitute for clay material or it can be used in conjunction with a clay barrier, as researched by Zornberg (2005). An example of where this was used was in the Zhoushou Reservoir, in the Sichuan province of China, as stated by Zornberg (2013).

2.3.3.2 Separation and protection

One of the major usages so far in dam construction, for geotextiles, is that of using the product to act as separation layers. The geotextile would be used to separate certain soil layers from each other, as any mixing of adjacent layers, may lead to dam failure, as any change to the composition of the designed layers may lead to excess erosion or drainage. An example would be the transition zone between a granular filter and a downstream earth fill. Figure 2.42 label a, shows how this example looks in design cross section. For the example mentioned, regarding the transition zones, the geotextile would provide assurance of a " definite and consistent boundary of clean uncontaminated material in the drainage aggregate layer" as documented in the status report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation.

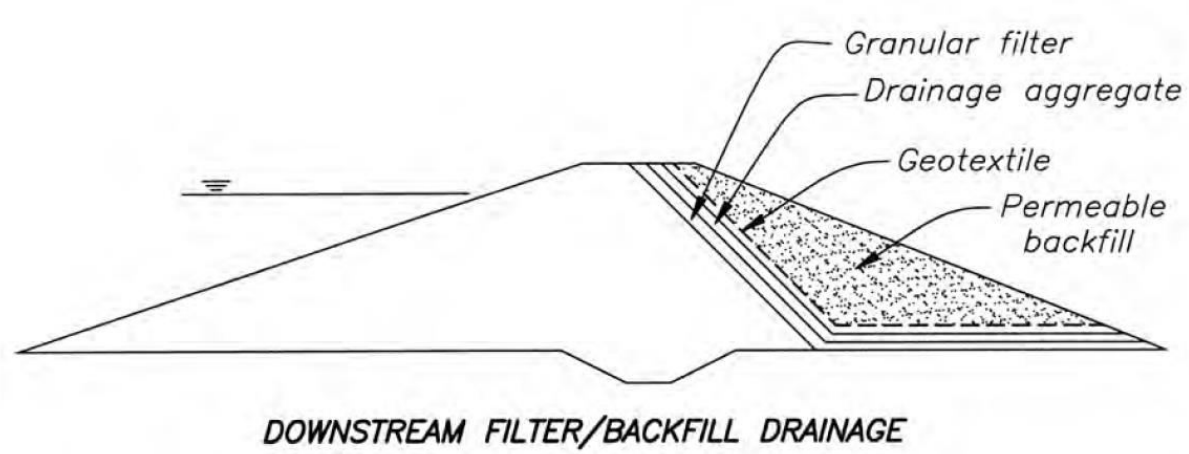


Figure 2.42 diagram showing geotextile separation (Geotextiles in Embankment Dams-Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation, 2008)

Protection

The concept of protection is mainly focused on preventing any erosion or loss of earth. This need to stop the loss of soil and maintain structural integrity is highly important, especially when one looks at phase building in dams. Certain dams aren't completed necessarily after the first phase of building and often need a second or third phase to compete the staged build. With this staged building, it is sometimes needed to alter existing parts of a dam, without any interference from water to maintain the structural integrity of the new construction. For this alternate ideas are needed and one such alternate idea is the use of geotubes. Geotubes or geocontainers, as they are sometimes known as, are geotextiles woven to form tube structures in which sand is pumped into. These tubes once filled, provide a form of protection from erosion and wave action, as water can filter through but known of the sand can pass back. An example of this, was a dam built near Rabat Morocco, as stated by Koffler, Choura, Bendriss, & Zengerink (2008) , where an embankment was built using these geotubes to allow for alterations on certain parts of the dam. Figure 2.43 shows the placement of the geotubes.

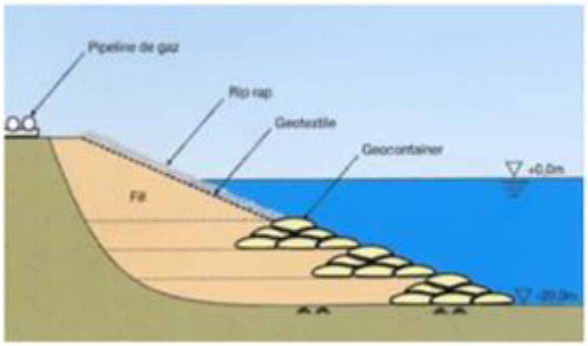


Figure 2.43 The left picture shows the placement of geotubes and the right shows the dry works space (Koffler, Choura, Bendriss, & Zengerink, 2008)

Another form of protection would be that of using geomembranes as a waterproofing layer, within slopes and base. In an article written by Scuero and Vaschetti (2004), it was noted that 232 dams made use of geomembranes and where include in the design of the dam as a exposed or covered element in the dams design. One of the first dams that made use of geomembranes in its design was the Contrada Sabetta dam in Italy, as staed by Zornberg (2005). This dam, was a very steep, as the ratios of the slope where 1H:1V. Geomembranes where "installed as a hydraulic barrier underneath concrete slabs paced on the upstream face" as documented by Zornberg (2005). Porous concrete was then further installed beneath the geomembrane. Another usage of geomembranes, as hydraulic barriers, is using the material as a possible substitute to the clay core in zoned embankments. Due to the material being of low-permeability, it is ideal to limit seepage. This usage of geomembranes instead of clay is ideal when there is limited or no clay available.

Geosynthetics like geonets or geocells can be used to protect the banks of downstream slopes, by providing a solid foundation on which vegetation can be established. Geocells can form a solid foundation, as soil will be contained from eroding away. This property can be further strengthened by placing the geocells on top of a geotextiles. The function of the geosynthetics here would provide the following,1)protect slope from water/rain washing away soil while vegetation is being estabilished,2) keep soil in place,3) slow down surface run-off when vegetation has established and 4) provide the vegetation of a layer for root growth . This was all noted by Thomas(2010).

Not only can geosynthetics be used during the initial stage of the construction of a dam, but also during the rehabilitation of the dam. The Kandaleru earthen bund reservoir, in Chennai province of India, underwent substantial erosion of the upper slope due to wave action (Raju, 2010). A solution was sought after, as to find a way to stop the erosion going further and as a result cause overtopping of the dam. The eventual design that was implemented was to remove any obstruction on the upper slope and place a small sand layer, as stated by Raju (2010). A geotextile was placed on the sand and filled gabions where placed on top of the geotextile. With this system in place, wave action was slowed down, while also maintaining the soil structure of the upper slope.

Reinforcement

Geotextiles, with high tensile strength, can be placed within or below the downstream and upper slope. The extra tensile strength can "reduce stress and strains within the soil mass or embankment enabling the embankment to resist large differential settlements and lateral spreading or slope movements", as stated in the status report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation. With the added reinforcement, your designed system sees an increase in safety factor, less material can be used during construction and one can increase the height of a designed dam. This reinforcement property of geotextiles is ideal when stabilizing a weak foundation on a soft clay layer. During potential earthquakes areas, reinforced soils can aid in providing overall soil equilibrium, in terms of soil stability. Ideally one would us a geo-composite, consisting of a geogrid and geotextile, in the design. The geogrid will add to soil rigidity, while maintaining soil structure due to filtration and drainage. An example of this would be during the January 17, 1995 Hyogoken-nanbu earthquake as noted by Koseki (2012) where reinforced retaining walls in the affected area, survived without some form of damage. The damage was severe, as that it caused ultimate failure and overturning in the retaining walls. One retaining structure, a retaining wall with reinforcement, however suffered minor displacement of 10-20 cm.

Chapter 3: Design of dam

Having considered the information from Chapter 2, regarding the influence of the various factors that go into designing an embankment dam, we must then look towards the types of tests which need to be conducted which will confirm the designing of the various parts of the dam. Various geotechnical and hydrological investigations were conducted and the data collected were used to design the dam.

3.1 Geotechnical investigations

Geotechnical investigations give you the general geological setting and gives you the properties of the soil that you have available, to work with and design your dam with.

3.1.1 Soil profiles

As mentioned earlier, the farm had an uncompleted dam constructed on the upper part of the mountain. The site, the yellow demarcated block in Figure 3.1 below, was the uncompleted construction site. Figure 3.2 is a zoomed in area photo. Due to farming restrictions and a lack to possible sites for dam placement, the uncompleted dam was considered as a good preliminary site to investigate.

Figure 3.1 Farm outline and old dam site (Google Maps)

Figure 3.2 Positions of soil test pits on old dam sites (Google Maps)

Three tests pits where dug along the length of the dam and the soil/rocks in the tests pits were observed and described in the attached Figures 3.4 to Figures 3.6 below. The key to reading the soil profiles, can be seen in Figure 3.3. The terms used in describing the figures, all come from the attached figures in Appendix B. It must be noted that the soil was wet, when the descriptions where made. The soils/rocks in the test pits can be summarized as the following:

- Test pit 1 (TG1) - Combination of sand and organic overburden, with clay layers below that
- Test pit 2 (TG2) - Combination of sand and organic overburden, with silty sand and clay below that.
- Test pit 3 - Weathered sandstone and mudstone.

The positions of the test pits correspond to the numbers in Figure 3.2 above, which is a zoomed in area of the uncompleted dam.

From these tests pits, two soils from TG1 and TG2 were selected and further tests were conducted on them at the Geotechnical laboratories of the University of Stellenbosch. Another soil was noted that was used in the uncompleted dam and a sample of this soil was also taken for further tests. This soil was used on both embankment slopes. It was assumed that this unknown soil was transported to the site, for use on the now uncompleted dam.

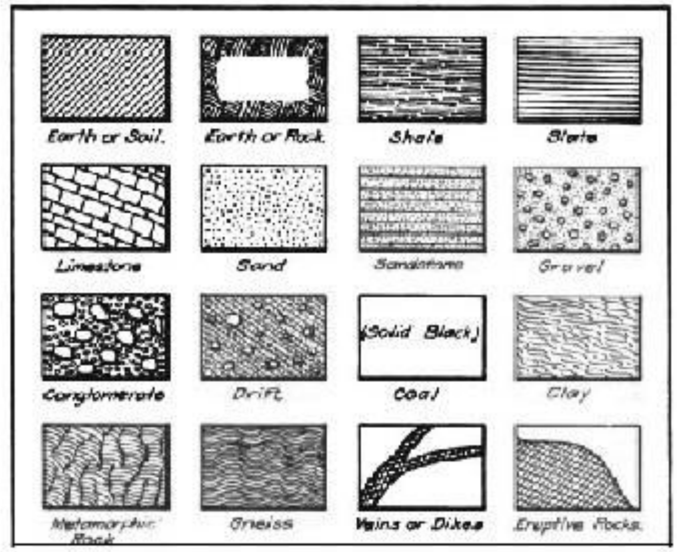


Figure 3.3 Key to geology of soil profiles (Pinterest-Technical Drawing for Geology)

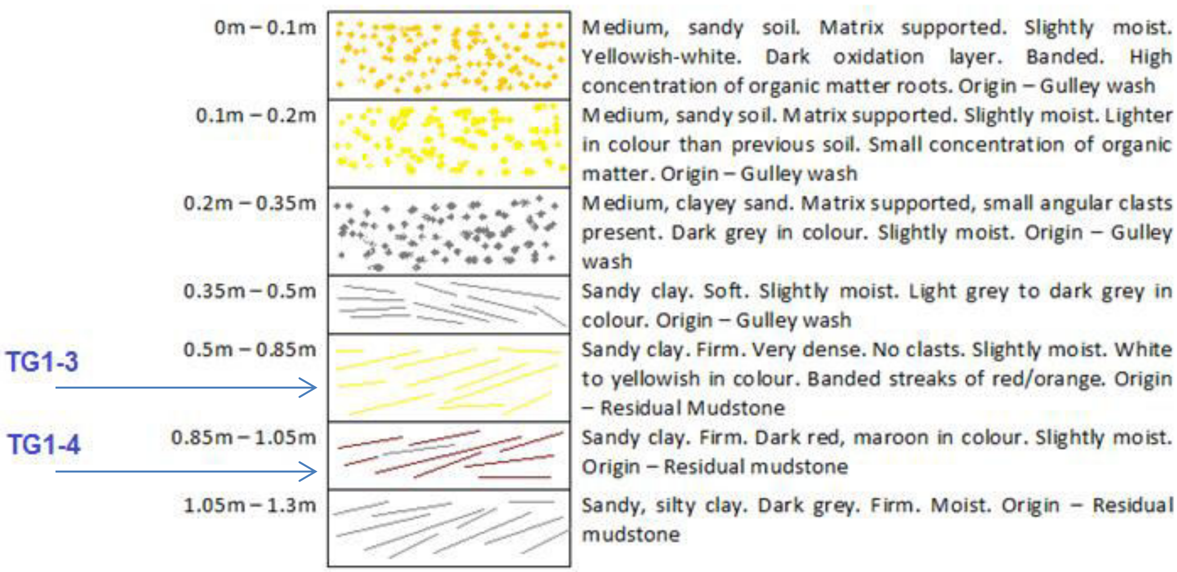


Figure 3.4 TG1 Soil profile

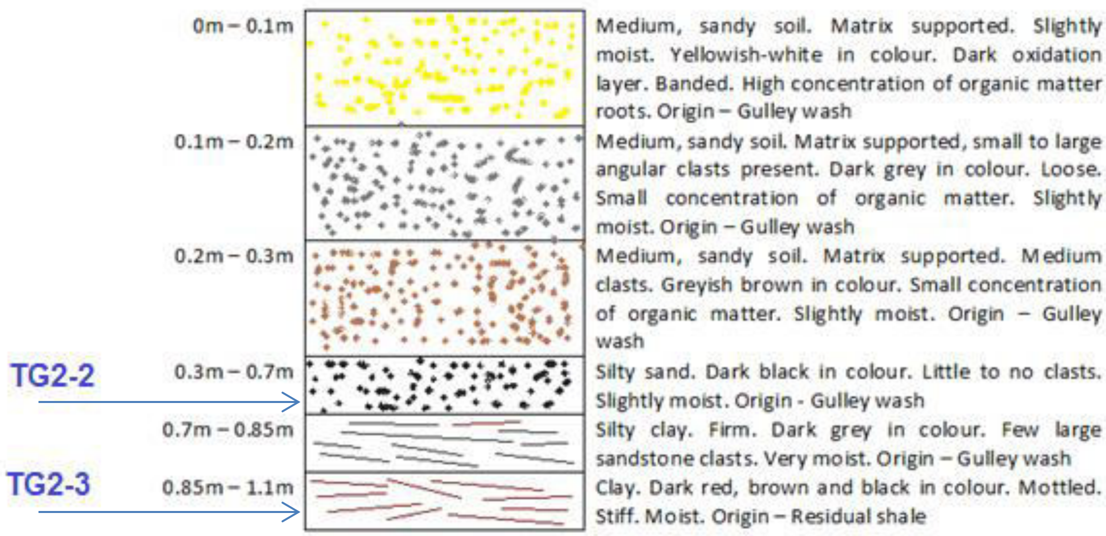


Figure 3.5 TG2 Soil profile

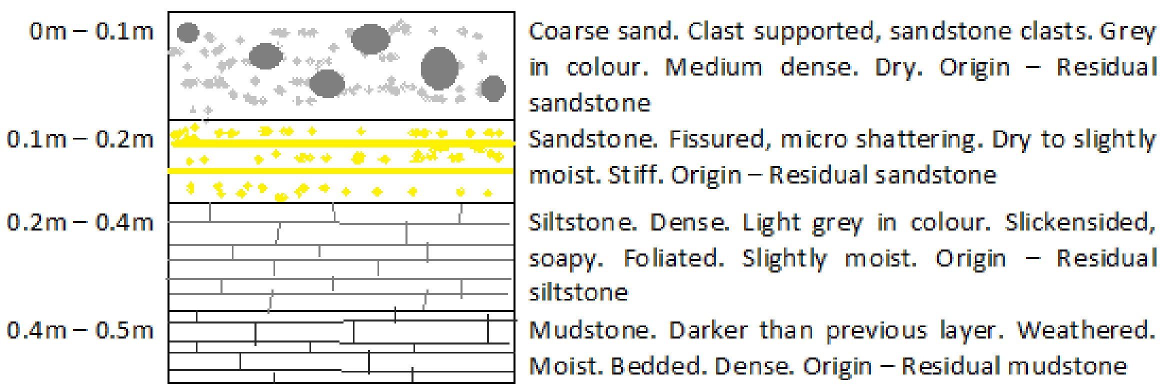


Figure 3.6 TG3 Soil profile

3.1.2 Sieve analysis and Atterberg limits

An analysis was conducted on the above five mentioned soils, to determine the various properties associated with each soil. The analysis was conducted at the Geotechnical laboratories of Stellenbosch University and a full set of results can be seen in appendix C regarding each soil. A summary of each soil will be given in Table 3.1 and Figure 3.7 below, with each soil being classified using the Unified classification system and Unified plasticity chart:

TG1-3 : Clayey sands (SC) – No Plasticity

TG1-4: Clayey sands (SC) – No Plasticity

TG2-2: Silty Sands (SM) – No Plasticity

TG2-3: Inorganic clays, silty clays, sandy clays of low plasticity (CL) –Liquid limit 30.9, Plastic limit 18.3

Dam wall: Inorganic clays, silty clays, sandy clays of low plasticity (CL) – Liquid limit 15.9, Plastic limit 13.2

Table 3.1 Summary of Sieve analysis of soils

PARTIKELGROOTTE	**Persentasie kleiner**				
PARTICLE SIZE (mm)	***Percentage smaller***				
	TG1-3	***TG1-4***	**TG2-2**	***TG2-3***	**Dam wal**
75	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0
26.5	100.0	100.0	100.0	100.0	100.0
19	100.0	100.0	100.0	100.0	100.0
13.2	100.0	100.0	95.8	98.8	100.0
4.75	99.7	100.0	94.2	96.9	98.3
2	99.2	100.0	89.4	94.7	95.4
0.425	88.2	93.4	58.5	73.3	68.7
0.212	64.3	59.7	36.7	57.9	46.6
0.15	46.4	39.4	24.3	47.1	34.4
0.075	34.9	26.1	12.2	37.0	24.0
0.05	19.4	20.5	7.0	25.7	17.2
0.005	13.2	14.0	2.9	19.8	11.0
Liquid limit	/	/	/	30.7	15.9
Plastic limit	/	/	/	18.3	13.2

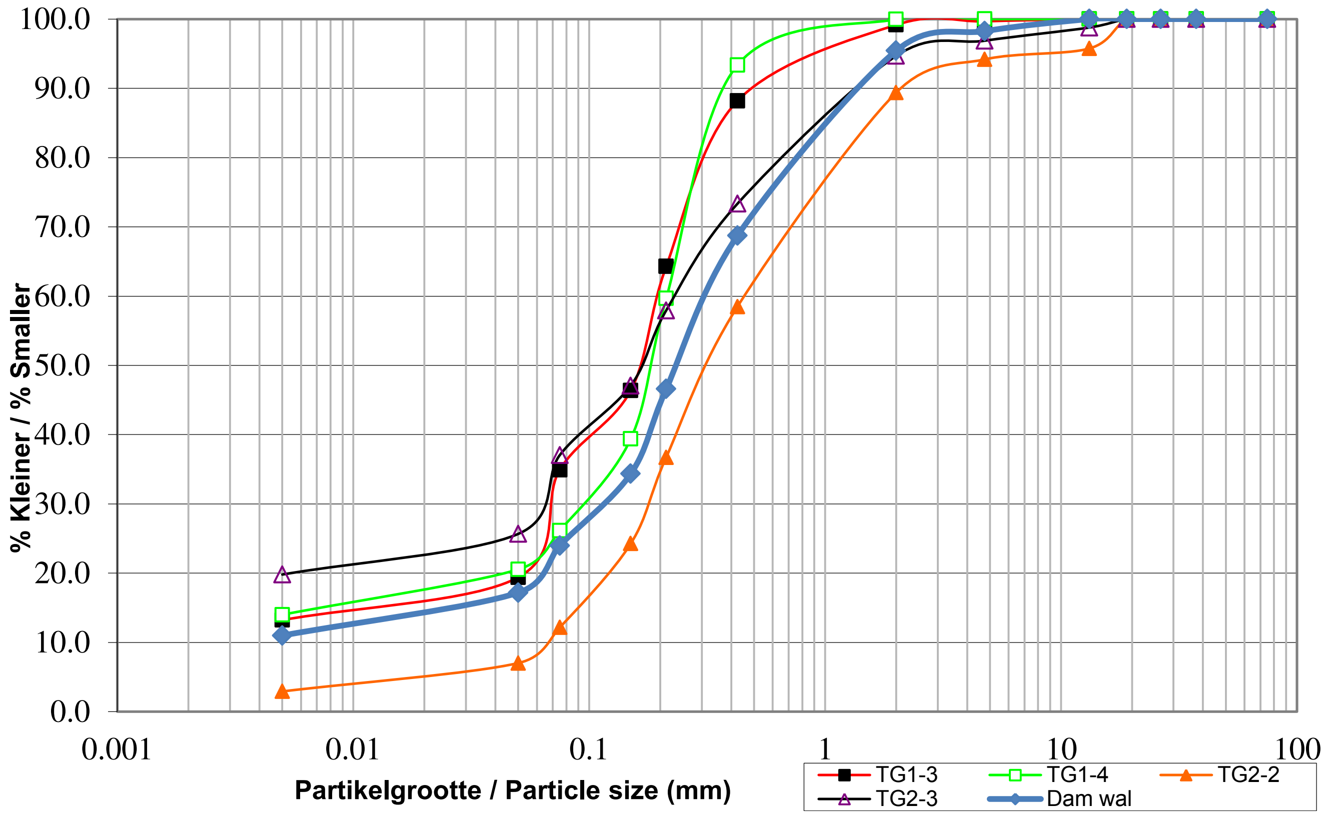


Figure 3.7 Summary of sieve analysis of soil samples

3.1.3 Permeability

For the permeability results, estimations where taken regarding the permeability from graphs given in the Geotechnical Engineering Investigation Handbook (2005). Field tests were also conducted, which the results where compared to.

From the graphs and using the determined D_{max} and D_{min} of the soils, it was estimated that the soils had the following permeability coefficients:

TG1-3 : = 1x 10^{-6} m/s

TG1-4: = 1x 10^{-6} m/s

TG2-2: = 0.01 x 10^{-2} m/s

TG2-3: = 0.5 x 10^{-6} m/s

Dam wall: = 4x 10^{-6} m/s

From these results it was determined that material TG1-3, TG1-4, TG2-3 and the dam wall material, where all material that are very highly impermeable. TG2-3 was determined to be the least permeable of all of them. TG2-2 was determined to be very permeable and offer good drainage.

(Ideally it should be noted that either the constant-head permeameter or the failing-head permeameter test should be conducted on the samples. The reason for not completing these tests where due to Geotechnical laboratory not having these tests available.)

3.1.4 Dispersion

Although material, like clay, can have a low permeability and high cohesion, tunnel erosion can still occur and eventually lead to dam failure. Clay is dispersive, when it's added to water and the clay particles enter suspension. Crumb tests were conducted on three samples and there reaction was noted and classified using Table 3.2 below:

TG1-4: Grade3

TG2-3: Grade 1

Dam wall: Grade 1

Table 3.2 Grading of the dispersion reaction (Maharaj, 2011)

Grade	Reaction	Description
1	No reaction	Crumbs may slake, but no sign of cloudiness caused by colloids in suspension.
2	Slight reaction	Bare hint of cloudiness in water at surface of the crumb
3	Moderate reaction	Easy recognizable cloud of colloids in suspension, usually spreading out in thin streaks on bottom of beaker.
4	Strong reaction	Colloid cloud covers nearly the whole bottom of the beaker, usually as a thin skin.

3.1.5 Dry density

The maximum dry density and optimum water content of each of the five soils, were obtained using the proctor test. Mixes of each soil, at different moisture content was made and compacted using the Proctor machine. Settings where set at using the 304,8 hammer and three compactions of each sample took place, with 55 blows indicating one compaction. The compacted sample was measured and dry density and moisture content where determined using the following equations 3.1 and 3.2 from Craig's Soil Mechanics (2012):

$$p_d = \frac{W}{1+\omega} \, x \, F \qquad (3.1)$$

And

$$W = \frac{M_w + M_d}{M_d} \qquad (3.2)$$

Where: p_d = dry density
ω = moisture content
W = weight of sample
F = mould factor
M_w = wet mass
M_d = dry mass

The various mixes in terms of dry density were plotted against their corresponding moisture content and as a result the maximum dry density and optimum water content was determined.

The results for the five soils can be seen in Appendix D. A summary of the results will be given below:

TG1-3: Max dry density of 1943 kg/m^3 and optimum water content of 8.8 %

TG1-4: Max dry density of 1972 kg/m^3 and optimum water content of 9.0 %

TG2-2: Max dry density of 1940 kg/m^3 and optimum water content of 8.0 %

TG2-3: Max dry density of 1715 kg/m^3 and optimum water content of 15.4 %

Dam wall: Max dry density of 1990 kg/m^3 and optimum water content of 10.0 %

3.1.6 Shearbox tests

Shearbox tests were conducted on two compacted materials. The material that where tested was TG2-3 and Dam wall. These compactions where made as close to optimum water content, that was determined during the proctor results, as to achieve the maximum density. The results below are obtained from Appendix E and shows the original calculations, as well as the actual water content and dry density of the samples that where tested.

TG2-3: Max dry density of 1715 kg/m^3 and optimum water content of 15.4 %
Actual TG2-3 measured: Dry density of 1754 kg/m^3 and 16.7 %

Dam wall: Max dry density of 1990 kg/m^3 and optimum water content of 10.0 %
Actual Dam wall measured: Dry density of 2006 kg/m^3 and 10.1 %

Three samples where cut out of each compacted block and tested at three different confining pressures. Samples where first soaked for 12 hours and then each sample was tested for 24 hours at 0,005m/hour. After the tests were completed, the data was collected and using a design spread sheet the data was converted to graphs, from which the various Mohr's circles where obtained.

Initial results of TG2-3 was obtained, but had to be re-done, as results indicated a negative cohesion.

The summarized results of the tests obtained, can be seen in table 3.3 and full results can be viewed in Appendix E.

Table 3.3 Friction and cohesion parameters for tested soils

	C'	ϕ
TG2-3	10	28
Dam wall	8	35

3.2 Hydrological investigations

3.2.1 Catchment area

The natural catchment area for the dam site is 385 140 m^2. This was calculated using on site map data, as well as using aerial information from Google earth. See Figure 3.8, for the area. The calculation can be seen in the equation 3.3 below.

Figure 3.8 Demarcation of catchment area (Google Maps)

A = dam site + side of the mountain + land above dam (3.3)

= (212 x 187) + (508 x 512) + (610 x 140)

= 39 644 + 260096 + 85400

= 385 140 m^2

3.2.2 Run-off calculation

The total catchment area was determined to be 385 140 m^2. Taking into account the natural rainfall of 650mm for this past year (weatherbase-Caledon,South Africa) and the previous rainfall figures contained in Figures 2.28 to Figures 2.30, where can calculate the approximate value for the water, that is available.

Average water per year = (Value of 2013 + Value of 2014 + Value of 2015 + Value of 2016)/4

= (500 + 400 + 450 +650)/4

= 500 mm

The run-off for the area was calculated using the Rational method and relies on the following equations 3.4 and 3.5 from the Drainage Manual (2013):

$$Q = \frac{Cx\ Ix\ A}{3.6} \qquad (3.4)$$

And

$$C_1 = C_s + C_p + C_v \qquad (3.5)$$

Where Q = peak flow (m^3/s)

C = run-off coefficient (dimensionless)

I = average rainfall intensity of catchment (mm/hour)

A = catchment area (km^2)

3.6 = conversion factor

C_1 =run-off coefficient for rural areas

C_s = factor relating to the Surface slope

C_p = factor relating to the Permeability

C_v = factor relating to the vegetation

For calculating the various C-factors, the factors were taken from the Drainage Manual 6th edition (2013) and will be discussed below.

For C_s in terms of the rural classification, the slopes of the catchment area was considered to be >30 % (steep areas) and using the average rainfall of 500 mm, the factor was taken as:

$C_s = 0.26$

For C_p for rural classification, the permeability of the soil was considered to be semi-permeable (silt, loam, clayey sand) and using the average rainfall of 500 mm, the factor was taken as:

$C_p = 0.16$

For C_v for rural classification, the vegetation was considered to be light bush and cultivated lands, and using the average rainfall of 500 mm, the factor was taken as:

$C_v = 0.11$

Thus taking the run-off coefficient, we get the following:

$$C_1 = C_s + C_p + C_v$$

$$C_1 = 0.26 + 0.16 + 0.11$$

$$= 0.53$$

And therefore

$$Q = \frac{Cx\ Ix\ A}{3.6}$$

$$Q = \frac{0.53 * 0.057077 * 0.385140}{3.6}$$

$$= 3.24 \times 10^{-3}\ m^3/s$$

Thus the total water available is:

$$= 3.24 \times 10^{-3}\ m^3/s \quad = 102\ 177\ m^3/year$$

3.2.3 Volume of water needed

The total area of the farm that needs irrigation is 35 hectares and 5000m^3 of water is needed per hectare per year. Thus the total amount of water needed, is 175 000m^3 yearly. The lowest rainfall for the farm, which was noted for this year, was 27mm in January (weatherbase-Caledon,South Africa). Thus the volume of water for lowest rainfall month was:

Water needed = 0.027 * 35 * 100^2

= 9450 m^3

Thus the storage of the dam is:

175 000m^3 – 9450m^3 = 165 550 m^3

The formula above is used to calculated the minimum water that is available for the site at one time and shows the maximum amount of water that will be needed to be stored, to provide the needed irrigation.

3.2.4 Design flood calculation

The Peak discharge for the area, was calculated using the empirical peak discharge formula, equation 3.6 and 3.7 from the Drainage Manual (2013)

$$Q_T = 0.377x\ K_T x\ Px\ A^{0,6} x\ C^{0.2} \qquad (3.6)$$

Where Q_T = peak flow for T-year period (m^3/s)

K_T = constant for T-year period

A = size of catchment area (km^2)

P = mean annual rainfall over catchment (mm/a)

And $$C = \frac{A\ x\sqrt{S}}{L\ x\ L_C} \qquad (3.7)$$

(Catchment parameter with regard to reaction time)

And where S = average slope of stream (m/m)

L = hydraulic length of catchment (km)

L_c = distance between outlet and the centroid of the catchment (km)

First working out C $= \frac{0.385140\ x\ \sqrt{3}}{0.65\ x\ 0.35}$

= 2.93

Taking the K_T factor for a 1 in 100 year flood, we get the following:

$$Q_{100} = 0.377x\ 1.60\ x\ 650\ x\ 0.385140^{0.6}\ x\ 2.93^{0.2}$$

$= 274\ m^3/s$

3.2.5 Determining the peak flow of the pipe, during the irrigation season

A volume of 165 550 m^3 water is required during the irrigation season of 12 months. A volume of 166 00 m^3 was used for the determining the peak flow rate using equation 3.7.

$$Q_{Avg} = \frac{V_{needed}}{T} \qquad (3.8)$$

Where Q_{Avg} = average flow (m^3/s)

V_{needed} = volume water needed (m^3)

T= time period (seconds(s))

So using the above equation

$$Q_{Avg} = \frac{V_{needed}}{T}$$

$$Q_{Avg} = \frac{166000}{(12\ x\ 30\ x\ 24\ x\ 60\ x\ 60)}$$

$$Q_{Avg} = 5.2638\ x\ 10^{-3} m/s$$

3.3 Final design

3.3.1 Site selection

Due to farming restrictions and space issues, the old dam site has been re-selected for the construction of the new dam. This site adheres to the design critea that was discussed in the literature study in chapter 2 and will elaborate how this site is ideal. The main critea that makes this site suitable for an embankment dam is the topography of the land, but the site is also ideal to the soil that the site is situated on.

The site selection in terms of topography, allows for the spillway to be situated in a stable bedrock foundation on the right hand point of the dam wall. The topography also allows for a straight earth embankment, to be founded in cut in excavations from the previous dam's remnants. Also the topography of the surrounding area, allows for a large catchment area, ideal for the necessary requirements in terms of water supply. As mentioned, the soil of the site also plays an important part in the dam's site selection. Due to the geology of the area, the rocks present and the soils derived from the rocks, make for an ideal site. Most of the dam's foundation can either be constructed on solid bedrock or on a low permeable, but strong clay base. When looking at Figure 3.9 below, the foundations can be founded in hard rock in the black and green blocks, with the red and yellow blocks providing a low permeable clay foundation. Surrounding materials on site, also allows for soil usage in various structures such as in the dam's embankment, core or spillway filling.

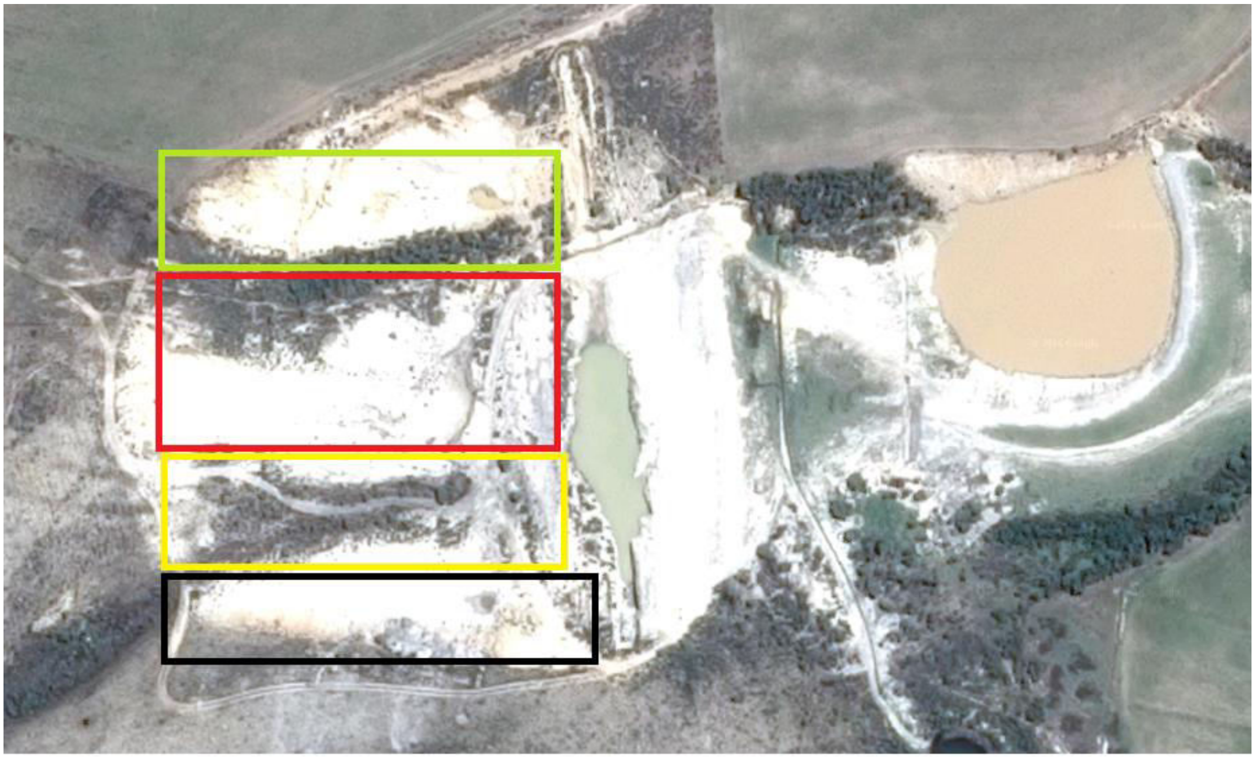


Figure 3.9 Geology of area (Google Maps)

3.2.2 Type Selection, Position of dam and embankment shape

Taking into consideration the old dam's original position, it was decided that a similar approach would be taken in deciding the position of the new dam. With this choice of position, the farmer was able to add in valuable knowledge of how much land we could use, in terms of the embankment placement as well as the area that would be covered by water. This input was definitely considered when the various volumes of the above mentioned elements, where being determined. It was decided that the embankment be placed on the two excavated points, on each of the opposing sides of the valley. With this decision, it will allow the spillway to be put in one of the sides of the embankment.

In terms of the shape of the embankment, due to the constant gradient of the site, it was decided to build a straight line embankment from the one side of the valley to the other. In terms of dam type selection, as clay core embankment was selected, due to the porosity of some of the soils. For a provisional design, an upstream slope gradient of 1:3 and a downstream slope gradient of 1:2 were selected, for design purposes. The core of the embankment, used side gradients of 2:1 and a cut-off trench will also be included. A crest width of 4m was selected, as an access road would be established across the embankment. See figure 3.10 below, for a visual image of design. The specific spillway to be used will be discussed later on. Material usage for Figure 3.10, will be discussed in Stability of the design.

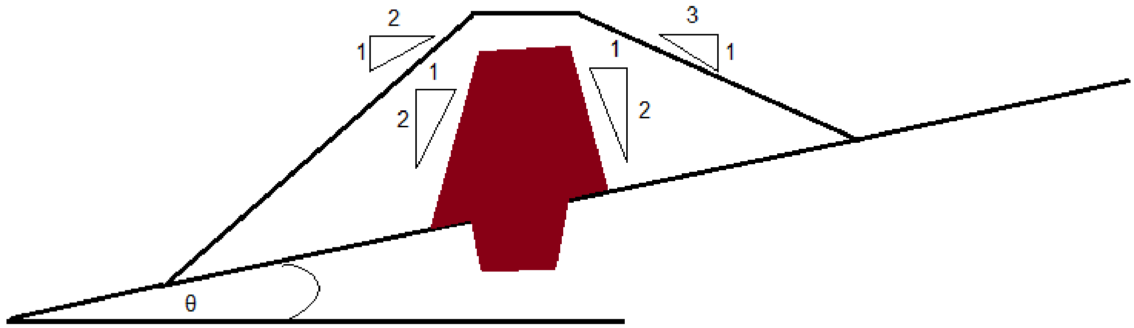


Figure 3.10 Diagram of proposed design

3.2.3 Determining design volumes

To make the dam as economical as possible, it was decided to make use of as much of the natural material available, while maintaining that the dam held the intended 169600 m^3 of water .Using the above information for the design of the embankment slopes, it yielded the following Figure 3.11 and 3.12 below, which was used to determine the volume of the water. The difference between the two figures, is that Figure 3.12 contains a straight, flat piece of horizontal water section.

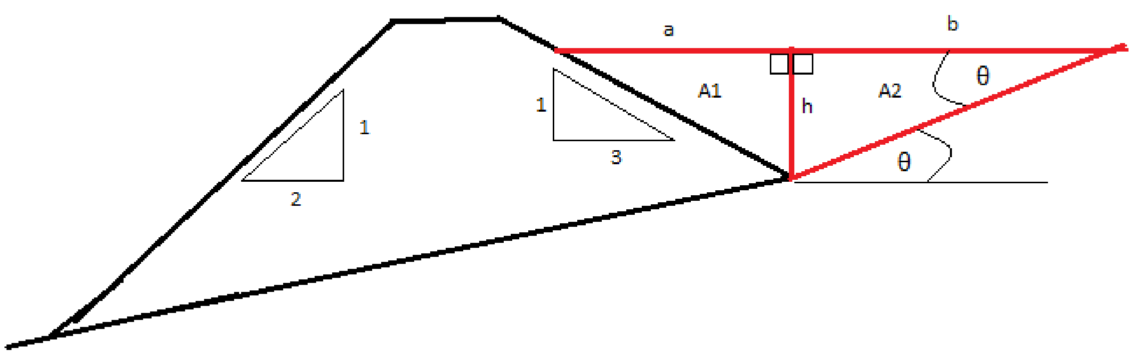


Figure 3.11 Diagram used for volume calculation

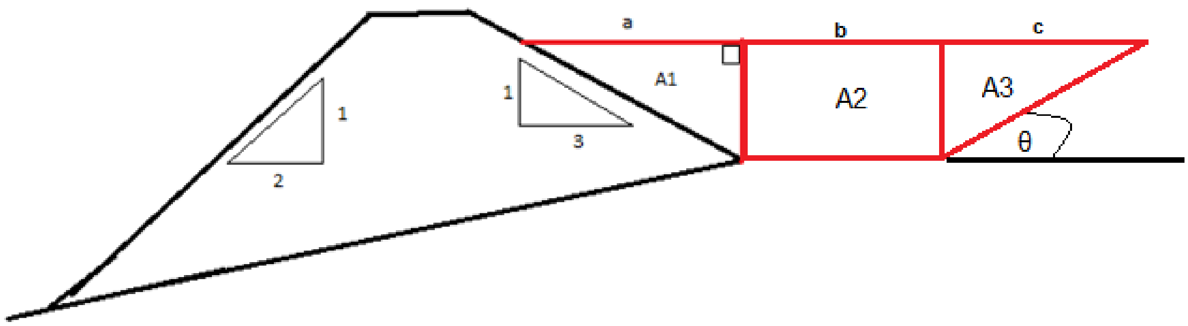


Figure 3.12 Diagram used for volume calculation

From the figure 3.11 and 3.12 above, θ is the angle of the excavation slope behind the upstream slope, A1, A2 and A3 are the relevant water areas, “a”, “b” and “c” are the bases of A1,A2 andA3 respectively and h is the maximum height of the water. By varying the height of the water (h) and the excavation angle (θ), it was tried to maintain a balance of material to be used for construction purposes, as well as have the required 165 550 m^3 of water. A maximum length that a, b and c could add up was determined to be 160 m and the length of the dam to be 200m. The slope that the embankment will be situated on was also varied at different angles, as in Figure 3.13 below. For simplicity of volume calculations, it was decided to use around 166 000 m^3, for the required volume of water. A freeboard of 3m was selected, which will be discussed later in the spillway design. The selected freeboard height, was also selected on the defensive design approach that was discussed in the segment on earthquakes, as to prevent possible failures possibly from overtopping. A crest of 4m was used across the whole design process, as a road has to be situated across the whole of the dam. From the volume calculations, which can be seen in Appendix F, it was decided on the following results:

Height of dam = 13.4m

Volume of Embankment = 237 908 m^3

Volume of excavated soil from slope @ angle (α) of 10° = 348 654 m^3

Volume of excavation behind dam @ angle (θ) of 0° and 20°= 326 729 m^3

For the decision above, after the different volume heights had been determined. Various decisions made in terms of embankment width, height of dam and excess of soil left over. These values are now going to be used for the stability of the design.

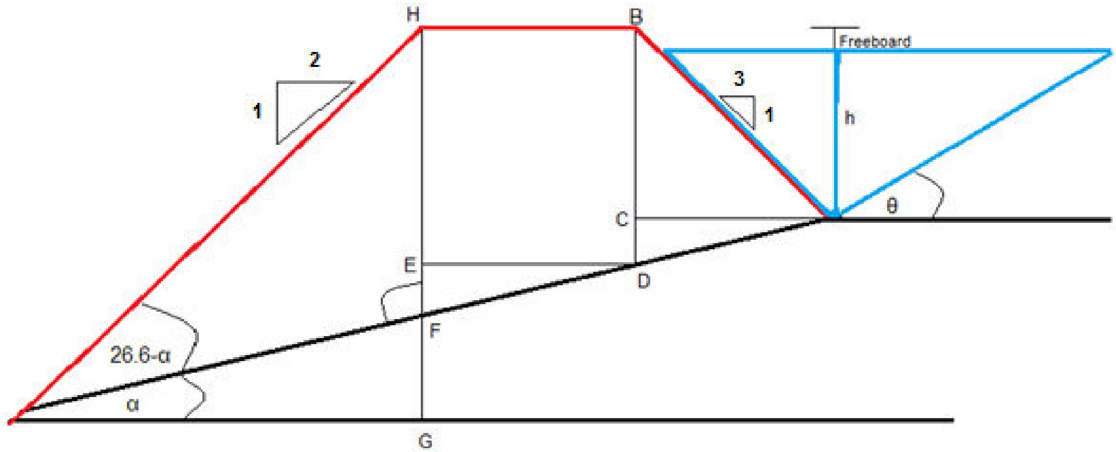


Figure 3.13 Embankment Calculations

3.2.4 Stability of design

For the stability of the dam, the software program GeoStudio (Geostudio 2016 designing software, 2016) was used to analyze the various conditions for slope stability. Material TG2-3 was selected for the core material and the dam wall water was selected for the embankment material.

3.2.4.1 Determining the design parameters

Table 3.4 Design parameters used

	Φ	c
Embankment material	35	8
Core Material	28	10
Slope/Foundation conditions	28	8

For the design tests, the following information contained in Table 3.4 was used in modeling of the materials. The weakest cohesion and frictions angle was assumed for the slope/foundation that the embankment will be situated on.

3.2.4.2 Material Density

For the construction circumstances, the density can be determined from the following equation 3.9 from Craig's Soil Mechanics (2012):

$$p = \frac{G_s x(1 + \omega)}{1 + e} x \, p_w = p_d(1 + \omega) \qquad (3.9)$$

Where: Gs = specific gravity (2.65)

ω = moisture content (as a percentage)

e = void ratio

p_w = water density (1000 kg/m^3)

p_d = dry density (kg/m^3)

For the embankment material (Dam wall), the p_d = 2006 and ω = 10.1 %, from (heading 3.1.6) thus:

$$p = 2006(1 + 0.101) = 2207 \text{ kg}/m^3$$

For the core material (TG2-3), the p_d = 1754 and ω = 15.4 %, from (heading 3.1.6) thus:

$$p = 1754 \,(1 + 0.154) = 2024 \text{ kg}/m^3$$

The highest density is assumed for the slope/foundation material, which is 2207 kg/m^3.

For the full supply circumstances, the density can be determined from the following equation 3.10 from Craig's Soil Mechanics (2012) , as it assumed that the material is saturated:

$$p_{sat} = \frac{G_s + S_e}{1 + e} x \, p_w \qquad (3.10)$$

Where S_e = degree of saturation (= 1 for fully saturated and 0 for dry soil)

For the Embankment material (Dam wall), the p_d = 2006 and ω = 10.1 %, from (heading 3.1.6) thus:

$$p_d = \frac{G_s}{1 + e} x \, p_w$$

$$2006 = \frac{2.65}{1+e} x\, 1000$$

e = 0.32

Using e =0.32

$$p_{sat} = \frac{G_s + S_e}{1+e}\ x\ p_w$$

$$p_{sat} = \frac{2.65 + 0.32}{1+0.32}\ x\ 1000$$

P_{sat} =2250 kg/m^3

For the core material (Dam wall), the p_d = 1754 and ω = 15.4 %, from (heading 3.1.6) thus:

$$p_d = \frac{G_s}{1+e}\ x\, p_w$$

$$1754 = \frac{2.65}{1+e}\ x\ 1000$$

e = 0.51

Using e =0.51

$$p_{sat} = \frac{G_s + S_e}{1+e}\ x\ p_w$$

$$p_{sat} = \frac{2.65 + 0.51}{1+0.51}\ x\ 1000$$

P_{sat} = 2093 kg/m^3

The highest density is assumed for the slope/foundation material, which is 2250 kg/m^3.

3.2.4.3 Pore pressure relationship

For the rapid drawdown situation, the value of r_u was determined using the following equation 3.11:

$$r_u = \frac{p_w}{p_{sat}} x\,(1 + \frac{h_w}{h} x\,(1 + B) - \frac{h'}{h} \qquad (3.11)$$

A conservative approach was taken when calculating r_u as B = 1 and h' = 0, thus the equation, was altered as follows:

$$r_{u} = \frac{p_w}{p_{sat}}$$

For p_w = 1000 kg/m^3 and p_{sat} = 2250 kg/m^3, thus

$$r_u = 0.44$$

3.2.4.4 Stability of the dam

The dimensions for the maximum cross section area, is described by 30 vertical slices

Downstream slope stability

For this slope's stability, the following situations where modelled:

- After construction
- Full supply

For both conditions, a minimum factor of safety of 1.3 and 1.5 is required respectively. For the analysis, it was found that the design has a 1.73 and 1.55 Factor of safety, respectively for the analysis. Please see Appendix G for the results to the analysis.

Upstream slope stability

For this slope's stability, the following situations where modelled:

- After construction
- Rapid drawdown

For both conditions, a minimum factor of safety of 1.3 and 1.2 is required respectively. For the analysis, it was found that the design has a 2.9 and 2.1 Factor of safety, respectively for the analysis. Please see Appendix G for the results to the analysis.

Stability of the cutting

For this slope's stability, the following situations where modelled:

- After construction
- Rapid drawdown

For both conditions, a minimum factor of safety of 1.3 and 1.2 is required respectively. For the analysis, it was found that the design has a 2.0 and 1.2 Factor of safety, respectively for the analysis. Please see Appendix G for the results to the analysis.

3.2.5 Seepage

A seepage analysis was also conducted with the software GeoStudio and the seepage for the embankment can be seen in Appendix G. Two results were analyzed, one design with a filter and one design without a filter.

3.2.5.1 Particle size of filter

Out of all our soil samples, TG2-2, was investigated to be used as a suitable filter material. According to Craig's Soil mechanics (2012), to avoid piping from happening, the material has to adhere to the following equation 3.12 from Craig's Soil Mechanics (2012):

$$\frac{(D_{15})f}{(D_{85})s} < 4 - 5 \qquad (3.12)$$

Where f = filter material

s = boundary material

With reference to Figure 3.6 on grain sizes:

$(D_{15})f = 0.09$

And

$(D_{85})s$ = 1 (of Dam wall material)

$$\frac{(D15)f}{(D85)s} = 0.09 < 4 - 5$$

Material adheres to property!

Another property that the filter has to adhere to is that the permeability of the material has to provide effective drainage, which can be calculated from equation 3.13 from Craig's Soil Mechanics (2012):

$$\frac{(D15)f}{(D15)s} > 5 \qquad (3.13)$$

With reference to Figure 3.6 on grain sizes:

$(D_{15})f$ = 0.09

And

$(D_{15})s$ = 0.015 (of Dam wall material)

$$\frac{(D15)f}{(D15)s} = 6 > 5$$

Material adheres to property!

3.2.5.2 Thickness of filter

For determining the thickness of the filter, the size of the filter was altered in Geostudio and was varied. The length of the filter was decided to be 40 m and the thickness to be 8 m. See appendix G for results.

3.2.6 Spillway design

The most appropriate position for the placement of the spillway is on the right hand side of the embankment, if you are looking up from the downstream side. This choice was made, that even though both sides of provide adequate foundations for a spillway, an access road is needed on the left hand side of the downstream side. By placing the spillway on the right, water can be diverted and linked with the existing overflow spillway of the dam below. A concrete base can be founded to prevent erosion at the edge of the spillway or at a part of the embankment wall where the spillway will be situated.

A simplistic ogee spillway was selected and the length of the spillway was determined using the following equation 3.14 from Dams and Appurtenant Hydraulic Structures (2005):

$$Q = \frac{2}{3}\, x\ \mu\, x\ \sqrt{2g}\, x\, L\, x\, h^{\frac{3}{2}} \qquad (3.14)$$

Where Q = 1 in 100 year designed flood

μ = 0.75

g = 9.81 m/s^2

L = length of spillway (m)

H = height above water level to top of dam (freeboard) (m)

So using the 3m we used for a freeboard in the design of the dam, we get the following length for our spillway:

$$274 = \frac{2}{3}\, x\ 0.75\, x\, \sqrt{2(9.81)}\, x\, L\, x\, 3^{\frac{3}{2}}$$

$$L = 23.8\, m$$

This is an acceptable length for the spillway, as a section of the spillway length opening can be placed on the edge of the embankment, with a section that can be founded in the bedrock, adjacent to the embankment.

3.2.7 Pipe design

An irrigation pipe will be placed below the slope of the embankment, where the dam's water will be the deepest. An adequate ditch has to be excavated beforehand, to place the pipe inside and will be covered with TG2-3 material to limited water seepage. The pipe will have a float attached to a rubber part at the one end (inside the body of water), to allow the pipe to float and stay above any settling soil particles that can clog the pipe. Below the downstream side a tap with adequate water housing, will provide the joining place for the water to flow off in the irrigation pipes.

The size of this pipe to go under the embankment was determined using the Orifice Formula (Pipe flow calculations-flow in pipe) as follows, using equation 3.15 and 3.16:

$$\mathrm{Q} = \mathrm{KA}\sqrt{(2\mathrm{gH})} \qquad (3.15)$$

And

$$K = \left(1 + \lambda\,\frac{L}{d} + \sum \zeta\right)^{-\frac{1}{2}} \qquad 3.16$$

With Q = flow in pipe m^3/s

K = Coefficient of discharge

A = cross sectional area of pipe (m^2)

g = 9.81 m/s^2

H = Height of water (m)

λ = friction coefficient

L = Length of pipe (m)

d = diameter of pipe (m)

∑ζ = sum of secondary loss

Assuming the following:

= 0,015 friction coefficient for PVC pipe

= 0.3 Secondary loss in pipe system

The length of the pipe was determined from the maximum length of the embankment. This was determined to be 120 m. An assumption is made, that the water level reaches 1 m at the end of the season. A 200mm pipe is chosen and the Q is calculated as follows:

$$K = \left(1 + (0.015 * \frac{120}{0.200}) + 0.3\right)^{-\frac{1}{2}}$$

K = 0.31

$$Q = 0.31 * \frac{0.200^2}{4} * \pi * \sqrt{(2 * 9.81 * 1)}$$

Q = 0.04 m^3/s

Q_{needed} = 0.005 m^3/s (from flow calculations 3.2.5)

Thus the pipe diameter selected is adequate for the planned dam.

3.2.8 Protection of slope

For the downstream slope protection, we will use a dual system of both a geosynthetic and grass, to protect the downstream slope from erosion. A geonet, similar to the Kaytech product Soil Saver (Kaytech, 2015), can be used in conjunction with a suitable grass like kikuyu. The geosynthetic, due to the material that it is made from, is able to absorb water and thus reduce the flow over the slope. The material can also provide moisture to the plants, as the water starts to move out of the material, as the material dries. This decrease in surface flow will result in the plant materials being able to establish on the slope and lead to eventual self-stabilizing performance on the slope.

For the upstream slope protection, a geotextile will be installed under rip rap, to prevent erosion. Some specifics for the installed geotextile will be based on an example from Geofabrics (Geofabrics). The permeability of the Dam material is 4 $\times 10^{-6}$ m/s, thus the geotextile material has to be > 10x permeability of the soil. This gives us a permeability value that must be greater than 4 x 10^{-5} m/s. For the filtration of the geotextile, the O_{90} of the geotextile has to be less than D_{50} of the soil. So the O_{90} of the geotextile has to be less than 0.25mm. A layer of fine gravel material will be placed on top of the geotextile.

3.2.9 Final Design

Below, in Figure 3.14, is a simplistic sketch of the final design

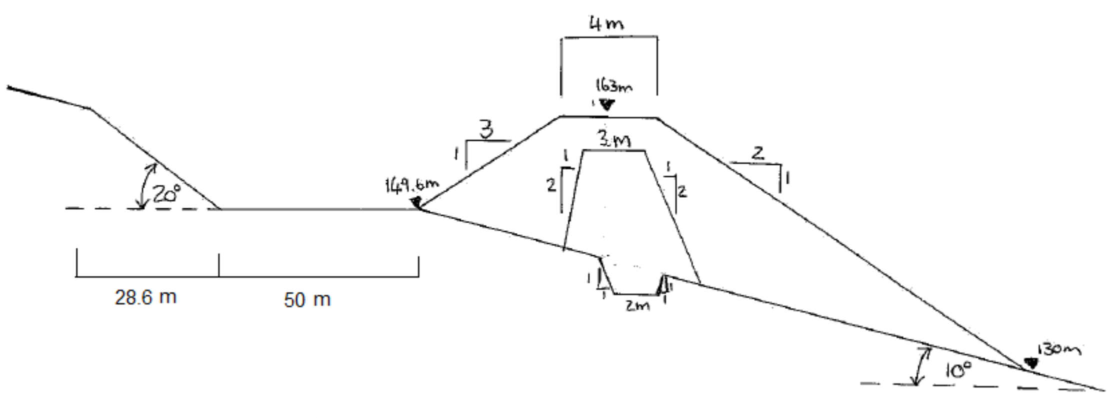


Figure 3.14 Sketch of final design

4. Construction of dam

Although this dam won't be constructed in the near future, it would be important to look at certain aspects of the construction procedure that will take place, when the farm considers implementing the proposed embankment design.

4.1 Site preparation

The site for the embankment should change in terms of preparing it for the construction of the embankment material. Firstly all vegetation should be removed in and around the dam site, as to prevent any roots from entering the embankment, after it has been constructed. With the removal of the vegetation, the removal of unwanted top soil should also be done. This includes the soil from TG1-3 and TG1-4.This soil however, has other uses, as the farmer has added that any soil left over can be added to protecting and maintaining of farm roads. Soil from the TG2-2 and TG2-3 layers, should be kept to one side, as this soil will be used in the embankment. The dam wall material of the old embankment will also be removed and kept to one side, as this material will be reused in the embankment. After this, the slope will be reworked down to the specific slope angle in the design. When the compaction of the material takes place, densities should be measured at various intervals, to make sure that embankment is being compacted at the right densities.

For the installation of the geosynthetics, the Soil Saver product should be placed on a cleared slope, with as much soil-material contact as possible. Product should be unrolled from the top of the embankment and overlap with adjacent one. The overlapping part should be fastened to each other and tied off. A trench to keep the geonet in place should be constructed on the top of the crest. The material should be pegged in a grid formation, with 1m spacing in either horizontal or vertical direction, separating the pegs. Grass should be watered on a daily basis, to make sure the roots grow and stabilize the embankment as soon as possible. For the placement of the geotextile on the upstream slope, care should be taken when placing the gravel on top of the geotextile, as to limit the fall height of the gravel, to prevent puncture.

4.2 Instrumentation

As seen across the whole of chapter 3, various properties of samples were analyzed and determined. As a result, these results were used to design an appropriate embankment dam, which agrees to all the various designing critea of a stable embankment. Thus it makes sense that if there is any change in the properties or expected reactions of the materials, there might be a possible dam failure. Various instrumentation is included in the design, to either see if the design that you have designed acts like you have predicted or to see if there is different reactions to what you have determined. Instrumentation can be used to investigate the following things:

- Deformation
- Pore Pressures and seepage
- Slope Stability
- Seismic Areas

Deformation: Due to their sizes, large dams can "undergo compression" (Hunt, 2005) and as a result lead to foundation settlements. As a result of the settlement, cracks develop across various parts like the crests and even inside the core. The settlement can be measured using extensometers and strain meters inside or outside the embankment.

Pore Pressure and seepage: This information is quite crucial, as a majority of dam stability is done, by using a certain seepage and pore pressure relation. Zones of importance for this measurement, is the toe of the embankment as well as seepage barriers like the core and grout curtains. Instrumentation to measure these influence are piezometers, which are used to measure the pore pressure. Another instrument which is used is acoustic emission devices, which are used to determine the seepage flow path and possible piping areas/zones.

Slope stability: To measure the stability of the designed slope, inclinometers and pressure cells are incorporated in the slope. These instruments measure deflections and stresses respectively.

Seismic areas: Accelographs are used to measure the related pressures and stresses in an embankment, during an earthquake. Piezometers are also used in this measurement, to observe the buildup of pore pressure

5. Conclusion

Embankment design can be an easy task to tackle for seasoned embankment designers, for whom the task of designing an embankment can take a short time, knowing all the factors. However the knowledge that they base their designs on, are based on the interaction of soil and water, as well as the inter soil interaction and characteristics. Thus, the proper understanding of the key geotechnical factors in embankment design is very important. The farm Stone House Estate provided an appropriate site to conduct the research for this thesis. Through the research, this thesis has concluded that geotechnics plays a wide role, across the whole planning, designing and constructions aspects of an embankment dam. The added advances in geosynthetics have made a material available to designers, to incorporate in designs where these materials can solve certain problems.

For our specific clay core embankment, for the farm Stone house Estate, we have constructed a safe and stable design, while adhering to the specified designs that where given for the task. It was determined that a 13.4 meter high earth clay core embankment dam would be constructed on the specific farm site. The water would be stored at a height of 10.4m and the slope gradients would be 1:2 for the downstream slope and 1:3 for the upstream slope. The core would have slope of 2:1 and a cut-off trench with slopes of 1:1. Geosynthetics would be used as upslope protection, for wave erosion. The embankment was able to use mostly on site material to design the embankment, with some geosynthetics having to be brought in. One key note to take away from the dam is to make sure, that levels of the water remain stable for the most part of the year, as a drastic decrease can make the cutting behind the dam close to be too unstable. This means that some method of replacing the water used is needed. One would have to have a pump system from the river to replace used water. Rain catchment is too uncertain as the summer months are becoming drier.

Although this thesis has touched on the influence of geotechnics and geosynthetics in embankment dam design and construction, there are areas of the thesis that can be further expanded for future research. Areas, which can be further investigated, are which material provides the best resource for building earth embankments, material usage vs cost of construction and possibly the effect of climate has on the selection of the type of embankment dam. Another section which can be focused on, is looking at the geotechnical factors and application of geosynthetics in some of the other types of dams like arch or gravity dams.

References

(n.d.). Retrieved January 23, 2017, from Google Maps: https://www.google.co.za/maps/@-34.3805394,19.630825,2791m/data=!3m1!1e3?hl=en

(n.d.). Retrieved January 23, 2017, from Pipe flow calculations-flow in pipe: http://www.pipeflowcalculations.com/pipe-valve-fitting-flow/flow-in-pipes.php

Department of Water and sanitiation- Republic of South-Africa. (2008, September 03). *Department of Water Affairs*. Retrieved September 13, 2016, from Department of Water Affairs- Provincial Rainfall: https://www.dwa.gov.za/hydrology/Provincial%20Rain/Default.aspx

Telford, T. (Ed.). (1990). The Embankment dam. *Proceedings of the Sixth Conference of the British Dam Society*, (p. 186). Nottingham.

(2004). *General Design and Construction Considerations for Earth and Rock-Fill Dams.* U.S.A Army corps of Engineers. Washigton: Department of the army.

(2008). *Geotextiles in Embankment Dams-Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation.* Federal Emergency Management Agency.

Geostudio 2016 designing software. (2016, September 8).

aboutcivil.org. (n.d.). Retrieved September 22, 2015, from spillways-defintion,types and classification: http://www.aboutcivil.org/spillway-definition-types-classification.html

Affairs, D. o. (2015, March 23). *DWS Documents :Department of Water Affairs and Sanitation,Republic of South Africa.* Retrieved November 7, 2016, from Department of Water Affairs and Sanitation,Republic of South Africa: https://www.dwa.gov.za/Documents/default.aspx?type=policy

Anderson, H., & Robert, D. P. (n.d.). *Borehole water association of southern africa.* Retrieved September 22, 2015, from Borehole water association of southern africa: http://www.bwa.co.za/Articles/DAMS%20IN%20SOUTH%20AFRICA.pdf

Artières, O., Oberreiter, K., & Aschauer, F. (n.d.). Geosynthetic systems for earth dams – 35 years of experience. *Long term behaviour of dams*, 98-103.

Brink, A. (1981). *Engineering Gelogy of Southern Africa Volume 2 : Rocks of 2000 to 300 million years in age.* Pretoria: Building Publications.

Brink, A., & Bruin, R. (1990). Guidelines for Soil and Rock logging in South Africa,2nd Impression 2002. *Proceedings, Geoterminology Workshop organised by AEG, SAICE and SAIEG* (pp. 1-45). AEG, SAICE and SAIEG.

Brink, A., & Bruin, R. (2001). Guidelines for soil and rock logging in South Africa,2nd Impression. *Proceedings, Geoterminology Workshop.* AEG, SAICE and SAIEG.

Cambridge Dictionary. (n.d.). Retrieved September 27, 2016, from Cambridge Dictionary: http://dictionary.cambridge.org/dictionary/english/inundate

Chadwick, A., Morfett, J., & Borthwick, M. (2013). *Hydraulics in civil and environmental 5th Edition.* Boca Raton: Talyor & Francis Group.

Council for Geoscience. (n.d.). Retrieved October 29, 2016, from Council for Geoscience-Applied geoscience solutions: http://www.geoscience.org.za/

Department of Water and Sanitation of the Republic of South Africa. (n.d.). Retrieved March 9, 2016, from https://www.dwa.gov.za/default.aspx

Djik, M. v., Vuuren, S. v., & Smithers, J. (2013). *Drainage Manual,6th Edition.* Pretoria: The South African national roads agency.

Fell, R., MacGregor, P., & Stapledon, D. (1992). *Geotechnical engineering of embankment dams.* Rotterdam: A.A.Balkema.

Fell, R., MacGregor, P., Stapeldon, D., & Bell, G. (2005). *Geotechnical engineering of dams.* Leiden: A.A Balkema Publishers.

Fourie, P. H. (2010). *Provenance and Paleotectonic Setting of the Devonian Bokkeveld Group, Cape Supergroup, South Africa.* Johannesburg: University of Johannesburg.

Geofabrics. (n.d.). *Designing rock and rip-rap with geotextiles.* Retrieved November 6, 2016, from Geofabrics: file:///C:/Users/16191935/Downloads/Designing%20rock%20&%20riprap%20structures%20with%20geotextiles.pdf

Geology IN:The Mohs scale of mineral Hardness. (n.d.). Retrieved 10 29, 2016, from Geology IN: http://www.geologyin.com/2014/05/the-mohs-scale-of-mineral-hardness.html

Getschow, G. (2015, December 12). *The dam called troube*. Retrieved January 23, 2017, from The Dallas Morning News: http://interactives.dallasnews.com/2015/lewisville-dam/

Hagen, D. (2015). *Embankment Design and Construction Course.* Geotechnical Engineering. Cape Town: Aurecon.

Hassellf, A. (Ed.). (n.d.). *The South China Karst Landscapes*. Retrieved January 24, 2017, from Travelling tour guide: http://weloveyatours.blogspot.co.za/2012/12/the-south-china-karst-landscapes.html

Hunt, R. (2005). *Geotechnical Engineering Investigation Handbook,2nd Edtion.* Taylor and Francis group.

Karst-What is a karst made of (2016). (n.d.). Retrieved September 27, 2016, from Caving Canada: http://www.cancaver.ca/docs/karst.htm

Kaytech. (2015, January). *Kaytech-Soil Saver.* Retrieved November 6, 2016, from Kaytech: https://www.specifile.co.za/brochure/2233-soilsaverandreg/

Knappet, J., & Craig, R. (2012). *Craig's soil mechanics- eighth edition.* New York: Spon Press.

Knappet, J., & Craig, R. (2012). *Craig's soil mechanics,8th edition.* Abingdon: Spon Press.

Koerner, R. M. (1984). *Construction and geotechnical methods in foundation engineering.* New York: McGraw-Hill Book company .

Koerner, R. M. (1986). *Designing with geosynthetics.* New Jersey: Prentice-Hall.

Koffler, A., Choura, M., Bendriss, A., & Zengerink, E. (2008). Geosynthetics in protection against erosion for river and coastal banks and marine and hydraulic construction. *Coast Conserv*, 11-17.

Koseki, J. (2012). Use of geosynthetics to improve seismic performance of earth structures. *Geotextiles and Geomembranes*, 51-68.

Kruger, E. (2013). Drainage Manual,6th edition. In M. v. Djik, *Drainage Manual,6th edition* (pp. 3.1-3.69). Pretoria: The South African National Roads Agency SOC Ltd,c2013.

Maharaj. (2011). The Use of the Crumb Test as a Preliminary Indicator of Dispersive Soils . *Proceedings of the 15th African Regional Conference on Soil Mechanics and Geotechnical Engineering* (pp. 299-306). IOS Press.

Mayne, P. W., Christopher, B. R., & DeJong, J. (2002). *Subsurface investigations-Geotechnical site characterization.* National highway institute.

McCarthy, T., & Rubidge, B. (2005). *The Story of Earth and Life: A southern african perspectiveon a 4.6 billion-year journey.* Cape Town: Struik Nature.

Meso. (2011, March 17). *Storm Chasing South Africa-1969 Tulbagh And Ceres Earthquake*. Retrieved September 27, 2016, from Storm Chasing South Africa: http://www.stormchasing.co.za/tulbagh-ceres-1969-earthquake/

Narita, D. E. (2000). *Design and contruction of embankment dams.* Aichi: Department of civil engineering,Aichi Institute of Technology.

Nichols, G. (2009). *Sedimentology and Stratigraphy:Second Edition.* West Sussex: Wiley-Blackwell.

Pinterest-Technical Drawing for Geology. (n.d.). Retrieved January 23, 2017, from Pinterest: https://za.pinterest.com/carstoncurd/technical-drawing-for-geology/

Rafferty, J. (Ed.). (n.d.). *Encyclopedia Britannica - Soil Liquefaction*. Retrieved September 27, 2016, from Encyclopedia Britannica: https://global.britannica.com/science/soil-liquefaction

Raju, N. (2010). Case Studies on the Usage of Geosynthetics in Earthen Dams and Embankments. *Indian Geotechnical Conference – 2010, GEOtrendz* (pp. 149-154). Mumbai: IGS Mumbai Chapter & IIT Bombay.

Scuero, A., & Vaschetti, G. (2004). Watertightness and safety of dams using geomembranes . *Long-term benefits and performance of dams*.

Sentürk, F. (1994). *Hydraulics of Dams and reservoirs.* Colorado,USA: Water Resources Publications.

Sharma, R. .., & Kumar, A. (2013, April 29). *Case histories of earthen dam failures.* Retrieved January 24, 2017, from http://scholarsmine.mst.edu/icchge/7icchge/session03/8

Solanki, P. (2014, March). *Hydropedia Pratik Solanki's blog-water, the hub of life; Spillways*. Retrieved October 29, 2016, from Hydropedia Pratik Solanki's blog-water, the hub of life: http://hydropedia.blogspot.co.za/2014/03/spillway.html

South African Weather Service- Historical rain maps. (n.d.). Retrieved September 13, 2016, from South African Weather Service: http://www.weathersa.co.za/climate/historical-rain-maps

Sowers, G. F. (1962). *Earth and rockfill dam engineering.* Bombay: Asia Publishing House.

Stateler, J. (2015). Dam Breach Modeling and Consequence Assessment-Overview of Failure Modes. *National Dam Safety Program Technical Seminar No. 22* (pp. 1-59). U.S. Department of the interior- Bureu of Reclamation.

Stephens, T. (2010). *Manual on small earth dams.* Rome: FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS.

Tahmiscioglu, M., Anul, N., Ekmekci, F., & Durmus, N. (2007). *Postive and Negative impacts of dams on the environment.* Ankara.

Tahmiscioglu, M., Anul, N., Ekmekci, F., & Durmus, N. (n.d.). *Postive and Negative impacts of dams on the environment.* Ankara.

Tancev, L. (2005). *Dams and appurtenant hydraulic structures.* London: A.A Balkema publishers.

Thomas, J. (2010). Geosynthetic solutions for geotechnical problems associated with. *Joint Conference on Advances in Bridge Engineering-II* (pp. 298-410). Dhaka, Bangladesh: IABSE-JSCE.

United States society of dams. (n.d.). *USSD - type of embankments*. Retrieved January 24, 2017, from USSD: http://www.ussdams.org/dam-levee-education/overview/types-of-dams/

weatherbase-Caledon,South Africa. (n.d.). Retrieved October 18, 2016, from weatherbase: http://www.weatherbase.com/weather/weather.php3?s=604891&cityname=Caledon-South-Africa

Zornberg, J. (2005). *Advances on the use of geosynthetics in hydraulic systems.* Austin, Texas, USA: University of Texas at Austin.

Zornberg, J. (2013). Recent Examples of Innovation in Projects using Geosynthetics. *Congreso Geosintec Iberia 2013*, 37-50.

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

REGISTRATION/LICENSING PART 1 INDIVIDUAL

1. GENERAL INFORMATION

Mark the applicable option(s) with an *X* and/or complete details where applicable/available.

Indicate the nature of this application:

☐ New registration ☐ Minor change
☐ Formal amendment
Registration Number

2. PARTICULARS OF THE APPLICANT

2.1 **Surname**

Title

Gender ☐ Male ☐ Female

Population Group ☐ Asian ☐ Black ☐ Coloured ☐ White

ID Number

2.2 **Passport Number** (if not a holder of an South African ID)

Expiry Date (ccyy/mm/dd)

Country Of Issue

2.3 **VAT Registration Number**

2.4 **Postal Address**

Postal Code

2.5 **Street Address**
(Only if diffferent from postal address)

Postal Code

2.6 **Contact Telephone Number During Office Hours**

Area/cell code Number Ext

Alternative Contact Number

Area/cell code Number Ext

2.7 **E-mail**

3. CONTACT PERSON DETAILS

3.1 **Title** ______

3.2 **Name** ______

3.3 **Surname** ______

3.4 **Telephone**

Area/cell code | | | | | | | | Number | | | | | | | | Ext | | | |

3.5 **Cell Phone Number**

Area/cell code | | | | | | | | Number | | | | | | | |

3.6 **Fax**

Area/cell code | | | | | | | | Number | | | | | | | | Ext | | | |

3.7 **E-mail** ______

3.8 **Preferred Form Of Communication** ______

Declaration by applicant *(or person who was granted power of attorney by the applicant)*

Surname of delegated person:

Title:

Initials:

ID number:

Passport number:
(if not a holder of South African ID)

Expiry date (ccyy/mmdd):

Delete the words that are not applicable I/we ____ (FULL NAME(S)) hereby declare that the information provided by me/us in this application form is, to the best of my/our knowledge, true and correct.

Signature

Thumb print

Contact number during office hours

Designation of signatory

Date (ccyy/mm/dd)

It is a criminal offence to provide information that is false or misleading.

4. LIST OF PART 2 DOCUMENTS (WATER USE RELATED FORMS)

Mark with an X which of the following documents have been submitted with this application

- ☐ DW760 NWA-Section 21(a)
- ☐ DW761 NWA-Section 21(b)
- ☐ DW762 NWA-Section 21(b)
- ☐ DW763 NWA-Section 21(c)
- ☐ DW764 NWA-Section 21(d)
- ☐ DW765 NWA-Section 21(e)
- ☐ DW766 NWA-Section 21(f)
- ☐ DW767 NWA-Section 21(g)
- ☐ DW768 NWA-Section 21(i)
- ☐ DW780 NWA-Section 21(h)
- ☐ DW805 NWA-Section 21(j)
- ☐ DW806 NWA-Section 21(k)
- ☐ DW901 Property or properties where water use occurs
- ☐ DW902 Details of property owner
- ☐ DW903 Actual/Monitored waste discharge details NWA-Section 21(f/h)
- ☐ DW904 Actual/Monitored waste discharge details NWA-Section 21(e/g)

5. THIS SECTION IS RESERVED FOR OFFICE USE ONLY

5.1	**Billing information**		
5.1.1	WMA for billing*		
	* Water Management Area Codes		
	01 Limpopo	05 Vaal	09 Berg-Olifants
	02 Olifants	06 Orange	
	03 Inkomati-Usuthu	07 Mzimvubu-Tsitsikamma	
	04 Pongola-Umzimkulu	08 Breede-Gouritz	
5.1.2	District Municipal Establishment Levy Payable	☐ Yes ☐ No	
5.2	**Mark with an X which of the following documents have been submitted with this application**		
	☐ Certified copy of South African identity document		
	☐ Certified copy of passport		

File number *(i.e. Office Hardcopy Register File No)*

Water Use Register Number

Received by:

Surname

Initials

Position / Rank

Signature

Date (ccyymmdd)

Captured on NRWU database

Captured by:

Surname

Initials

Signature

Date stamp of receiving office

Quality Assurance Executed by:

Surname

Initials

Position / Rank

Signature

)

Date (ccyymmdd)

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

Registration / Licensing **Section 21(b) of the National Water Act**

Part 2 **STORING WATER – DAM REGISTRATION**

SPECIAL NOTE

This application form may be used for registering any of the following:

2. *A dam which can store more than 50 000 cubic meters and has a dam wall which is more than 5 meters high;*
3. *A dam belonging to a category of dams declared under section 118(2) of the NWA to be dams with a safety risk; or declared under section 118(3)(a) of the NWA to be a dam with a safety risk;*
4. *A dam which is deemed to be significant by the applicant or the relevant office.*

Registration of:
(mark only one block with an X)

☐ New Dam
☐ Existing dam registered on NRWU

3. WATER USE DETAILS

2.3 Have you already registered a water use with the Department of Water Affairs and Forestry?

☐ Yes ☐ No

Registration Number:

Water Use Number:

☐ Licence Related WU

RLA Reference

NRWU Licence Number

								/		

RLA Business Unit

(NRWU = National Register of Water Use; RLA = Responsible Licensing Authority; WU = Water Use)

For office use only

Allocated Reg. No.

WU No.

1.2 **Applicant Type** (mark only one block with X)

- ☐ Individual (complete 1.3)
- ☐ Company, business, partnership or community (complete 1.4)
- ☐ National Department (complete 1.5)
- ☐ Provincial Department (complete 1.6)
- ☐ Water Services Provider (complete 1.7)
- ☐ Water User Association (complete 1.8)

2.4 **If the applicant is an individual**

1.3.1 Title ______ Surname ______ Initials ______

1.3.2 **South African ID (if holder of South African Id) alternatively Passport Number:**

ID Number or Passport Number ______

Passport Expiry Date (ccyymmdd) ______

Passport Country Of Issue ______

2.6 **If the applicant is a company, business, partnership or community:**

2.7 Name of company, business, partnership or community:

______

4. Business Enterprise Registration Number ______ / ______

5. Date Established (ccyymmdd) ______

Country Where Established ______

3.5 **If the applicant is a National Department:**

National Department Name: ______

3.6 **If the applicant is a Provincial Department:**

Province: ______

Provincial Department Name: ______

3.7 **If the applicant is a Water Services Provider:**

Name of WSP: ______

3.8 **If the applicant is a Water User Association:**

Name of WUA: ______

Declaration by applicant

Delete the words that are not applicable *I/we ________________________________ (FULL NAME(S)) hereby declare that the information provided by me/us in this application form is, to the best of my/our knowledge, true and correct.*

Signature

Thumb print

Contact number during office hours

Designation of signatory

Date (ccyy/mm/dd)

It is a criminal offence to provide information that is false or misleading.

5. SUCCESSION/TRANSFER AND SOURCE PART 2 DETAILS

2.1 Is this a Succession or a Transfer related Water Use? (Mark only one box with an X)

☐ Yes

☐ No

2.2 If yes, mark with an X the Succession / Transfer Type

☐ Full Temporary Transfer ☐ Partial Temporary Transfer

☐ Permanent Transfer ☐ Succession in Title

6. Source Register Number WU Number

Source Register Number WU Number

Source Register Number WU Number

3. GENERAL DAM INFORMATION

3.1 Name of the dam

3.2 If the water is to be stored in a watercourse, then enter the name of the watercourse

3.3 For off-stream storage, enter the name of the watercourse to which the water would naturally drain

3.4 Surname and initials or business name of designer or consultant Initials

3.5 Surname and initials or business name of contractor — Initials

4. PURPOSE OF DAM

Identify the purpose that the dam is used for:

- ☐ Agriculture: Irrigation (DW787)
- ☐ Agriculture: Watering Livestock
- ☐ Agriculture: Aquaculture
- ☐ Industry
- ☐ Mining (DW788)
- ☐ Recreation
- ☐ Schedule 1
- ☐ Water Supply Service (DW789)

5. DAM SIZE AND BASIN INFORMATION

5.1 Date of completion of the dam (ccyymmdd)

5.2 Size of dam

a) Maximum wall height ** — metres

*** "wall height" is the vertical difference between the lowest downstream ground elevation on the dam wall and the non-overspill crest level or the general top level of the dam wall*

b) Crest length of wall *** — metres

**** The length of the crest includes the length of the spillway, where applicable.*

c) Gross storage capacity — thousand cubic metres

d) Water surface area at full supply level — hectares

5.3 Water depth at full supply level — metres

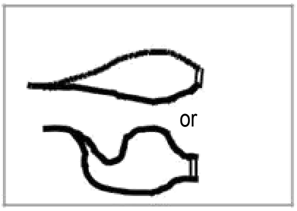
5.4 For off-stream storage, select the most appropriate dam basin shape below

☐ Triangular ▲ ☐ Rectangular ■ ☐ Circular ● ☐ Branched Y

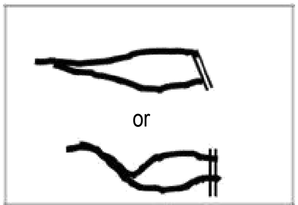
5.5 For in-stream storage, select the shape below that is most similar to the dam basin

(in these diagrams, flow is from left to right and the || symbol shows the position of the dam wall)

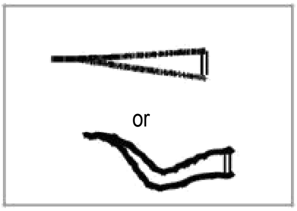
☐ Bulbous



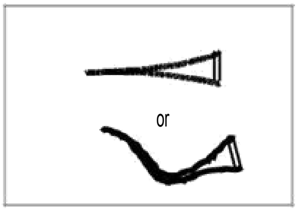
☐ Carrot



☐ Triangle



☐ Funnel



5.6 Dam basin dimensions

a) Length (or diameter if round) — metres (for in-stream storage, measure along the centre-line)

b) Width (leave blank if round) — metres (for in-stream storage, measure at the widest point)

6. CONTACT DETAILS OF PERSON IN CONTROL OF THE DAM

6.1 Surname First Name Title

6.2 Phone Number Ext

Fax Number

6.3 Cellphone Number

6.4 Email Address

7. CLASSIFICATION INFORMATION

7.1 Has the dam been classified? ☐ Yes ☐ No (if no, complete form DW793: Dam Classification)

7.2 If the dam has been classified, then complete the following

Date of classification of the dam (ccyymmdd)

Category classification (mark only one block with an X)	*I*	*II*	*III*
Size class (mark only one block with an X)	Small	Medium	Large
Hazard potential rating (mark only one block with X)	Low	Significant	High

8. DAM STRUCTURE

8.1 Type of dam (mark applicable type with an X – mark more than one for composite dams)

☐ Arch ☐ Earth reservoir ☐ Multi-arch

☐ Buttress ☐ Gravity ☐ Reinforced concrete reservoir

☐ Other (specify)

9. SPILLWAY INFORMATION

9.1 Information about the spillway

a) Type of spillway (mark applicable type with an X – mark more than one if necessary)

☐ By-wash ☐ Drop inlet ☐ Shaft

☐ Cascade ☐ Free fall (straight drop) ☐ Side channel

☐ Chute (baffled, etc.) ☐ Labyrinth ☐ Siphon

☐ Chute (lined) ☐ Morning glory ☐ Stepped

☐ Conduit ☐ Ogee (overflow) ☐ Other (describe)

☐ Culvert ☐ Open channel

b) Crest length of spillway [| | | . | |] metres

c) Description of spillway gates, if any

d) Details on any auxillary or second spillway

Location ("left bank", "saddle", etc.)

Nature or type of spillway

Crest length of auxillary spilway [| | | . | |] metres

9.2 Does the dam structure incorporate a fish ladder or fish way? ☐ Yes ☐ No

10. LOCATION OF DAM

10.1 Nearest city or town

10.2 Distance from nearest city or town [| | |] km

10.3 Direction to dam from nearest city or town ☐ ↑ ☐ ↗ ☐ → ☐ ↘ ☐ ↓ ☐ ↙ ☐ ← ☐ ↖

10.4 Number of 1:50 000 scale topographic map (or 1:10 000) [| | |] [|] ([|])

10.5 Geographic position of center of dam wall (in one format only)

Latitude S ° ' . " or S . ° or S ° . '

Longitude E ° ' . " or E . ° or E ° . '

Datum Type: ☐ Cape (Modified Clarke 1880) ☐ WGS-84

10.6 Geographic position of center of river at the point where the river crosses the dam wall (in one format only)

Latitude S ° ' . " or S . ° or S ° . '

Longitude E ° ' . " or E . ° or E ° . '

Datum Type: ☐ Cape (Modified Clarke 1880) ☐ WGS-84

10.7 Quaternary Drainage Region [| | |]

11. WUA or WSP DETAILS

11.1 Is the dam controlled by a Water Use Association or Water Services Provider? ☐ WUA ☐ WSP

11.2 Name of Water User Association or Water Services Provider

12. EXISTING AUTHORISATION

12.1 Existing permit information

	Permit number	**Date** (ccyymmdd)
Permit No.		
Permit No.		
Permit No.		
Permit No.		
Permit No.		
Permit No.		

12.2 If water use takes place in terms of the General Authorisation, mark with an X ☐

*If yes complete the following details after confirmation with relevant DWAF/CMA officials:

Date(s) from which applicable GA is/was applicable to this water use

South African Act: ____________________ Applicable section of the act ____________

[E.g. National Water Act (Act No. 36 of 1998)] [E.g. Section 21]

Date From (ccyymmdd) | Government Notice No.

Date To (ccyymmdd) | Government Notice Date (ccyymmdd)

Applicable Section Of The General Authorisation ____________________

Date From (ccyymmdd) | Government Notice No.

Date To (ccyymmdd) | Government Notice Date (ccyymmdd)

Applicable Section Of The General Authorisation ____________________

Date From (ccyymmdd) | Government Notice No.

Date To (ccyymmdd) | Government Notice Date (ccyymmdd)

Applicable Section Of The General Authorisation ____________________

12.3 If an authorisation has been issued under other legislation

Law /Regulation ____________________

13. PROPERTY RELATIONSHIP DETAILS *(Complete supplementary forms DW901 & DW902)*

Property Name	Surveyed Property		Unsurveyed property		Property Relationship Date From:	To:
	Title Deed Number		Surname of the Leader of Village, Community or Tribal Authority			
	Surveyor-General Cadastral Code		Initial of the Leader of Village, Community or Tribal Authority			
	Property Number		Local Authority (if applicable)			
	Portion of property		Magisterial District (if applicable)			
			Tribal Authority/Council (if applicable)			
	Title Deed Number		Surname of the Leader of Village, Community or Tribal Authority			
	Surveyor-General Cadastral Code		Initial of the Leader of Village, Community or Tribal Authority			
	Property Number		Local Authority (if applicable)			
	Portion of property		Magisterial District (if applicable)			
			Tribal Authority/Council (if applicable)			
	Title Deed Number		Surname of the Leader of Village, Community or Tribal Authority			
	Surveyor-General Cadastral Code		Initial of the Leader of Village, Community or Tribal Authority			
	Property Number		Local Authority (if applicable)			
	Portion of property		Magisterial District (if applicable)			
			Tribal Authority/Council (if applicable)			
	Title Deed Number		Surname of the Leader of Village, Community or Tribal Authority			
	Surveyor-General Cadastral Code		Initial of the Leader of Village, Community or Tribal Authority			
	Property Number		Local Authority (if applicable)			
	Portion of property		Magisterial District (if applicable)			
			Tribal Authority/Council (if applicable)			

14. FOR OFFICE USE ONLY

14.1 Billing information

				Start date (ccyymmdd)	End date (ccyymmdd)
14.1.1	Applicant billed as:	☐ An Individual	☐ Via a WUA/WSP		
14.1.2	Applicant to be charged:	☐ On actual volume	☐ Registered volume		
14.1.3	Billing Frequency:	☐ Annually	☐ Bi-annually	☐ Monthly	

14.1.4 If to be billed via a WUA /WSP

Name of WUA/WSP

Is WUA/WSP a Billing Agent? ☐ Yes ☐ No

Billing Agent Register Number

14.1.5 If this WU is to be billed via Bulk Billing Party that is not a WSP/WUA, complete the following

Name of Customer

Bulk-Bill-to-Party Register Number

14.1.6 Is the Dam billable? ☐ Yes ☐ No

14.1.7 Is this a Safety Risk Dam? ☐ Yes ☐ No

14.1.8 Annual Average Evaporative Loss Start Date (ccyymmdd) Volume

14.2 District Municipality

District Municipality Name (if applicable)

14.3 Late Registration Penalty

Is this a late registration? ☐ Yes ☐ No

If yes, mark with an X, the applicable penalty to be levied

☐ R300.00 **OR**

☐ 10% (ten percent) of the annual water use charge outstanding at the date of registration which ever is greater

Specify the penalty amount payable

☐ Waive penalty

File number

Water Use Register Number

Received by:

Surname

Initials

Position / Rank

Signature

Captured on NRWU database (ccyymmdd)

Capured by:

Surname

Initials

Signature

Date stamp of receiving office

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

SUPPLEMENTARY WATER USE INFORMATION

STORING WATER
DAM CLASSIFICATION

1. PARTICULARS OF DOWNSTREAM DEVELOPMENT THAT WOULD BE THREATENED BY DAM FAILURE

Describe, with reference to a 1:50 000 map, the nature and situation of downstream development that would be threatened by failure of the dam. "Development" means any houses, dwellings, other buildings, roads, bridges, cultivated lands, orchards, powerline foundations, etc.

Definition of the downstream area wherein all development must be described:

1. *for every one metre of maximum wall height, analyse at least one kilometre of the valley downstream of the wall*
2. *for the calculation of the width of the strip, assume the following heights above the river bed:*
2/3 of maximum wall height for the first kilometre downstream and 1/2 of the maximum wall height for the rest of the downstream distance.

1.1 Buildings such as houses, dwellings and other similar structures:

Distance down-stream (km)	Purpose or use of structure	Height above river bed (metres)	Distance from river (metres)	Number of inhabitants or users

1.2 Road and railway crossings downstream of the dam:

Distance down-stream (km)	Type [1]	Tar road? (X)	Height above river bed (metres)	Bridge, culvert or pipe openings			Crossing type [2]	Visibility distance (m)		Number of vehicles per day
				width and height (metres)	diameter (metres)	how many		first and second approach [3]		

Type — *Use one of the following abbreviations:*
MRD = main road, SRD = secondary road, DRD = district road, FRD = farm road,
STR = single track railway, MTR = multi-track railway. Explain other abbreviations below:
___ = ___ ___ = ___

Crossing — *Use one of the following abbreviations:*
C = culverts or pipes encased in concrete, E = culverts or pipes buried in earthfill or rockfill, B = concrete bridge with piers.
Explain other abbreviations below:
___ = ___ ___ = ___

Visibility — *This is the distance to a bridge or crossing from where a motorist can see if there is any danger in using the bridge or crossing. Both approach distances are required. If the distance equals or exceeds 1 kilometre, enter 999.*

1.3 Other development downstream of the dam, not covered by 1.1 or 1.2:

FOR OFFICIAL USE ONLY

category (mark one with X)	I	II	III
size class (mark one with X)	*small*	*medium*	*large*
hazard potential rating (mark one with X)	*low*	*significant*	*high*

File number

Water use licence or registration number

Water management area

Received by:

Surname *Initials*

Rank

Signature

Date stamp of receiving office

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

SUPPLEMENTARY WATER USE INFORMATION
PROPERTY WHERE WATER USE OCCURS

DW901 serves to address the following: The property (or properties) where water use(s) is to take place.

•Complete one DW901 form for each property impacted / applicable to a water use registration application.

•Should more than one property owner be applicable to a "property where water occurs" an additional DW902 must be completed for each additional property owner.

5. PROPERTY WHERE WATER USE(S) OCCURS

4. **Property where water use takes place (farm, stand or community): description as per the Deeds Act if applicable, or name of agricultural holding, farm, township, town or city.**

Registration Date (ccyymmdd):

2.4 **Property Type** (mark only one with an X)

- ☐ Agricultural Holding
- ☐ Exclusive Use Areas (EUA)
- ☐ Sectional Scheme (To Obtain EUA)
- ☐ Sectional Scheme Unit
- ☐ Unspecified
- ☐ Erf
- ☐ Farm
- ☐ Sectional Scheme (to obtain units)
- ☐ Township
- ☐ Unsurveyed

1.3 **If the property type is unsurveyed, complete the following:**

a) Surname and initials of leader of village, community or tribal authority

Initials

b) Local Authority

&/or

c) Magisterial District

&/or

Tribal Authority/Council

2.7 **If the property type is not equal to unsurveyed, complete the following:**

a) Deeds Office

b) Registration Division

c) Property No (i.e. Farm No./Erf No./Holding Area No./Scheme No.)

d) Portion of Property

e) Title Deed Number

f) Surveyor-General Cadastral Code

1	2	3	4	5
-	-	-	-	

2.8 Refers to the Surveyor's-General Office (T = Pretoria, F = Free State, C = Cape Town & N = Kwazulu-Natal)
2.9 Major Code (Registration Division)
2.10 Minor code
2.11 Property No (i.e. Farm No./Erf No./Holding Area No./Sheme No.)
2.12 Portion Number
Note: *All fields "left padded with 0"*

6. **Property Area Size**

Measure Unit: ☐ Hectares ☐ Square Meters ☐ Acres

3.7 **Ownership of the property** (mark only one with an X)

☐ Property owned by applicant (100% Share value)
☐ Property owned by applicant (Share value less than 100%)
☐ Property leased by applicant
☐ The property is communal land

3.6 PROPERTY OWNER RELATIONSHIP

Individual (Identity Number or Passport Number)	Company, Business, Partnership or Community (Business Enterprise Registration Number)	Property Owner Name	Property Owner Document Number (Owner's Title Deed Reference Number)	Property Owner and Property Relationship Date		Owner Share Value %
				From:	To:	

3.9 DECLARATION BY APPLICANT *(or person that was granted power of attorney by the applicant)*

Full names

Surname

Signature

Date (ccyy/mm/dd)

Thumbprint (only if requested)

3.9 FOR OFFICE USE ONLY

Received by:

Surname

Initials

Position / Rank

Signature

Captured on NRWU database (ccyymmdd)

Captured by:

Surname

Initials

Signature

Date stamp of receiving office

Quality Assurance Executed by:

Surname

Initials

Position / Rank

Signature

Date (ccyymmdd)

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

SUPPLEMENTARY WATER USE INFORMATION

TAKING WATER FROM A WATER RESOURCE
IRRIGATED FIELD AND CROP INFORMATION

TO BE COMPLETED FOR ALL IRRIGATION ABSTRACTION

Field Number	*Area (hectares)*	*Crop*	*Planting date*	*Growing season (days)*	*Rotation factor %*	*Irrigation system code **	*Resource type*	*Resource name (excluding boreholes)*	*Annual mm used (official use)*
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						
	,		M D D						

*Code	Irrigation system	*Code	Irrigation system	*Code	Irrigation system
A	Flood: Furrow	H	Sprinkler: Big gun	O	Micro sprinkler
B	Flood: Border	I	Sprinkler: Side roll	P	Micro spray
C	Flood: Basin	J	Sprinkler: Boom	Q	Drip
D	Sprinkler: Dragline	K	Sprinkler: Travelling gun	R	Subsurface
E	Sprinkler: Quick-coupling	L	Sprinkler: Travelling boom		
F	Sprinkler: Permanent	M	Centre pivot		
G	Sprinkler: Hop-along	N	Linear		

DESCRIPTION OF ANY IRRIGATION SCHEDULING METHODS USED

DESCRIBE ANY OTHER METHODS USED TO ENHANCE IRRIGATION EFFICIENCY

FOR OFFICIAL USE ONLY

File number

Water use number

Received by:

Surname *Initials*

Rank

Signature

Captured by:

Initials

Date stamp of receiving office

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

SUPPLEMENTARY WATER USE INFORMATION

TAKING WATER FROM A WATER RESOURCE
PUMP TECHNICAL DATA

1. PUMP IDENTIFICATION

1.1 Pump number (if more than one, enter a sequence number starting from 001) 0 0 1

1.2 Installation date Y Y Y Y M M D D

1.3 Geographic location of the pump (use one format only)

S ° ' . " *or* S ° *or* S ° '
E 0 ° ' . " E 0 ° E 0 ° '

Cape datum Clarke
WGS-84 datum

2. PUMPING HOURS

2.1 Maximum pumping hours per week h 2.2 Total pumping hours per year h

3. PUMP DATA

3.1 Pump type (mark one with *X*)

a) Centrifugal b) Positive displacement c) Turbine d) Axial flow
e) Other (specify)

3.2 Pump model

3.3 Pulley diameter mm

3.4 Speed rpm

3.5 Impeller size (only for a centrifugal pump) mm

3.6 Suction hose

3.6.1 Hose material

3.6.2 Hose diameter mm

3.6.3 Hose length m

3.7 Type of flow meter (mark one with *X*)

a) Inline b) Bypass c) Doppler effect d) None e) Other (specify below)

3.8 Pressure gauge reading At inlet = m At outlet = m

4. POWER SOURCE DATA

4.1 Power source type (mark one with *X*)

☐ a) Electric ☐ b) Diesel ☐ c) Petrol ☐ d) Tractor ☐ e) Wind

☐ f) Other (specify) ____________

4.2 Model ____________

4.3 Pulley diameter ______ mm

4.4 Speed ______ rpm

4.5 Coupling:

a) Type (mark one with *X*)

☐ V-belt ☐ Flat belt ☐ Gearbox ☐ Direct ☐ Other (specify below)

____

b) For *gearbox coupling* or *direct coupling*, enter the ratio ______ : ______

4.6 Power rating ______ kW

5. PUMP OPERATION

		Maximum pressure	Maximum discharge	Average operation	
5.1	Discharge				litres / second
5.2	Suction height				metres
5.3	Static height				metres
5.4	Working height				metres
5.5	Friction height				metres
5.6	Other losses				metres
5.7	Total head				metres
5.8	Efficiency				%
5.9	Power absorbed				kilowatts
5.10	Ammeter reading				amps

6. BOREHOLE INFORMATION (where applicable)

6.1 a) Borehole number ____________

b) Geographic location of the borehole, if different from pump

S ___ ° ___ ’ ___ . ___ ” *or* S ___ . ______ ° *or* S ___ ° ___ . ______ Cape datum Clarke ☐

E 0 ___ ° ___ ’ ___ . ___ ” E 0 ___ . ______ ° E 0 ___ ° ___ . ______ WGS-84 datum ☐

6.2 Yield of borehole ______ litres / second

6.3 Depth of borehole ______ metres

6.4 Previous authorisation or licensing reference ____________

7. ESKOM TRANSFORMER (where applicable)

7.1 a) ESKOM reference number

b) Geographic location of the transformer, if different from pump

S ° ' . " *or* S ° *or* S ° . ' Cape datum Clarke

E 0 ° ' . " E 0 . ° E 0 ° . ' WGS-84 datum

7.2 Power rating of the transformer kVA

FOR OFFICIAL USE ONLY

File number

Water use licence or registration number

Water Management Area

Received by:

Surname Initials

Rank

Signature

Captured by:

Initials

Date stamp of receiving office

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

DAM SAFETY OFFICE

PRIVATE BAG X313 PRETORIA 0001

APPLICATION FOR CLASSIFICATION OF A PROPOSED NEW DAM OR ENLARGEMENT OR ALTERATION OF AN EXISTING DAM

Only applicable if the maximum wall height of the dam exceeds 5 metres and the gross storage capacity is more than 50 000 cubic metres

1. PARTICULARS OF THE DAM OWNER

1.1. Name of dam owner

1.2. Owner's postal address

Postal code

1.3. Tel/cel. no. of dam owner

1.4. E-mail address of person in control of the dam

1.5 Name and postal address of person in control of the dam (*if applicable*)

Postal code

1.6. Tel/cel. no. of person in control of the dam

1.7. E-mail address of person in control of the dam

2. PROPERTY ON WHICH THE DAM IS OR WILL BE SITUATED AND LOCALITY

2.1. Property description as per title deed

2.2. Magisterial district

2.3. Nearest city/town

2.4. Distance to nearest city or town km

2.5. Direction from nearest city or town

2.6. Number of 1:50 000 scale topographical map *

* ***A copy of the relevant portion of this map which clearly indicates the position of the dam and downstream area must be attached***

2.7. Position of the centre of the dam wall to an accuracy of one second

Latitude: ° ' " Longitude: ° ' "

2.8. Title deed number

3. GENERAL INFORMATION

3.1. Name of dam

3.2. Name of watercourse or source

3.3. For **clean water** dams, give the purpose of the dam (*mark **all** applicable purposes with X*)

domestic supply | irrigation | industrial use

stock watering | fisheries | other (specify below)

Describe "other"

3.4. For **wastewater** dams, give the purpose of the dam (*mark **all** applicable purposes with X*)

pollution control | wastewater disposal | industrial residue

oxidation / evaporation | mine residue | other (specify below)

Describe "other"

3.5. For an existing dam describe the nature and extent of the proposed alterations or enlargements

3.6. Proposed starting date of construction Y Y Y Y M M

3.7. Name and postal address of designer or consultant (*if available*)

Postal code

3.8. Tel. no. of designer or consultant

3.9. E-mail adress of designer or consultant

4. PARTICULARS OF DAM AND BASIN

(*For enlargement or alteration of an existing dam, particulars must be for the completed structure*)

4.1. Type of dam (*mark applicable type with X - mark more than one for composite dams*)

earthfill | rockfill | gravity

buttress | arch | multi-arch

earth "service" reservoir | reinforced concrete "service" reservoir

mine residue deposit * | industrial residue deposit *

***** This also means any structure generally termed a "tailings or slimes dam"***

other (*specify*)

4.2. Maximum wall height ** , m

***** Note! Wall height is the vertical difference between the lowest downstream ground elevation on the outside of the dam wall and the non-overspill crest level or the general top level of the dam wall***

4.3. Crest length of wall m

4.4. Gross storage capacity m^3

4.5. Area of water surface at full supply level , ha

4.6. Maximum full supply water depth (*must be provided*) , m

5. PARTICULARS OF DEVELOPMENT DOWNSTREAM OF THE DAM

Describe with the aid of a 1:50 000 scale map the nature and situation of development downstream of a dam that would be threatened by a failure of the dam. Development means any houses, dwellings, other buildings, roads, bridges, cultivated lands, orchards, powerline foundations etc.

The area downstream of the dam wherein all development must be described is defined as follows;

- For every one metre of maximum wall height, at least one kilometre of the valley downstream of the dam wall should be analysed

- For the calculation of the width of the strip the following heights above river bed may be assumed;

2/3 of maximum wall height for the first kilometre downstream and 1/2 of the maximum wall height for the rest of the downstream distance

5.1. Development downstream of the dam (houses, dwellings and other similar structures)

Distance downstream (km)	Purpose or use of structure	Height above river bed (m)	Distance from river (m)	Number of inhabitants or users

5.2. Road and railway crossings downstream of the dam

Distance downstream (km)	(1) Type of road or railway	If a road, is it tarred? (Y/N)	Height of road / railway above river bed (m)	Bridge, culvert or pipe openings				(2) Type of crossing	(3) Visibility distance (m)	Number of vehicles per day
				Width (mm)	Height (mm)	Diameter (mm)	How many?			
			,						i ii	
			,						i ii	
			,						i ii	
			,						i ii	
			,						i ii	
			,						i ii	

(1) *Type of road or railway - Use one of the following abbreviations*

NRD = national road *MRD = main road* *SRD = secondary road* *DRD = district road*

FRD = farm road *STR = single track railway* *MTR = multi-track railway*

Explain other abbreviations =

(2) *Type of crossing - Use one of the following abbreviations*

C = culverts or pipes encased in concrete *E = culverts or pipes buried in earthfill or rockfill*

B = concrete bridge with piers *D = drift with same height as river bed*

Explain other abbreviations =

(3) *Visibility distance - This is the distance to a bridge or crossing from where a motorist can see if there is any danger in using the bridge or crossing. Both approach distances are required.* ***The order in which i and ii are written does not matter.***

If the distance equals or exceeds 1 kilometre, enter 999

5.3. Other development downstream of the dam, not covered by 5.1 or 5.2

6. DECLARATION BY APPLICANT

I declare that the information given by me for the classification of the above dam is true and correct.

Signature: ____________________________ Date: ________________

NB! Remember to attach a clear copy of the relevant topographical map (see 2.6)

Appendix B – Soil Descriptions (Brink & Bruin, 1990)- Guidelines for Soil and Rock logging in South Africa

MOISTURE CONDITIONS

DRY	Requires addition of water to reach optimum moisture content for compaction
SLIGTHLY MOIST	
MOIST	Near optimum moisture content
VERY MOIST	Requires drying to attain optimum moisture content
WET	Fully saturated and generally below water table

CONSISTENCY

Consistency	Gravels and clean sands Generally free-draining (cohensionless materials)	Typical Dry Density (kg/m^3)	Saturated SPT Blow counts N
Very loose	Crumbles very easily when scraped with geological pick.	<1450	<4
Loose	Small resistance to penetration by sharp end of geological pick.	1451 to 1600	4 – 10
Medium dense	Considerable resistance to penetration by sharp end of geological pick.	1601 to 1750	11 – 30
Dense	Very high resistance to penetration of sharp end of geological pick; requires many blows of pick for excavation.	1751 to 1925	31 – 50
Very dense	High resistance to repeated blows of geological pick; requires power tools for excavation.	>1925	>50

Consistency	Silts and clays and combinations thereof with sand. Generally slow draining (cohesive materials) (Φ= 0)	Unconfined Compressive Strength (kN/m^2)	Saturated SPT Blow counts Sensitive silts and clays	Saturated SPT Blow counts Insensitive silts and clays
Very soft	Pick head can easily be pushed in to the shaft of handle; easily moulded by fingers.	<50	<2	<5
Soft	Easily penetrated by thumb; sharp end of pick can be pushed in 30 – 40 mm; moulded with some pressure.	50 to 125	2 – 4	5 – 10
Firm	Indented by thumb with effort; sharp end of pick can be pushed in up to 10 mm; very difficult to mould with fingers; can just be penetrated with an ordinary hand spade.	126 to 250	5 – 8	11 – 25
Stiff	Penetrated by thumb nail; slight indentation produced by pushing pick point into soil; cannot be moulded by fingers; requires hand pick for excavation.	251 to 500	9 – 15	26 – 50
Very stiff	Indented by thumb nail with difficulty; slight indentation produced by blow of pick point; requires power tools for excavation.	500 to 1000	16 – 20	51 – 80

STRUCTURE

TERM	IDENTIFICATION
INTACT	Structureless, no discontinuities identified.
FISSURED	Soil contains discontinuities which may be open or closed, stained or unstained and of variable origin.
SLICKENSIDED	This term qualifies other terms to describe discontinuity surfaces which are smooth or glossy and possibly striated.
SHATTERED	Very closely to extremely closely spaced continuities resulting in gravel sized soil fragments which are usually stiff to very stiff and difficult to break down.
MICRO-SHATTERED	As above, but sand-sized fragments.
STRATIFIED & LAMINATED & FOLIATED	These and other accepted geological terms may be used to describe sedimentary structures in transported soils and relict structures in residual soils.
PINHOLED	Pinhole-sized voids or pores (up to say 2 mm) which may require a hand lens to identify.
HONEYCOMBED	Similar to pinholed but voids and pores >2 mm; (pore size may be specified in mm).
MATRIX-SUPPORTED	Clasts supported by matrix.
CLAST-SUPPORTED	Clasts touching (matrix may or may not be present).

SOIL TYPE

Grain size (mm)	Classification	Individual particles visible using	Mineralogical composition	Identification Test
<0.002	Clay	Electron microscope	Secondary minerals (clay minerals and Fe-oxides	Feels sticky or soapy. Soils hands. Shiny when wet.
0.002 to 0.06	Silt	Microscope	Primary and secondary minerals	Chalky feel on teeth. When dry rubs off hands. Dilatant*.
0.06 to 0.2	Fine sand	Hand lens	Primary minerals (mainly quartz)	Gritty feel on teeth
0.2 to 0.6	Medium sand	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye
0.6 to 2.0	Coarse sand	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye
2.0 to 6.0	Fine gravel +	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye
6.0 to 20.0	Medium gravel +	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye
20.0 to 60.0	Coarse gravel +	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye
60.0 to 200	Cobbles +	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye
>200	Boulders +	Naked eye	Primary minerals (mainly quartz)	Observed with naked eye

ORIGIN

ORIGIN	AGENCY OF TRANSPORTATION	PROBLEMS TO BE EXPECTED
Littoral and mobile dune sands	Waves, current and tides	Collapsible fabric; instability of dredged marine deposits excavations; high soluble salt content; variable carbonate cementation.
Estuarine and deltaic	Tidal rivers depositing into saline water	Compressibility; variability, sensitivity; quick-sand; high soluble salt content.
Talus (coarse colluvium)	Gravity (mass-wasting processes)	Slope instability.
Silty or clayey hillwash (fine colluvium)	Sheetwash	Expansive characteristics; compressibility; dispersive characteristics.
Aeolian deposits	Wind	Collapsible fabric; mobile (dunes); poor compaction characteristics.
Sandy soils of mixed origin	Sheetwash, wind, termites	Collapsible fabric; dispersive characteristics; compressibility, subject to flooding.
Alluvium	Streams	Collapsible fabric; dispersive characteristics; compressibility, subject to flooding.
Lacustrine	Streams depositing in lakes, pans, or vleis	Compressibility, expansive characteristics; high soluble salt content.

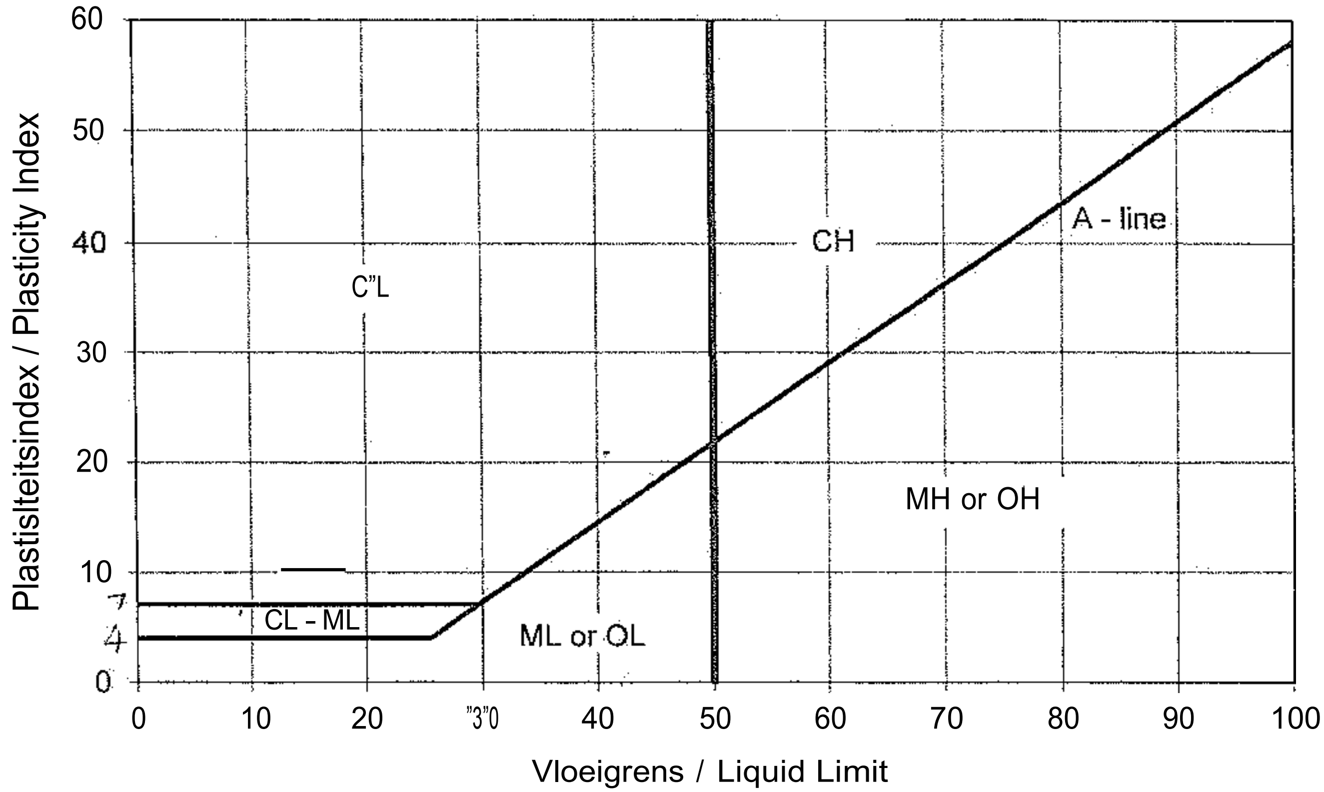
COLOUR

TERM	DESCRIPTION
SPECKLED	Very small patches of colour <2 mm.
MOTTLED	Irregular patches of colour 2 – 6 mm.
BLOTCHED	Large irregular patches of colour 6 – 20 mm.
BANDED	Approximately parallel bands of varying colour.
STREAKED	Randomly orientated streaks of colour.
STAINED	Local colour variations: associated with discontinuity surfaces.

UNIFIED CLASSiFICATION SYSTEM

Description			Group	Laboratory Criteria			
				Fines	Grading	Plasticity	Notes
Coarse grained. (More"than 50% > 0,063 mm)	Gravels (more gravel than sand: graV"el is > 2 mm and sand is between 0,063 mm and 2 mm)	Well-graded gravels, sandy gravels, with little or no fines	GW	0 - 5 %	Cu » 4 and 1 < Cz < 3		Oual symbols if 5-12°/« fines.
		Poorly graded gravels, sandy gravels, with little or no fines	GP	0 - 5 %	Not satisfying GW requirements		
		Silty gravels, silty sandy gravels	GM	> IZ %		Below A-line and PI < 4	Dual symbols if above A-line and 4 < PI < 7
		Clayey gravels, clayey sandy gravels	GC	»]2 %		Above A-riκe and PI » 7	
	Bands (more sand than gravel: gravel is > 2 mm and sand is between *0,063 mm* and 2 mm)	Well-graded sands, gravelly sands, with little or no fines	SW	0 - 5 %	Cu » 6 and 1 < Cz « 3		Oual symbols if 5-12⁷ fines.
		Poorly graded sands, gravelly sands, with little or no fines	SP	0 - 5 %	Not satisfying SW requirements		
		Silly sands	SM	» 12 %		Below A-line and PI < 4	Dual symbols if above A-line and 4 < PI < 7
		Clayey sands	SC	> 12 %		Above A-iine and PI » 7	
Fine grained. (Mare than 50% < 0,063	Silts and clays (liquid limit < 50%)	Inorganic sifts, silly or clayey fine sands; with slight plasticity	ML	Use pJasticity chart			
		Inorganic clays, silty clays, sandy	CL	Lfse pfasticity chart			
		Organic silts and organic silly clays	OL	Use plasticity chart			
	Silts and clays (liquid limit > 50%)	Inorganic silts of high plasticity	MH	Use plasticity chart			
		Inorganic clays of high platicity	CH	Use plasticity chart			
		Organic clays of high plasticity	OH	Use plasticity chart			
Highly organic soils		Peat and other hiqh)y organic soils	Pt				

Unified Plasticity Chart



Appendix C – Soil classifications

RESULTATE EKSPERIMENT Nr.: GZ

Uitgevoer deur Groep nr. .... Datum: 17/10/2016

Name: Ian Jones

Toetse uitgevoer op monster nr.: TG1-3

Gewig monster: 500g

Sif grootte	Agterblywende gewig	% op sif	% wat deurgaan
75			
53			
37,5			
26,5			
19			
13,2			100
4,75	1,6	0,32	99,68
2	2,6	0,52	99,16
0,425	54,9	10,98	88,18
< 0,425	2,0+438,9= 440,9	88,18	0

Totale Gewig: ....

Gewig in vrugtefles: .... gm.

BESINKINGSTOETS

		Temp.	Korr.	Gem.
40 sek lesing, < 0,05 mm	1.	19		22
	2.			
	3.			
1 uur lesing, < 0,005 mm		19		15

SIFTOETS

0,425 - ~~0,208~~ 0,212 mm	27,1
0,208 - 0,15 mm	20,3
0,15 - 0,075 mm	13,0
< 0,075 mm	0,4+39,2 = 39,6

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Monster verskaf deur: Ian Jones Datum: 17/10/2016

Monsterbeskrywing			Sifanalise										Besinkingstoets						Atterberggrense		
			% wat deurgaan										Fyn gruis	Sand							
															Fyn	Baie fyn	Slik	Klei			
Nr.	Afstand m	Diepte m	75	53	37,5	26,5	19	13,2	4,75	2	0,425	0,075	2 – 0,425	0,425 – 0,208	0,208 – 0,15	0,15 – 0,05	0,05 – 0,005	0,005 –	Vloeigrens	Plastisiteits-indeks	% Liniêre krimping
			100	100	100	100	100	100	99,7	99,2	88,2	34,9	11,1	24,1	18,1	27,2	6,2	13,3			
																			Non Plastic		

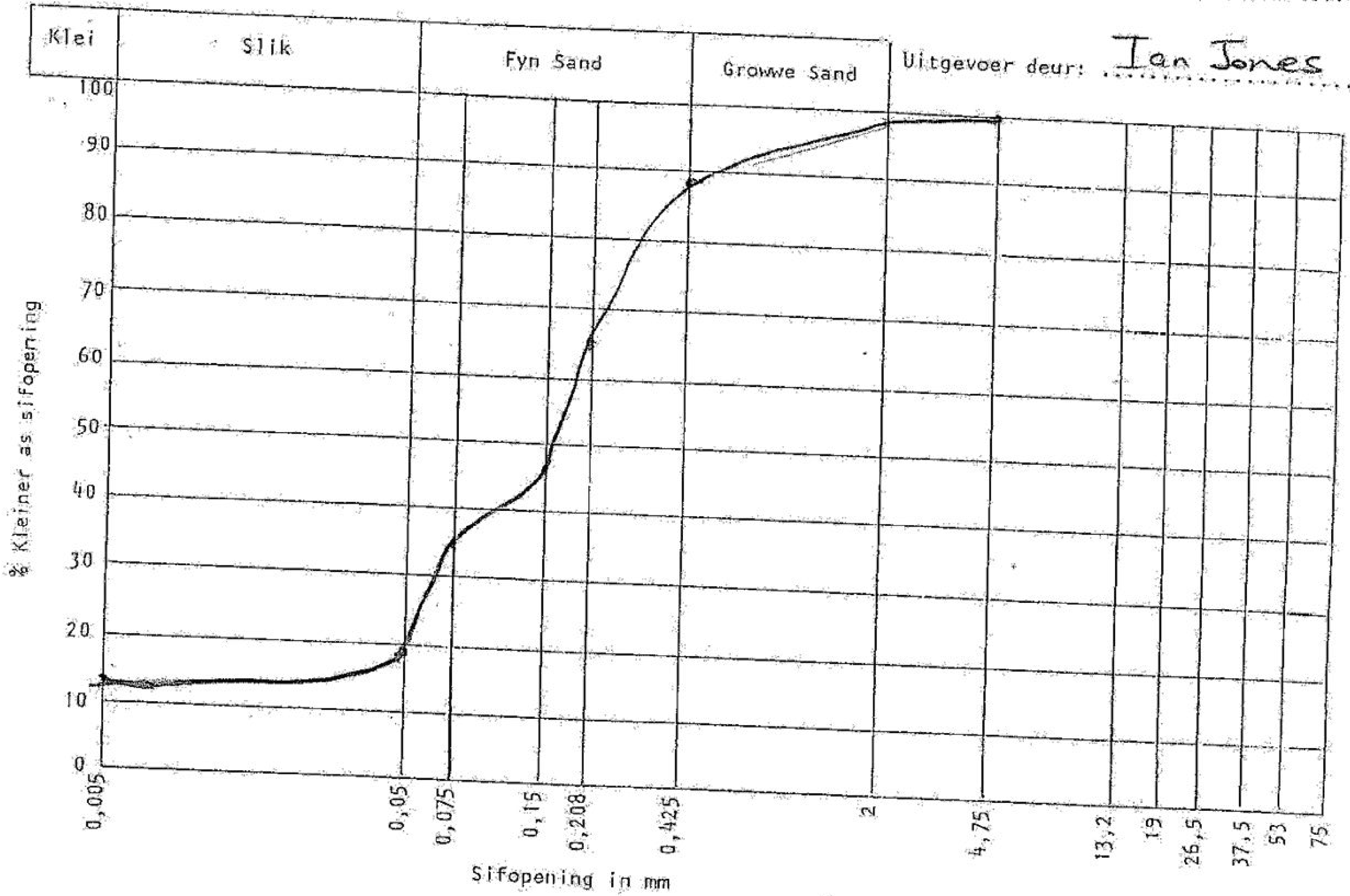
Sifopeninge in mm.

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Datum: 17/10/2016 Monster nr: .... Materiaal: TG1-3

Uitgevoer deur: Ian Jones



RESULTATE - EKSPERIMENT NO. G 1

Datum: 19/10/2016

Uitgevoer deur Groep No.

Name: Ian Jones

Toets uitgevoer op monster No. TG1-3

Kamer temperatuur

Relatiewe Humiditeit

Weersomstandighede

1. Vloeigrenstoets:

	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	gm	gm
b. Gewig van bottel en droë monster	gm	gm
c. Gewig van bottel	gm	gm
d. Vloeigrens = $(\frac{a-b}{b-c})x100$ 100%vog	%	%
Gemiddelde vloeigrens (V.G.)	%	

2. Linière Krimping

	1ste Toets	2de Toets
a. Lengte van nat monster	mm	mm
b. Lengte van monster na uitdroging	mm	mm
c. Verkorting van monster (a – b)	mm	mm
d. Linière Krimping $(\frac{c}{a})x100$ %	%	%
Gemiddelde Linière krimping	%	

3. Plastiese Grens

	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	gm	gm
b. Gewig van bottel en droë monster	gm	gm
c. Gewig van bottel	gm	gm
d. Gewig van vog afgedryf (a – b)	gm	gm
e. Gewig van droë monster (b - c)	gm	gm
f Plastiese Grens $(\frac{d}{e}).100)$ %vog	%	%
Gemiddelde Plastiese Grens (P.G.)	%	
g. Plastisiteitsindeks P.I. = (V.G. – P.G.)		

Handtekening van Studente

Not plastic

RESULTATE EKSPERIMENT Nr, G2

Uitgevoer deur Groep nr. .... Datum: 17/10/2016

Name: Ian Jones

....

Toetse uitgevoer op monster nr.: TG1-4

Gewig monster: 500g

Sif grootte	Agterblywende gewig	% op sif	% wat deurgaan
75			
53			
37,5			
26,5			
19			
13,2			
4,75			100
2	0,2	0,04	99,96
0,425	33,0	6,6	93,36
< 0,425	3,2 + 463,6 = 466,8	93,36	0

Totale Gewig: ....

Gewig in vrugtefles: .... gm.

BESINKINGSTOETS

		Temp.	Korr.	Gem.
40 sek lesing, < 0,05 mm	1.	22		22
	2.			
	3.			
1 uur lesing, < 0,005 mm		21		15

SIFTOETS

0,425 - ~~0,208~~ 0,212 mm ..36,1..........

0,208 - 0,15 mm ..21,7..........

0,15 - 0,075 mm ..14,2..........

....

< 0,075 mm 0,5 + 27,5 = 28

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Monster verskaf deur: Ian Jones .... Datum: 17/10/2016 .....

Monsterbeskrywing			Sifanalise										Besinkingstoets						Atterberggrense		
			% wat deurgaan											Sand							
Nr.	Afstand m	Diepte m	75	53	37,5	26,5	19	13,2	4,75	2	0,425	0,075	Fyn gruis		Fyn	Baie Fyn	Slik	Klei	Vloeigrens	Plastisiteits-indeks	% Liniêre krimping
													2 – 0,125	0,425 – 0,208	0,208 – 0,15	0,15 – 0,05	0,05 – 0,005	0,005 –			
			100	100	100	100	100	100	100	100	93,4	26,1	6,6	33,7	20,3	18,9	6,5	14,0			
																			Non Plastic		

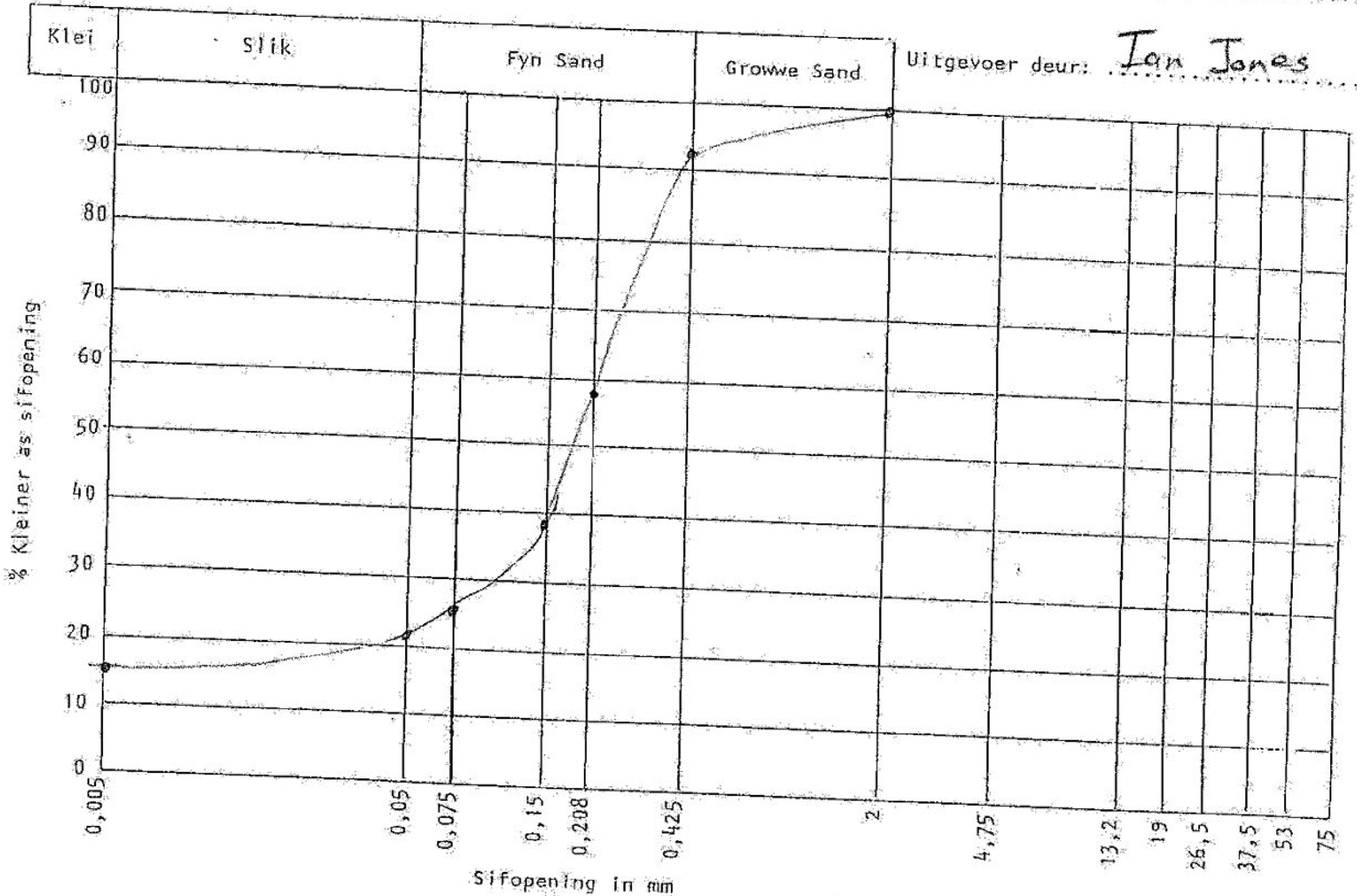
Sifopeninge in mm.

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Datum: 17/10/2016 Monster nr: .... Materiaal: TG1-4

Uitgevoer deur: Ian Jones



RESULTATE - EKSPERIMENT NO. G 1

Datum: 19/10/2016

Uitgevoer deur Groep No.

Name: Ian Jones

Toets uitgevoer op monster No. TG1-4

Kamer temperatuur

Relatiewe Humiditeit

Weersomstandighede

1. Vloeigrenstoets:	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	gm	gm
b. Gewig van bottel en droë monster	gm	gm
c. Gewig van bottel	gm	gm
d. Vloeigrens = $(\frac{a-b}{b-c})x100$ 100%vog	%	%
Gemiddelde vloeigrens (V.G.)	%	

2. Liniêre Krimping	1ste Toets	2de Toets
a. Lengte van nat monster	mm	mm
b. Lengte van monster na uitdroging	mm	mm
c. Verkorting van monster (a – b)	mm	mm
d. Liniêre Krimping $(\frac{c}{a})x100$ %	%	%
Gemiddelde Liniêre krimping	%	

3. Plastiese Grens	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	gm	gm
b. Gewig van bottel en droë monster	gm	gm
c. Gewig van bottel	gm	gm
d. Gewig van vog afgedryf (a – b)	gm	gm
e. Gewig van droë monster (b - c)	gm	gm
f. Plastiese Grens $(\frac{d}{e}).100)$ %vog	%	%
Gemiddelde Plastiese Grens (P.G.)	%	
g. Plastisiteitsindeks P.I. = (V.G. – P.G.)		

Handtekening van Studente

Nie Plasties

RESULTATE EKSPERIMENT Nr. G2

Uitgevoer deur Groep nr. .... Datum: 17/10/2016

Name: Ian Jones

....

Toetse uitgevoer op monster nr.: TG2-2

Gewig monster: 500 g

Sif grootte	Agterblywende gewig	% op sif	% wat deurgaan
75			
53			
37,5			
26,5			
19			100
13,2	21,2	4,24	95,76
4,75	7,9	1,58	94,18
2	24,0	4,8	89,38
0,425	154,5	30,9	58,48
< 0,425	1,1+291,3 = 292,4	58,48	0

Totale Gewig: ....

Gewig in vrugtefles: .... gm.

BESINKINGSTOETS

		Temp.	Korr.	Gem.
40 sek lesing, < 0,05 mm	1.	19		12
	2.			
	3.			
1 uur lesing, < 0,005 mm		19		5

SIFTOETS

0,425 - ~~0,208~~ 0,212 mm	37,2
0,208 - 0,15 mm	21,3
0,15 - 0,075 mm	20,7
< 0,075 mm	0,6 + 20,2 = 20,8

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Monster verskaf deur: Ian Jones Datum: 17/10/2016

Monsterbeskrywing			Sifanalise										Besinkingstoets						Atterberggrense		
			% wat deurgaan										Fyn gruis	Sand			Slik	Klei			
Nr.	Afstand m	Diepte m	75	53	37,5	26,5	19	13,2	4,75	2	0,425	0,075	2 – 0,125	0,425 – 0,208	Fyn 0,208 – 0,15	Baie Fyn 0,15 – 0,05	0,05 – 0,005	0,005 –	Vloeigrens	Plastisiteits-indeks	% Liniêre krimping
			100	100	100	100	100	95,8	94,2	89,4	58,5	12,2	34,6	24,3	13,9	19,3	4,6	3,3			
																			Non Plastic		

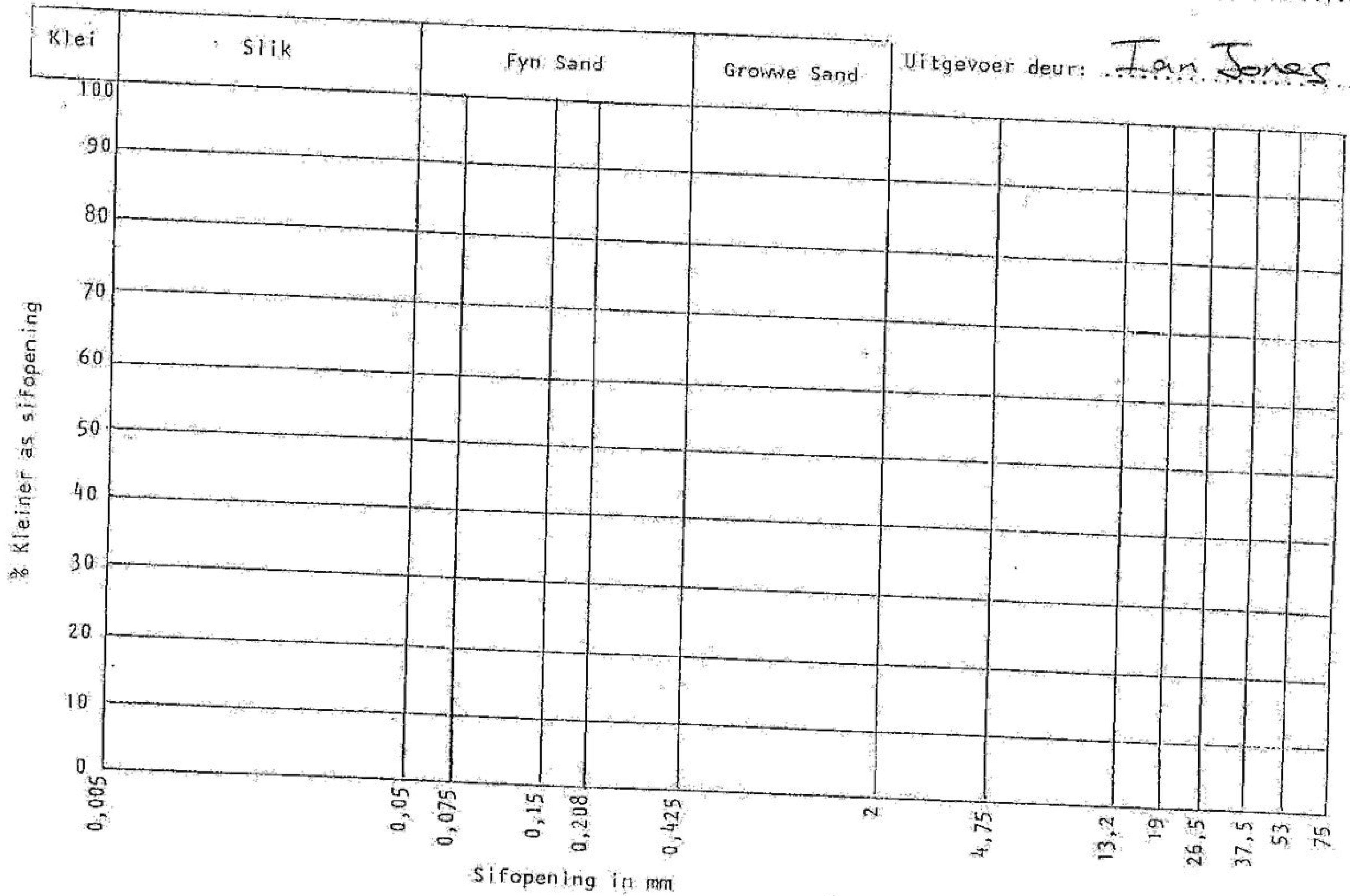
Sifopeninge in mm.

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Datum: 17/10/2016 Monster nr: .... Materiaal: TG2-2

Uitgevoer deur: Ian Jones



RESULTATE - EKSPERIMENT NO. G 1

Datum: 19/10/2016

Uitgevoer deur Groep No.

Name: Ian Jones

Toets uitgevoer op monster No. TG2-2

Kamer temperatuur

Relatiewe Humiditeit

Weersomstandighede

	1ste Toets	2de Toets
1. Vloeigrenstoets:		
a. Gewig van bottel en nat monster	gm	gm
b. Gewig van bottel en droë monster	gm	gm
c. Gewig van bottel	gm	gm
d. Vloeigrens = $(\frac{a-b}{b-c})x100$ 100%vog	%	%
Gemiddelde vloeigrens (V.G.)	%	
2. Liniêre Krimping	1ste Toets	2de Toets
a. Lengte van nat monster	mm	mm
b. Lengte van monster na uitdroging	mm	mm
c. Verkorting van monster (a – b)	mm	mm
d. Liniêre Krimping $(\frac{c}{a})x100$ %	%	%
Gemiddelde Liniêre krimping	%	
3. Plastiese Grens	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	gm	gm
b. Gewig van bottel en droë monster	gm	gm
c. Gewig van bottel	gm	gm
d. Gewig van vog afgedryf (a – b)	gm	gm
e. Gewig van droë monster (b - c)	gm	gm
f. Plastiese Grens $((\frac{d}{e}).100)$ %vog	%	%
Gemiddelde Plastiese Grens (P.G.)	%	
g. Plastisiteitsindeks P.I. = (V.G. – P.G.)		

Handtekening van Studente

SAND
Not Plastic

RESULTATE EKSPERIMENT Nr. G2

Uitgevoer deur Groep nr. .... Datum: 17/10/2016

Name: Jan Jones

Toetse uitgevoer op monster nr.: TG2-3

Gewig monster: 500g

Sif grootte	Agterblywende gewig	% op sif	% wat deurgaan
75			
53			
37,5			
26,5			
19			100
13,2	6,1	1,22	98,78
4,75	9,5	1,9	96,88
2	11,0	2,2	94,68
0,425	106,7	21,34	73,34
< 0,425	0,9 + 365,8 = 366,7	73,34	0

Totale Gewig: ....

Gewig in vrugtefles: .... gm.

BESINKINGSTOETS

		Temp.	Korr.	Gem.
40 sek lesing, < 0,05 mm	1.	22		35
	2.			
	3.			
1 uur lesing, < 0,005 mm		21		27

SIFTOETS

0,425 − ~~0,205~~ 0,212 mm 21,0

0,208 − 0,15 mm 14,8

0,15 − 0,075 mm 13,7

< 0,075 mm .... 0,4 + 50,1 = 50,5

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Monster verskaf deur: Ian Jones Datum: 17/10/2016

Monsterbeskrywing			Sifanalise										Besinkingstoets						Atterberggrense		
			% wat deurgaan										Fyn gruis	Sand			Slik	Klei			
Nr.	Afstand m	Diepte m	75	53	37,5	26,5	19	13,2	4,75	2	0,425	0,075			Fyn	Baie fyn			Vloeigrens	Plastisiteits-indeks	% Liniêre krimping
													2 – 0,425	0,425 – 0,208	0,208 – 0,15	0,15 – 0,05	0,05 – 0,005	0,005 –			
TG2-3			100	100	100	100	100	98,8	96,9	94,7	73,3	37,0	22,5	16,3	11,5	22,6	6,2	20,9	30,7	18,3	6,7

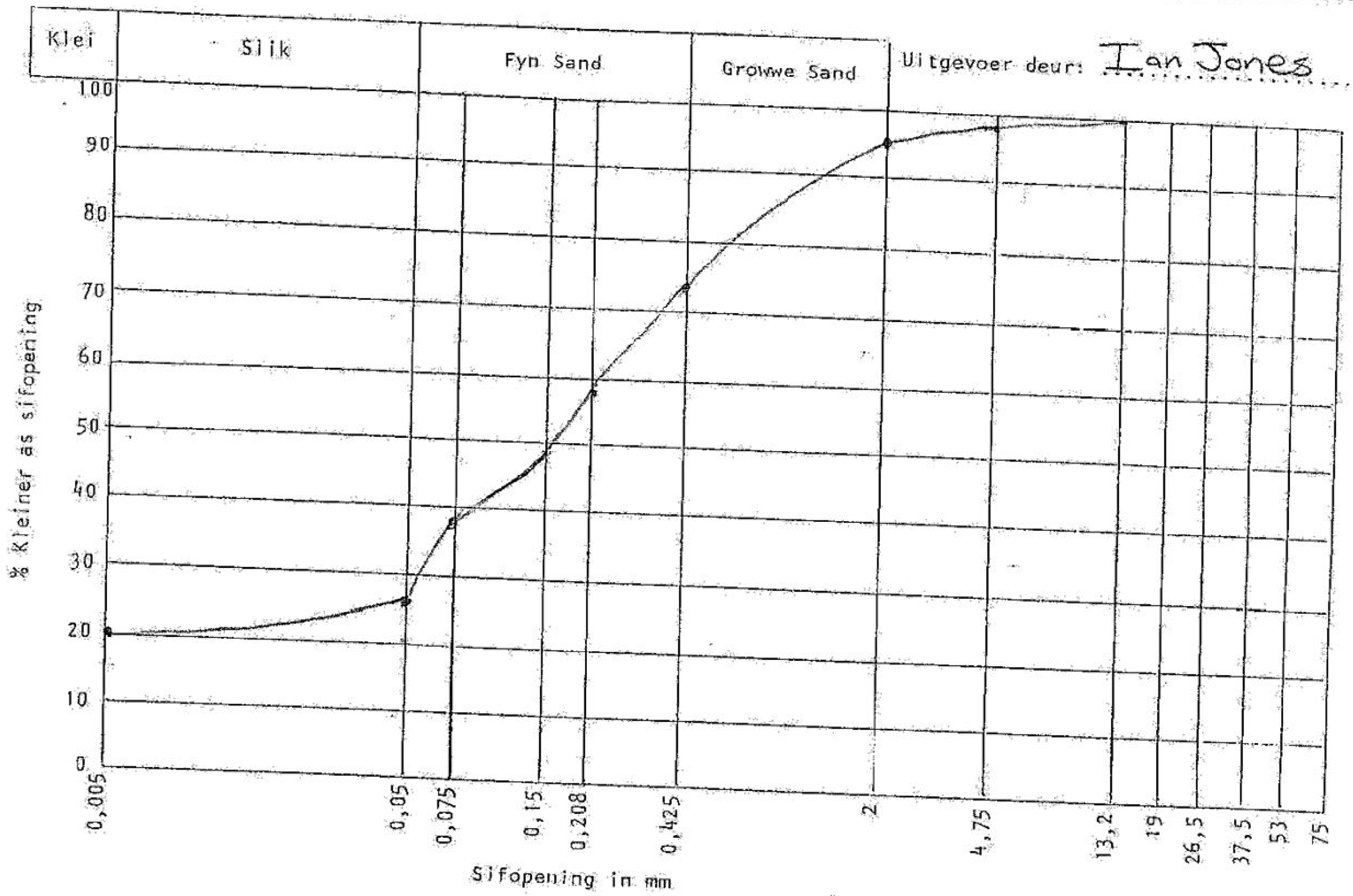
Sifopeninge in mm.

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Datum: 17/10/2016 Monster nr: TGZ-3 Materiaal: ....

Uitgevoer deur: Ian Jones



RESULTATE - EKSPERIMENT NO. G 1

Datum: 19/10/2016

Uitgevoer deur Groep No.

Name: Ian Jones

Toets uitgevoer op monster No. TG 2-3

Kamer temperatuur

Relatiewe Humiditeit

Weersomstandighede

1. Vloeigrenstoets:	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	52,4 gm	gm
b. Gewig van bottel en droë monster	42,9 gm	gm
c. Gewig van bottel (24)	12,0 gm	gm
d. Vloeigrens = $(\frac{a-b}{b-c})x100$ 100%vog	30,7 %	%
Gemiddelde vloeigrens (V.G.)	30,7 %	

2. Liniêre Krimping	1ste Toets	2de Toets
a. Lengte van nat monster	150 mm	mm
18 b. Lengte van monster na uitdroging	140 mm	mm
c. Verkorting van monster (a – b)	10 mm	mm
d. Liniêre Krimping $(\frac{c}{a})x100$ %	6,7 %	%
Gemiddelde Liniêre krimping	6,7 %	

3. Plastiese Grens	1ste Toets	2de Toets
a. Gewig van bottel en nat monster	33,2 gm	gm
b. Gewig van bottel en droë monster	29,8 gm	gm
c. Gewig van bottel (21)	11,2 gm	gm
d. Gewig van vog afgedryf (a – b)	3,4 gm	gm
e. Gewig van droë monster (b - c)	18,6 gm	gm
f Plastiese Grens $(\frac{d}{e}).100)$ %vog	18,3 %	%
Gemiddelde Plastiese Grens (P.G.)	18,3 %	
g. Plastisiteitsindeks P.I. = (V.G. – P.G.)	12,4	

Handtekening van Studente

RESULTATE EKSPERIMENT Nr. G2

Uitgevoer deur Groep nr. .... Datum: 17/10/2016

Name: Ian Jones

Toetse uitgevoer op monster nr.: Dam Wal

Gewig monster: 500g

Sif grootte	Agterblywende gewig	% op sif	% wat deurgaan
75			
53			
37,5			
26,5			
19			
13,2	—		100
4,75	8,7	1,74	98,26
2	14,2	2,84	95,42
0,425	133,5	26,7	68,72
< 0,425	1,9 + 341,7 = 343,6	68,72	0

Totale Gewig: ....

Gewig in vrugtefles: .... gm.

BESINKINGSTOETS

		Temp.	Korr.	Gem.
40 sek lesing, < 0,05 mm	1.	19		25
	2.			
	3.			
1 uur lesing, < 0,005 mm		19		16

SIFTOETS

0,425 - ~~0,208~~ 0,212 mm ..32,2..

0,208 - 0,15 mm ..17,8..

0,15 - 0,075 mm ..15,1..

....

< 0,075 mm 0,7 + 34,2 = 34,9

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Monster verskaf deur: Ian Jones Datum: 17/10/2016

Monsterbeskrywing			Sifanalise										Besinkingstoets						Atterberggrense		
			% wat deurgaan										Fyn gruis	Sand			Slik	Klei	Vloeigrens	Plastisiteits-indeks	% Liniêre krimping
Nr.	Afstand m	Diepte m	75	53	37,5	26,5	19	13,2	4,75	2	0,425	0,075	2 – 0,125	0,425 – 0,208	Fyn 0,208 – 0,15	Baie fyn 0,15 – 0,05	0,05 – 0,005	0,005			
			100	100	100	100	100	100	98,3	95,4	68,7	24,0	28,0	23,2	12,8	18,0	6,5	11,5	15,9	13,2	2

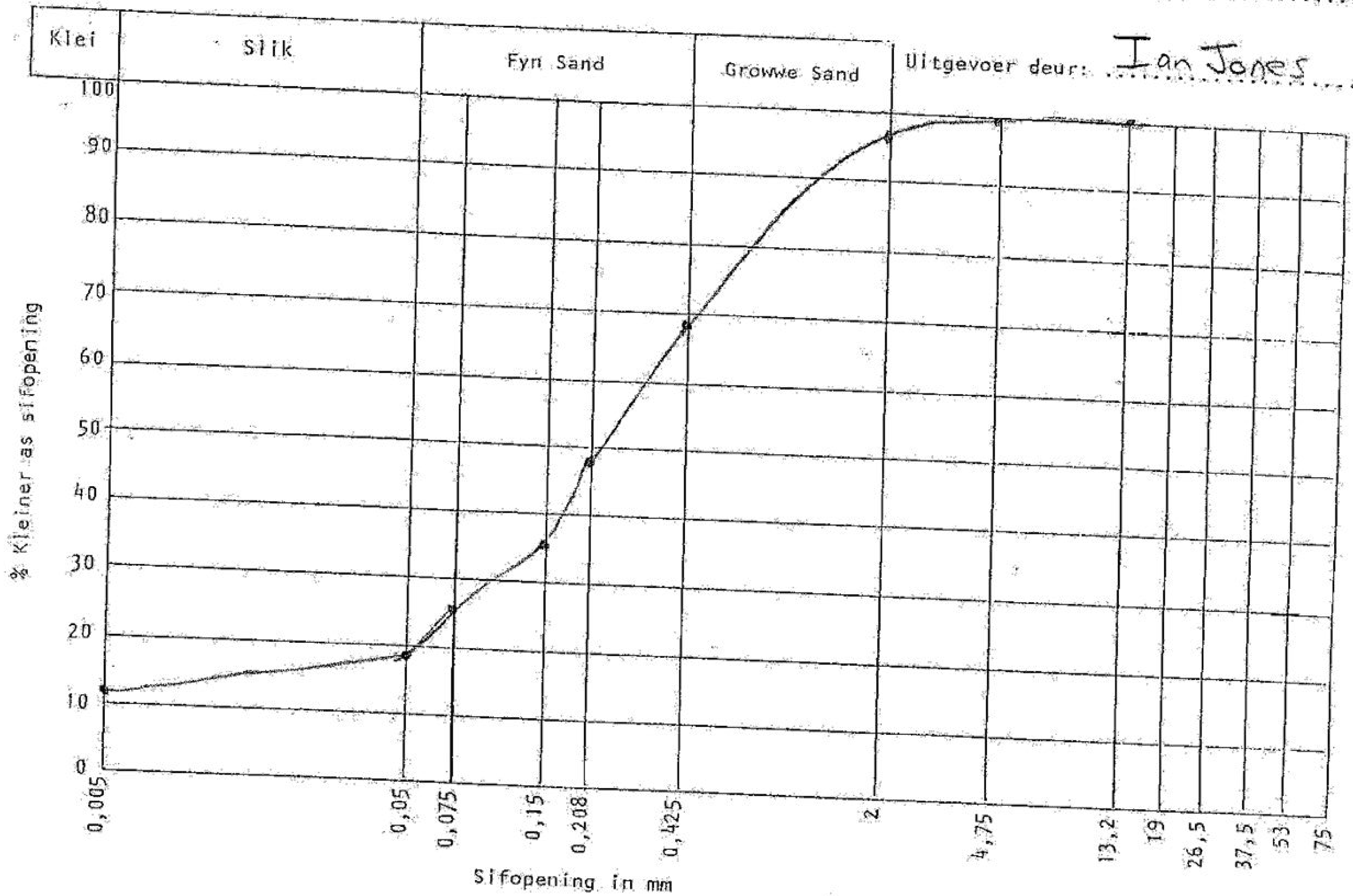
Sifopeninge in mm.

UNIVERSITEIT VAN STELLENBOSCH

Siviele Laboratorium

Datum: 17/10/2016 Monster nr: .... Materiaal: Dam Wal

Uitgevoer deur: Ian Jones



RESULTATE - EKSPERIMENT NO. G 1

Datum: 19/10/2016

Uitgevoer deur Groep No.

Name: Ian Jones

Toets uitgevoer op monster No. Dam Wal

Kamer temperatuur

Relatiewe Humiditeit

Weersomstandighede

	1ste Toets		2de Toets	
1. Vloeigrenstoets:				
a. Gewig van bottel en nat monster	51,2	gm		gm
b. Gewig van bottel en droë monster	45,9	gm		gm
c. Gewig van bottel (8)	12,5	gm		gm
d. Vloeigrens = $(\frac{a-b}{b-c})x100$ 100%vog	15,9	%		%
Gemiddelde vloeigrens (V.G.)	15,9	%		
2. Liniêre Krimping				
a. Lengte van nat monster	150	mm		mm
b. Lengte van monster na uitdroging	147	mm		mm
c. Verkorting van monster (a – b)	3	mm		mm
d. Liniêre Krimping $(\frac{c}{a})x100$ %	2	%		%
Gemiddelde Liniêre krimping	2	%		
3. Plastiese Grens				
a. Gewig van bottel en nat monster	32,0	gm		gm
b. Gewig van bottel en droë monster	29,6	gm		gm
c. Gewig van bottel (14)	11,4	gm		gm
d. Gewig van vog afgedryf (a – b)	2,4	gm		gm
e. Gewig van droë monster (b - c)	18,2	gm		gm
f. Plastiese Grens $(\frac{d}{e}).100)$ % vog	13,2	%		%
Gemiddelde Plastiese Grens (P.G.)	13,2	%		
g. Plastisiteitsindeks P.I. = (V.G. – P.G.)	2,7			

Handtekening van Studente

Appendix D – Proctor Tests

Method A7

SAMPLE NO/ MONSTERNR.: TG1-3

DATE/DATUM: 10/10/2016

MASS TAKEN / MASSA GENEEM: 7 Kg

OPERATOR/TOETSER: I. Jones

DESCRIPTION / BESKRYWING:

1. APPROXIMATE VALUES / BENADERDE WAARDES

a) WATER ADDED / WATER BYGEVOEG

MILLILITRE / MILLILITER	420	560	770	980	
PERCENTAGE / PERSENTASIE	6	8	11	14	
ASSUMED M.C. / VERONDERSTELDE VI %(D) % ADDED + HYGROSCOPIC MOISTURE CONTENT / % BYGEVOEG + HIGROSKOPIESEVOG-INHOUD					

b) DRY DENSITY / DROË DIGTHEID

MOULD NO. / VORMNR.	19	19	19	19	
MOULD FACTOR / VORMFAKTOR (F)	42	42	42	42	
MASS OF MOULD + WET SOIL / MASSA VAN VORM + NAT GROND	7930	8277	8356	8133	
MASS OF MOULD / MASSA VAN VORM	3321	3321	3321	3321	
MASS OF WET SOIL / MASSA VAN NAT GROND (W)	4609	4956	5035	4812	
APPROXIMATE DRY DENSITY / BENADERDE DROË DIGTHEID $\left(\frac{WXF}{100+D}\right)$	1826	1927	1905	1773	

2. ACTUAL VALUES / WERKLIKE WAARDES

a) MOISTURE / VOG

CONTAINER NO / HOUERNR	XL	14	25A	24	
MASS OF CONTAINER + WET SOIL / MASSA VAN HOUER + NAT GROND	574,4	613,4	734,7	773,9	
MASS OF CONTAINER + DRY SOIL / MASSA VAN HOUER + DROË GROND	554,6	583,8	694,5	707,8	
MASS OF CONTAINER / MASSA VAN HOUER	236,9	237,2	237	237,2	
MASS OF WATER / MASSA VAN WATER	19,8	29,6	40,2	66,1	
MASS OF DRY SOIL / MASSA VAN DROË GROND	317,7	346,6	457,5	470,6	
MOISTURE CONTENT(%) / VOGINHOUD(%) (D)	6,2	8,5	8,8	14,0	

b) DRY DENSITY / DROË DIGTHEID

$\left(\frac{WXF}{100+D}\right)$	1823	1918	1943	1772	

MAX DRY DENSITY / OPTIMUM MOISTURE CONTENT

MAKS DROË DIGTHEID / OPTIMUM VOGINHOUD

FORM A7/1

Recording sheet for the determination of the maximum dry density and the optimum moisture content

Standard Test Methods
TMH1, Pretoria, South Africa, 1986

Method A7

SAMPLE NO/ MONSTERNR.: TG1-4

DATE/DATUM: 11/10/2016

MASS TAKEN/MASSA GENEEM: 7 kg

OPERATOR/TOETSER: Ian Jones

DESCRIPTION/BESKRYWING:

1. APPROXIMATE VALUES / BENADERDE WAARDES

a) WATER ADDED/WATER BYGEVOEG

MILLILITRE / MILLILITER	350	560	770	910	
PERCENTAGE / PERSENTASIE	5	8	11	13	
ASSUMED M.C. / VERONDERSTELDE V.I. %(DI) % ADDED + HYGROSCOPIC MOISTURE CONTENT / % BYGEVOEG + HIGROSKOPIESEVOG-INHOUD					

b) DRY DENSITY / DROË DIGTHEID

MOULD NO. / VORMNR.	19	19	19	19	
MOULD FACTOR / VORMFAKTOR (F)	42	42	42	42	
MASS OF MOULD + WET SOIL MASSA VAN VORM + NAT GROND	7826	8439	8403	8262	
MASS OF MOULD / MASSA VAN VORM	3321	3321	3321	3321	
MASS OF WET SOIL / MASSA VAN NAT GROND (W)	4505	5118	5082	4941	
APPROXIMATE DRY DENSITY / BENADERDE DROË DIGTHEID $\left(\frac{W \times F}{100+DI}\right)$	1802	1990	1922	1836	

2. ACTUAL VALUES / WERKLIKE WAARDES

a) MOISTURE / VOG

CONTAINER NO / HOUERNR	AG	2AB	WP	12	
MASS OF CONTAINER + WET SOIL MASSA VAN HOUER + NAT GROND	642,5	610,9	786,5	752,4	
MASS OF CONTAINER + DRY SOIL MASSA VAN HOUER + DROË GROND	616,1	579,9	729,6	688,5	
MASS OF CONTAINER / MASSA VAN HOUER	176,5	236,8	237,3	236,9	
MASS OF WATER / MASSA VAN WATER	26,4	31	56,9	63,9	
MASS OF DRY SOIL / MASSA VAN DROË GROND	439,6	343,1	492,3	451,6	
MOISTURE CONTENT (%) / VOGINHOUD (%) (D)	6,0	9,0	11,6	14,1	

b) DRY DENSITY / DROË DIGTHEID

$\left(\frac{W \times F}{100+D}\right)$	1785	1972	1913	1819	

MAX DRY DENSITY / OPTIMUM MOISTURE CONTENT

MAKS DROË DIGTHEID / OPTIMUM VOGINHOUD

FORM A7/1

Recording sheet for the determination of the maximum dry density and the optimum moisture content

Standard Test Methods
TMH1, Pretoria, South Africa, 1986

Method A7

SAMPLE NO/ MONSTERNR. : TG2-2

DATE/DATUM: 12/10/2016 MASS TAKEN / MASSA GENEEM : 6 kg

OPERATOR / TOETSER: Ian Jones + G. Williams

DESCRIPTION / BESKRYWING :

1. APPROXIMATE VALUES / BENADERDE WAARDES

a) WATER ADDED / WATER BYGEVOEG

MILLILITRE / MILLILITER	180	360	480	600	
PERCENTAGE / PERSENTASIE	3	6	8	10	
ASSUMED M.C. / VERONDERSTELDE V.I. %(D) % ADDED + HYGROSCOPIC MOISTURE CONTENT / % BYGEVOEG + HIGROSKOPIESEVOG-INHOUD					

b) DRY DENSITY / DROË DIGTHEID

MOULD NO. / VORMNR.	18	18	18	18	
MOULD FACTOR / VORMFAKTOR (F)	428	428	428	428	
MASS OF MOULD + WET SOIL / MASSA VAN VORM + NAT GROND	7788	8049	8219	7895	
MASS OF MOULD / MASSA VAN VORM	3321	3321	3321	3321	
MASS OF WET SOIL / MASSA VAN NAT GROND (W)	4467	4728	4898	4574	
APPROXIMATE DRY DENSITY / BENADERDE DROË DIGTHEID $\left(\frac{W \times F}{100 + D}\right)$	1856	1909	1941	1779	

2. ACTUAL VALUES / WERKLIKE WAARDES

a) MOISTURE / VOG

CONTAINER NO. / HOUERNR	80	E3	70X	ZB	
MASS OF CONTAINER + WET SOIL / MASSA VAN HOUER + NAT GROND	720,9	691,1	685,7	750,9	
MASS OF CONTAINER + DRY SOIL / MASSA VAN HOUER + DROË GROND	703,9	664,5	648,3	701,0	
MASS OF CONTAINER / MASSA VAN HOUER	189,9	236,6	197,3	221,1	
MASS OF WATER / MASSA VAN WATER	17,0	26,6	37,4	49,9	
MASS OF DRY SOIL / MASSA VAN DROË GROND	514,0	427,9	451,0	479,9	
MOISTURE CONTENT (%) / VOGINHOUD (%) (D)	3,3	6,2	8,3	10,4	

b) DRY DENSITY / DROË DIGTHEID

$\left(\frac{W \times F}{100 + D}\right)$	1851	1905	1935	1773	

MAX DRY DENSITY / OPTIMUM MOISTURE CONTENT

MAKS DROË DIGTHEID / OPTIMUM VOGINHOUD

FORM A7/1

Recording sheet for the determination of the maximum dry density and the optimum moisture content

Method A7

SAMPLE NO/ MONSTERNR.: TG2-3

DATE/DATUM: 11/10/2016

MASS TAKEN / MASSA GENEEM: 7 kg

OPERATOR/TOETSER: Ian Jones

DESCRIPTION / BESKRYWING:

1. APPROXIMATE VALUES / BENADERDE WAARDES

a) WATER ADDED / WATER BYGEVOEG

MILLILITRE / MILLILITER	140	280	560	770	
PERCENTAGE / PERSENTASIE	2	4	8	11	
ASSUMED M.C. / VERONDERSTELDE V.I. % (DI) % ADDED + HYGROSCOPIC MOISTURE CONTENT / % BYGEVOEG + HIGROSKOPIESEVOG-INHOUD					

b) DRY DENSITY / DROË DIGTHEID

MOULD NO. / VORMNR.	19	19	19	19	
MOULD FACTOR / VORMFAKTOR (F)	42	42	42	42	
MASS OF MOULD + WET SOIL / MASSA VAN VORM + NAT GROND	7443	7801	8087	8063	
MASS OF MOULD / MASSA VAN VORM	3321	3321	3321	3321	
MASS OF WET SOIL / MASSA VAN NAT GROND (W)	4122	4480	4766	4742	
APPROXIMATE DRY DENSITY / BENADERDE DROË DIGTHEID $\left(\frac{W \times F}{100 + DI}\right)$	1697	1809	1853	1794	

2. ACTUAL VALUES / WERKLIKE WAARDES

a) MOISTURE / VOG

CONTAINER NO / HOUERNR	XL	14	25A	24	
MASS OF CONTAINER + WET SOIL / MASSA VAN HOUER + NAT GROND	719,9	674,3	606	662,6	
MASS OF CONTAINER + DRY SOIL / MASSA VAN HOUER + DROË GROND	671,6	622,4	552,5	589,4	
MASS OF CONTAINER / MASSA VAN HOUER	236,8	237,2	236,9	237,2	
MASS OF WATER / MASSA VAN WATER	48,3	51,9	53,5	73,2	
MASS OF DRY SOIL / MASSA VAN DROË GROND	434,8	385,2	315,6	352,2	
MOISTURE CONTENT (%) / VOGINHOUD (%) (D)	11,1	13,5	17	20,8	

b) DRY DENSITY / DROË DIGTHEID

$\left(\frac{W \times F}{100 + D}\right)$	1558	1658	1711	1649	

MAX DRY DENSITY / OPTIMUM MOISTURE CONTENT

MAKS DROË DIGTHEID / OPTIMUM VOGINHOUD

FORM A7/1

Recording sheet for the determination of the maximum dry density and the optimum moisture content

Standard Test Methods
TMH1, Pretoria, South Africa, 1986

Method A7

SAMPLE NO/ MONSTERNR: Dam Wal

DATE/DATUM: 12/10/2016 MASS TAKEN/MASSA GENEEM: 6kg

OPERATOR/TOETSER: G. Williams

DESCRIPTION/BESKRYWING:

1. APPROXIMATE VALUES / BENADERDE WAARDES

a) WATER ADDED / WATER BYGEVOEG

MILLILITRE / MILLILITER	180	360	480	600	
PERCENTAGE / PERSENTASIE	3	6	8	10	
ASSUMED M.C. / VERONDERSTELDE V.I. %(DI) % ADDED + HYGROSCOPIC MOISTURE CONTENT / % BYGEVOEG + HIGROSKOPIESE VOG-INHOUD					

b) DRY DENSITY / DROË DIGTHEID

MOULD NO. / VORMNR.	18	18	18	18	
MOULD FACTOR / VORMFAKTOR (F)	42,8	42,8	42,8	42,8	
MASS OF MOULD + WET SOIL / MASSA VAN VORM + NAT GROND	7810	8574	8455	8348	
MASS OF MOULD / MASSA VAN VORM	3321	3321	3321	3321	
MASS OF WET SOIL / MASSA VAN NAT GROND (W)	4489	5053	5134	5027	
APPROXIMATE DRY DENSITY / BENADERDE DROË DIGTHEID $\left(\frac{W \times F}{100 + DI}\right)$	1865	2040	2035	1956	

2. ACTUAL VALUES / WERKLIKE WAARDES

a) MOISTURE / VOG

CONTAINER NO / HOUERNR	WP	A12	YG	ZAB	
MASS OF CONTAINER + WET SOIL / MASSA VAN HOUER + NAT GROND	690,3	736,4	682,2	746,4	
MASS OF CONTAINER + DRY SOIL / MASSA VAN HOUER + DROË GROND	667,9	690,8	636,2	689,1	
MASS OF CONTAINER / MASSA VAN HOUER	237,5	200,5	236,9	236,8	
MASS OF WATER / MASSA VAN WATER	22,4	45,6	46	57,3	
MASS OF DRY SOIL / MASSA VAN DROË GROND	430,4	490,3	399,3	452,3	
MOISTURE CONTENT (%) / VOGINHOUD (%) (D)	5,2	9,3	11,5	12,7	

b) DRY DENSITY / DROË DIGTHEID

$\left(\frac{W \times F}{100 + D}\right)$	1826	1979	1970	1909	

MAX DRY DENSITY / OPTIMUM MOISTURE CONTENT

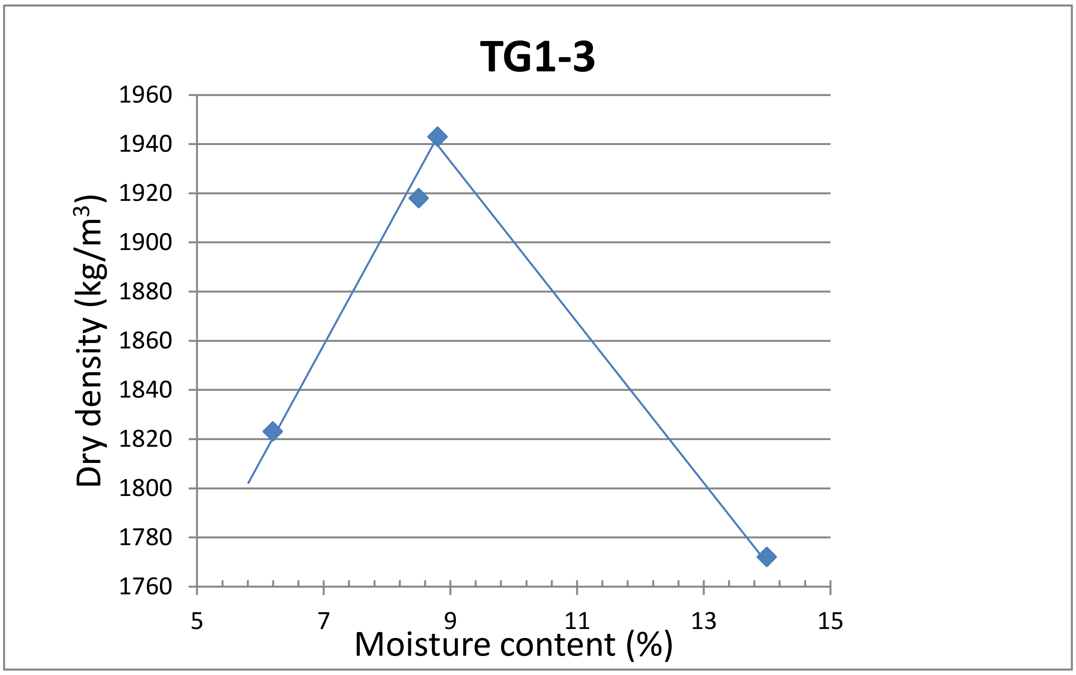
MAKS DROË DIGTHEID / OPTIMUM VOGINHOUD

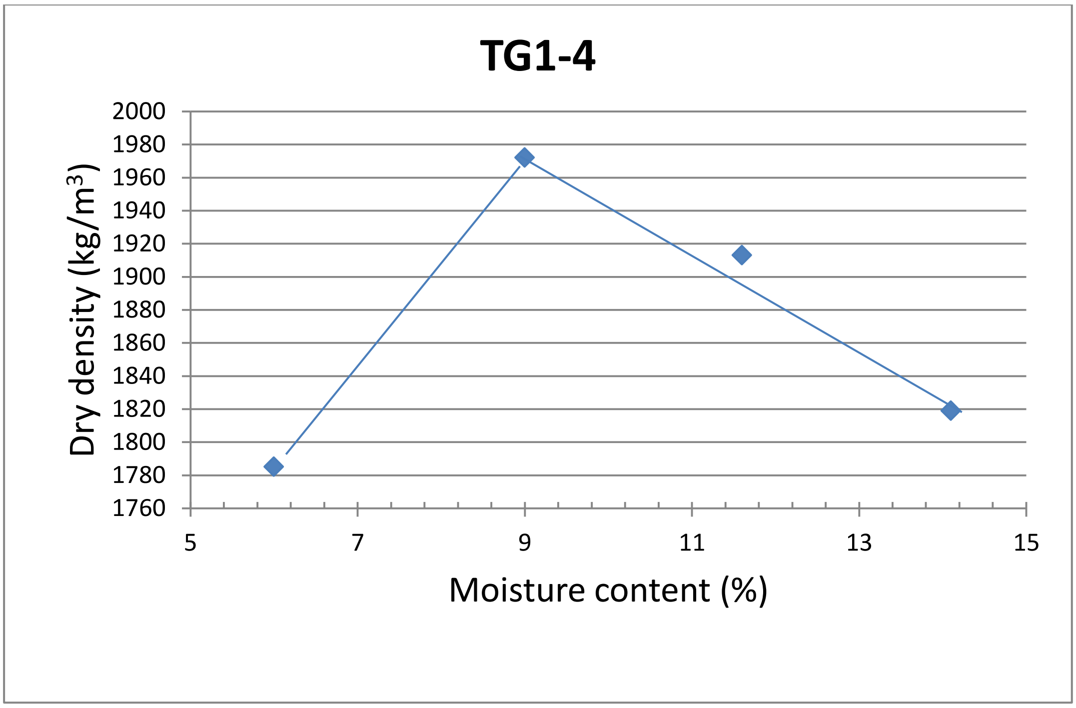
FORM A7/1

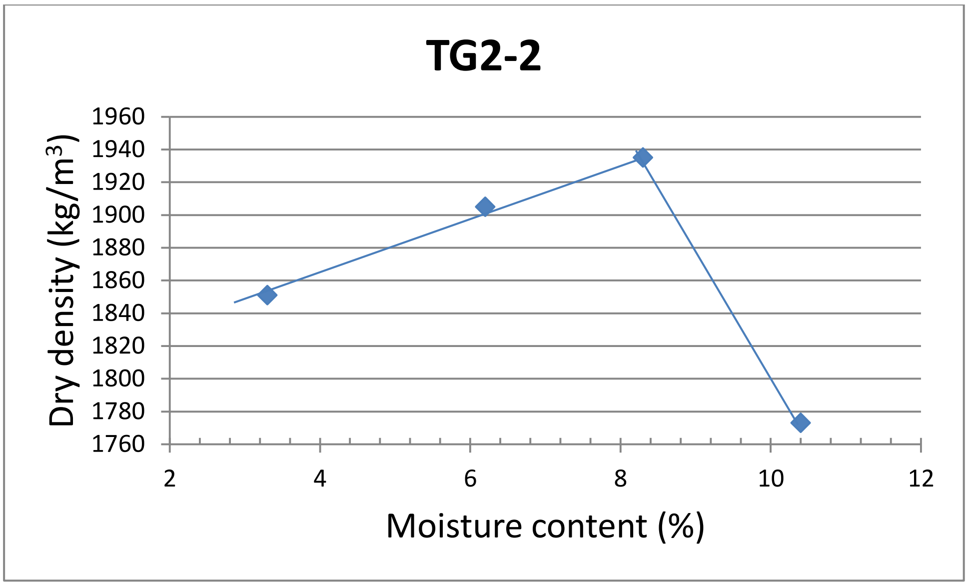
Recording sheet for the determination of the maximum dry density and the optimum moisture content

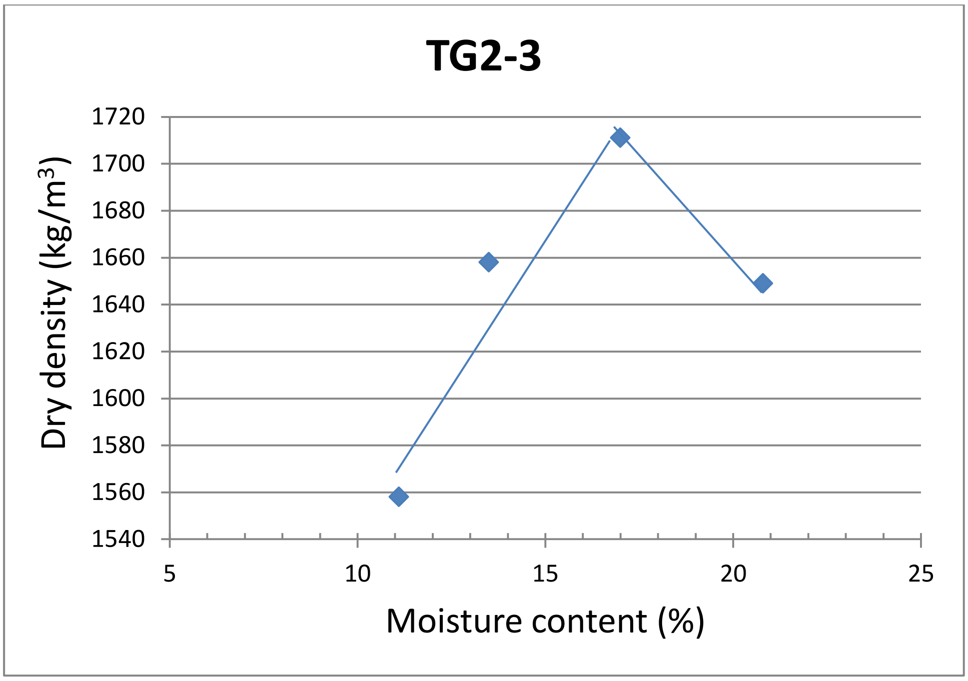
Standard Test Methods
TMH1, Pretoria, South Africa, 1986

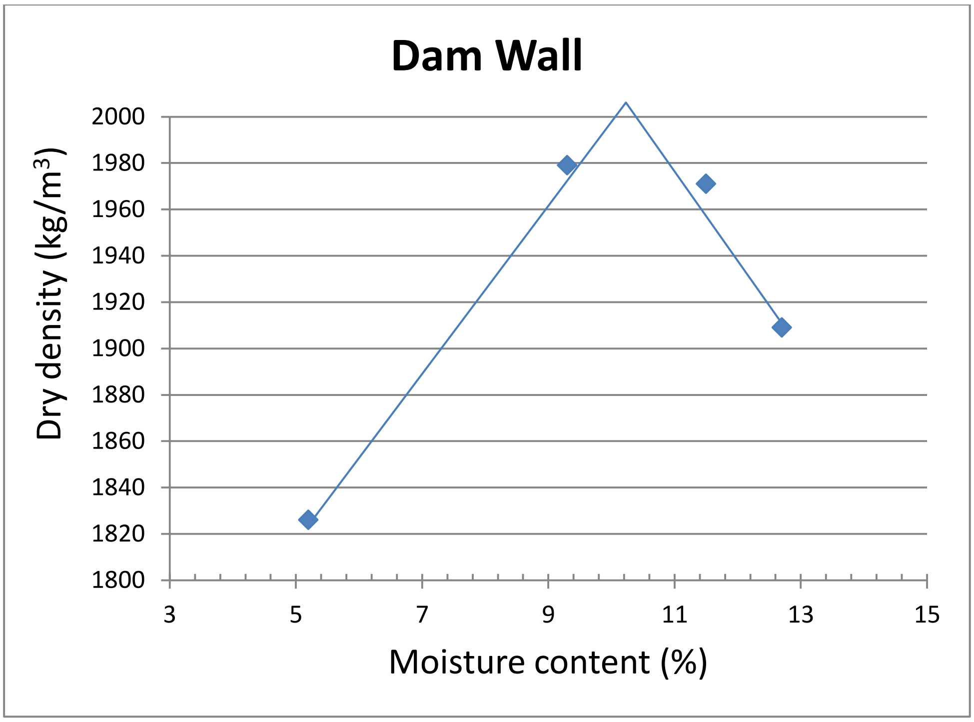
Density graphs



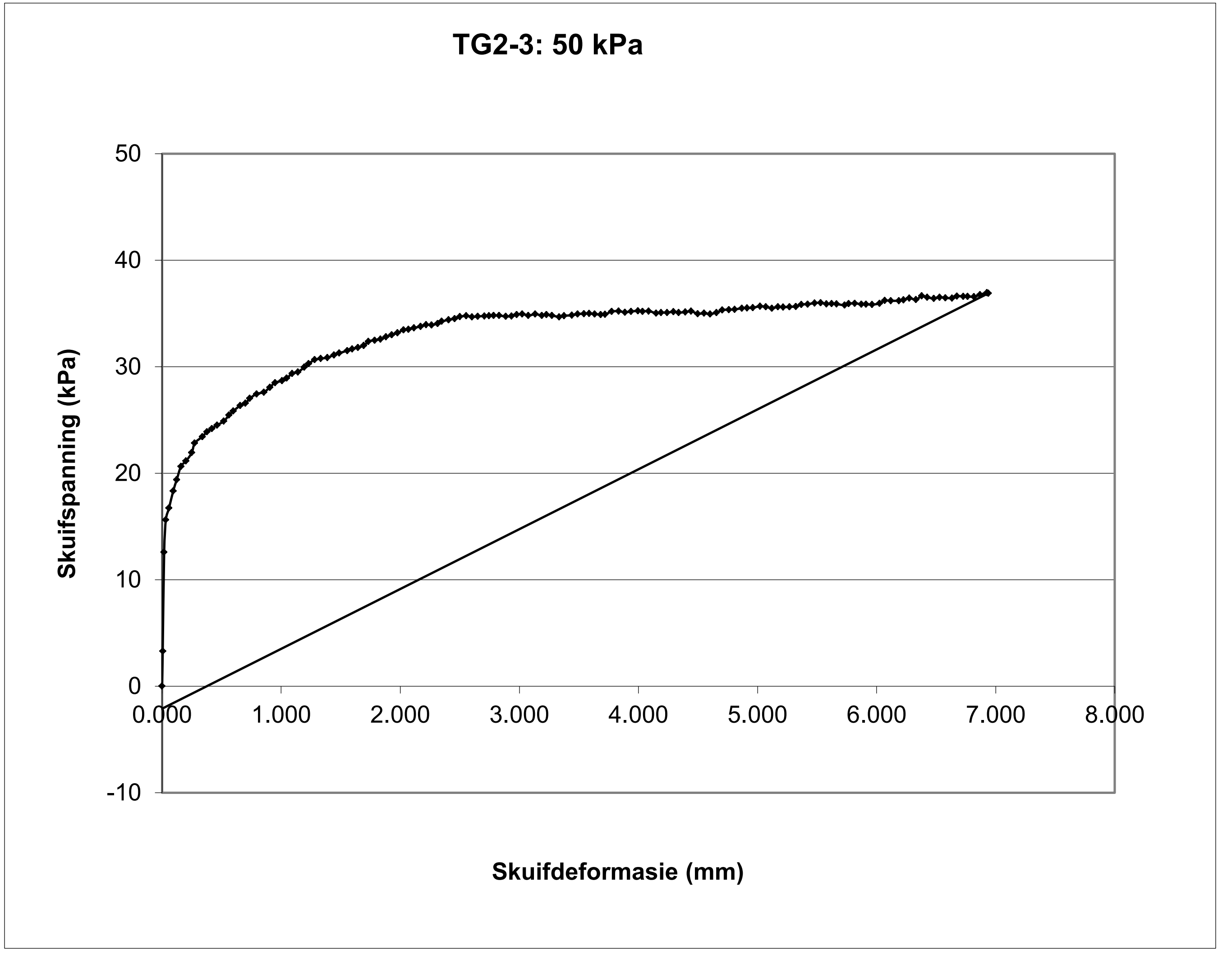


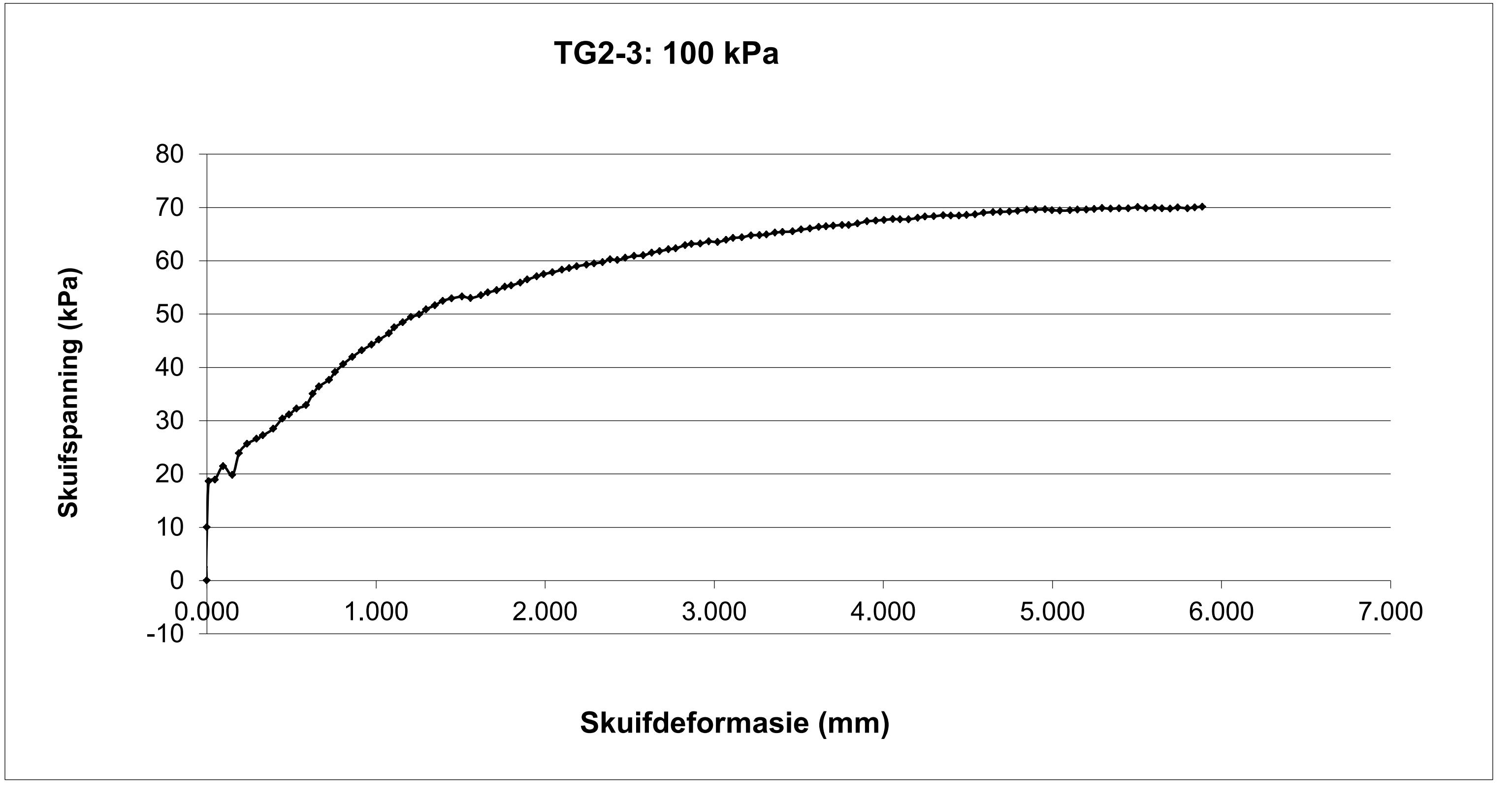

TG2-2
Dry density (kg/m3)
1960
1940
1920
1900
1880
1860
1840
1820
1800
1780
1760
2
4
6
8
10
12
Moisture content (%)

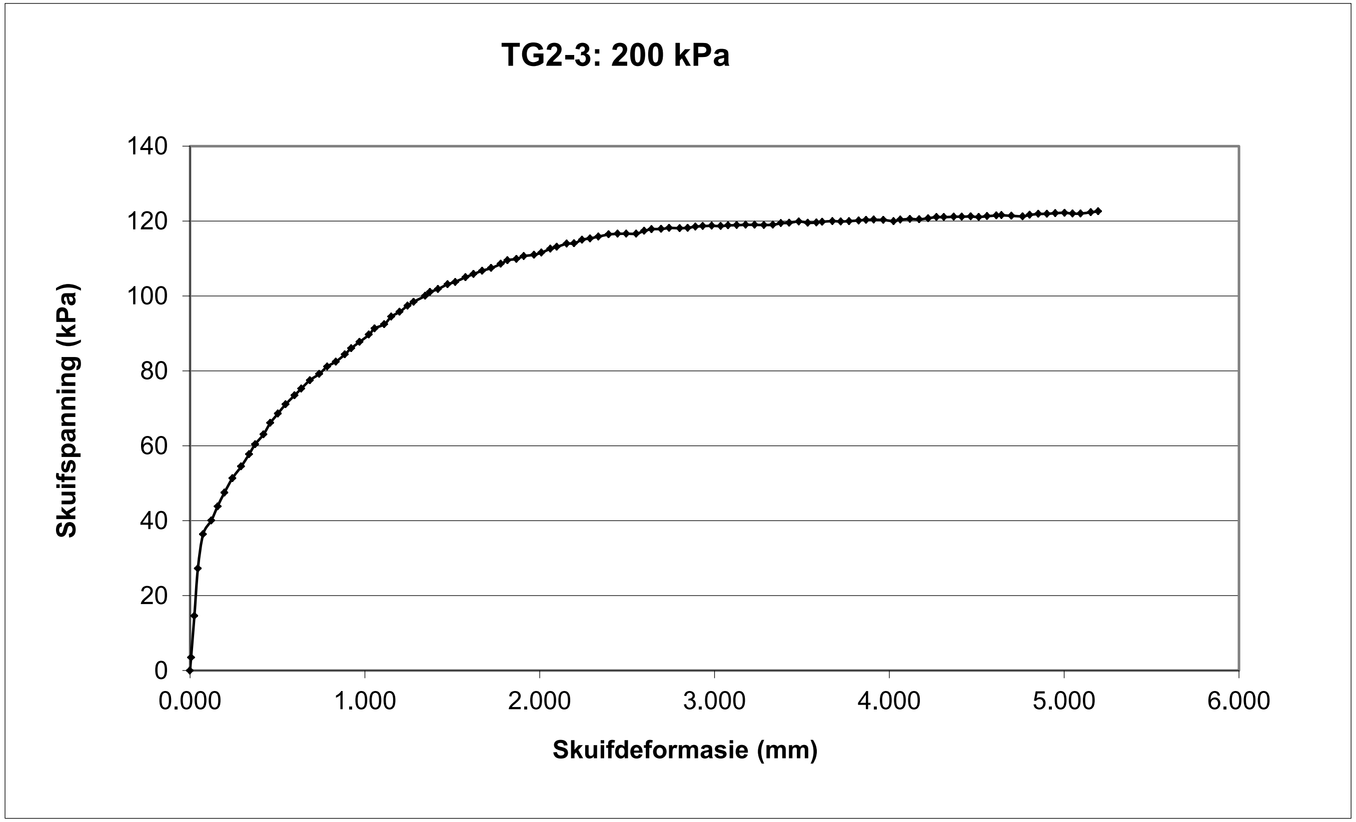

TG2-3
Dry density (kg/m3)
1720
1700
1680
1660
1640
1620
1600
1580
1560
1540
5
10
15
20
25
Moisture content (%)

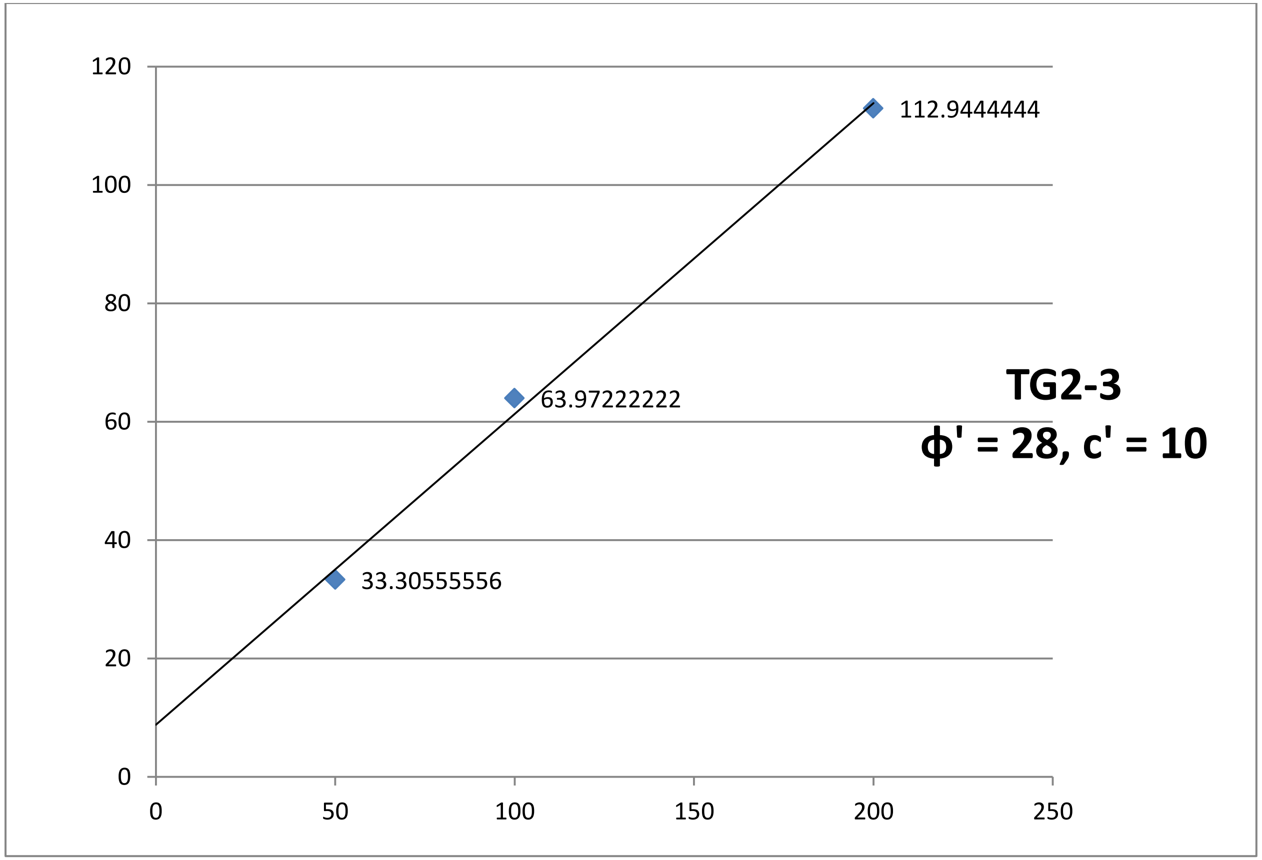

Dam Wall
Dry density (kg/m3)
2000
1980
1960
1940
1920
1900
1880
1860
1840
1820
1800
3
5
7
9
11
13
15
Moisture content (%)

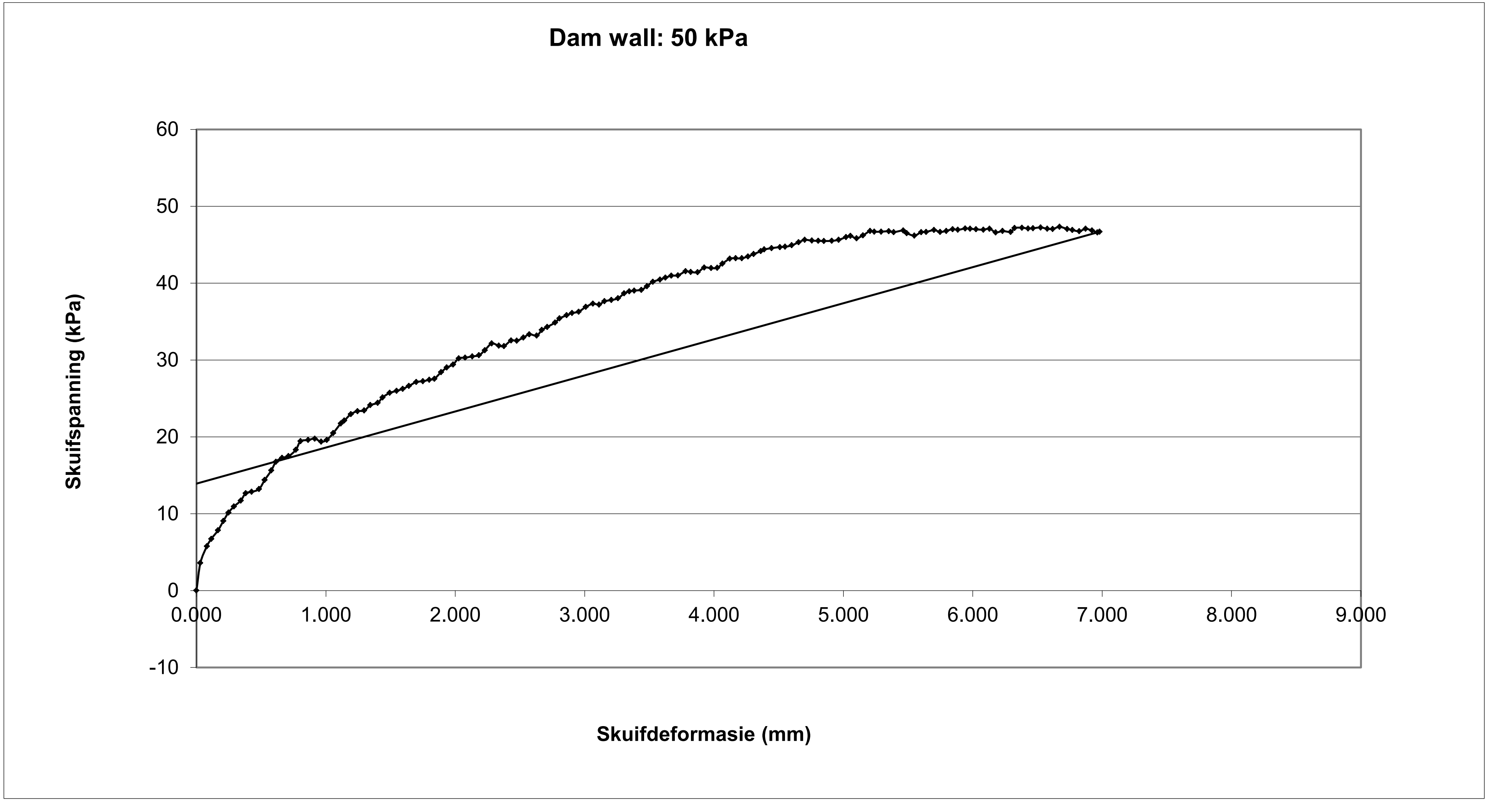
Appendix E – Shearbox results

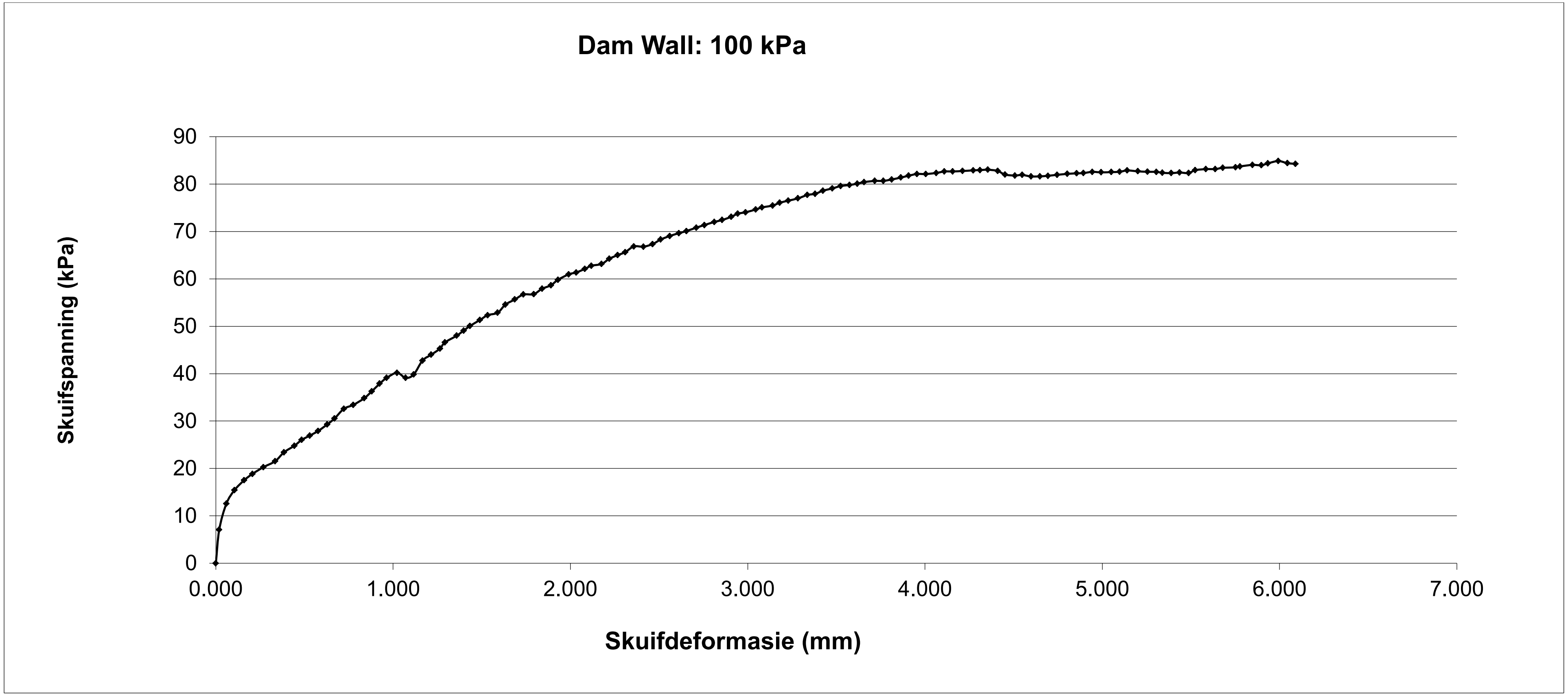


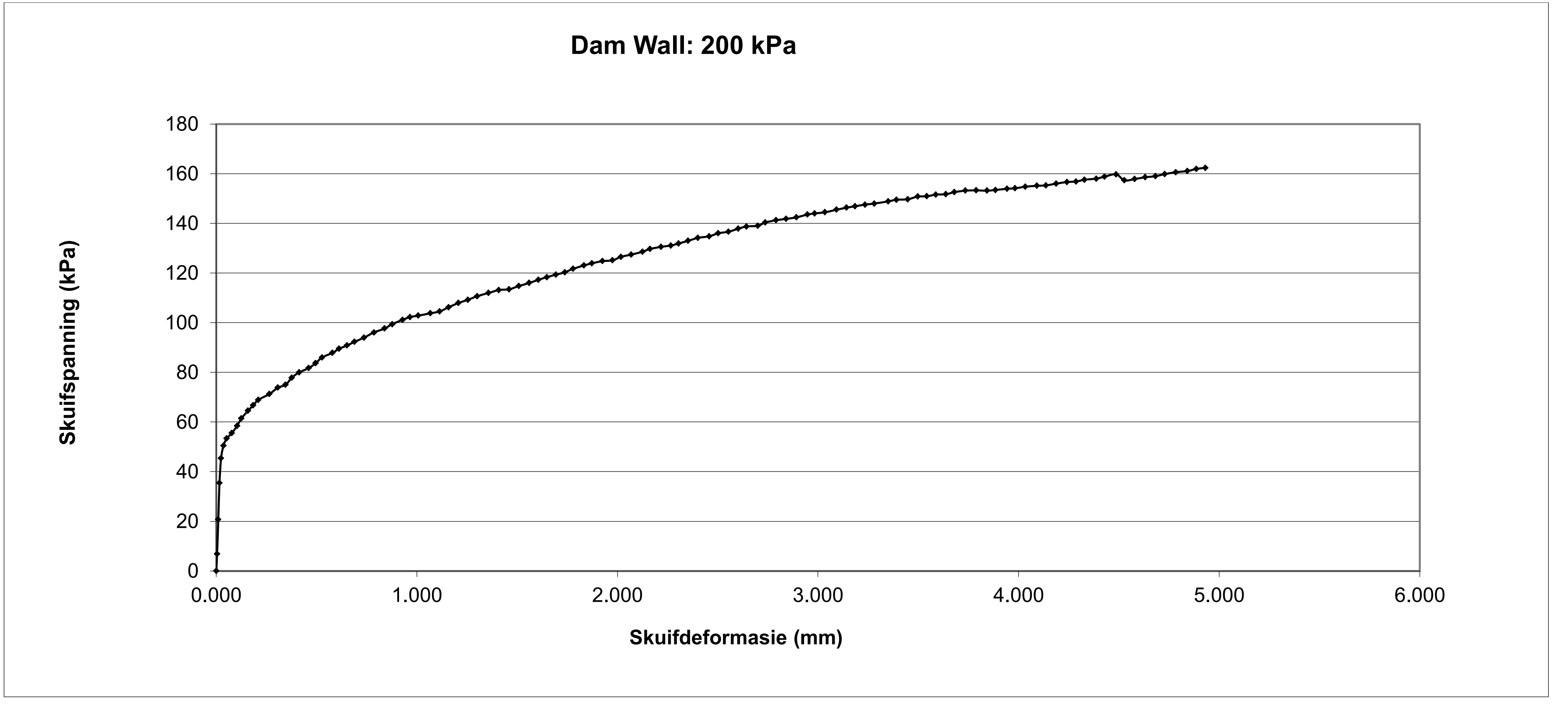

TG2-3: 100 kPa
80
70
60
50
40
30
20
10
0
-10
0.000
1.000
2.000
3.000
4.000
5.000
6.000
7.000
Skuifspanning (kPa)
Skuifdeformasie (mm)

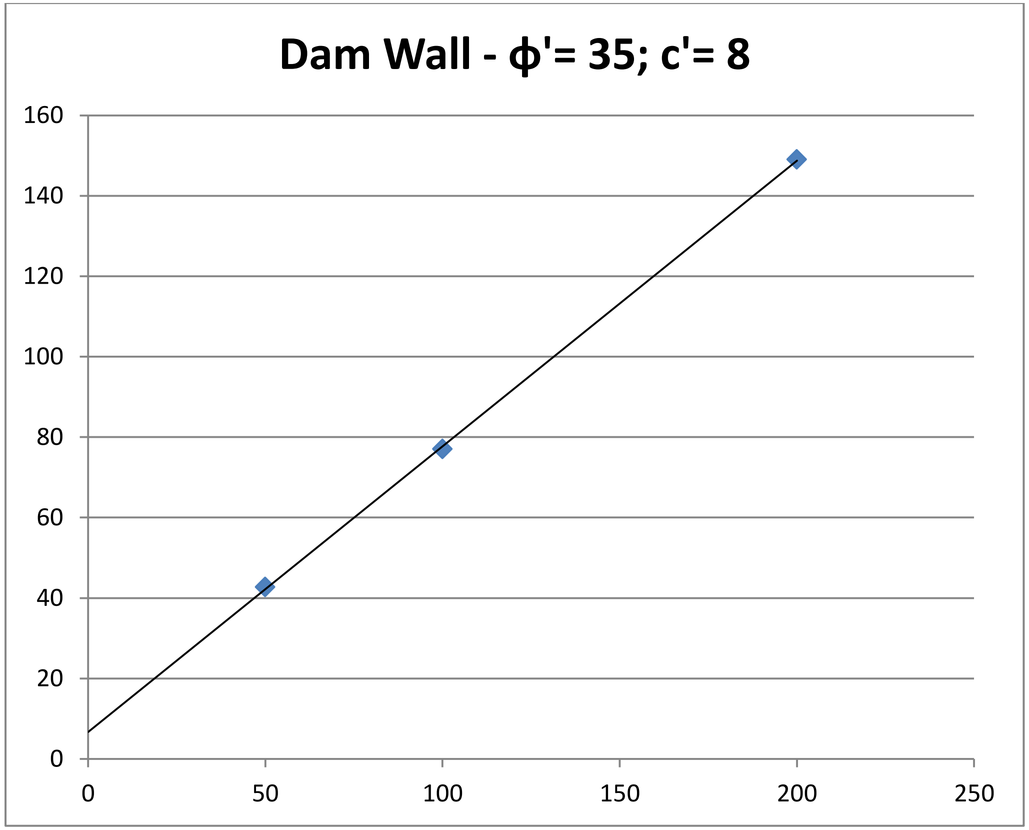

TG2-3: 200 kPa
Skuifspanning (kPa)
140
120
100
80
60
40
20
0
0.000
1.000
2.000
3.000
4.000
5.000
6.000
Skuifdeformasie (mm)


120
100
80
60
40
20
0
0
50
100
150
200
250
112.9444444
63.97222222
33.30555556
TG2-3
ϕ' = 28, c' = 10


Dam wall: 50 kPa
60
50
40
30
20
10
0
-10
0.000
1.000
2.000
3.000
4.000
5.000
6.000
7.000
8.000
9.000
Skuifspanning (kPa)
Skuifdeformasie (mm)


Dam Wall: 100 kPa
Skuifspanning (kPa)
Skuifdeformasie (mm)
0
10
20
30
40
50
60
70
80
90
0.000
1.000
2.000
3.000
4.000
5.000
6.000
7.000


Dam Wall: 200 kPa
Skuifspanning (kPa)
180
160
140
120
100
80
60
40
20
0
0.000
1.000
2.000
3.000
4.000
5.000
6.000
Skuifdeformasie (mm)


Dam Wall - ϕ'= 35; c'= 8
160
140
120
100
80
60
40
20
0
0
50
100
150
200
250

Appendix F – Volume Calculations

Volume of water

Height of water(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
7	21.01	12.12	73.52	42.44	23191.28	30
9	27.01	15.59	121.53	70.15	38336.61	30
11	33.01	19.05	181.55	104.79	57268.27	30
13	39.01	22.52	253.57	146.36	79986.26	30
15	45.01	25.98	337.60	194.86	106490.58	30
17	51.01	29.44	433.62	250.28	136781.24	30
18	54.02	31.18	486.14	280.59	153346.44	30
18.1	54.32	31.35	491.56	283.72	155055.02	30
18.2	54.62	31.52	497.00	286.86	156773.07	30
18.3	54.92	31.70	502.48	290.02	158500.58	30
18.4	55.22	31.87	507.99	293.20	160237.56	30
18.5	55.52	32.04	513.52	296.40	161984.01	30
18.6	55.82	32.22	519.09	299.61	163739.92	30
18.7	56.12	32.39	524.69	302.84	165505.30	30
18.8	56.42	32.56	530.31	306.09	167280.14	30

Height of water(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
14	42.01	30.02	294.08	210.16	100849.27	25
15	45.01	32.17	337.60	241.26	115770.84	25
16	48.01	34.31	384.11	274.50	131721.49	25
17	51.01	36.46	433.62	309.88	148701.22	25
17.6	52.82	37.74	464.77	332.14	159383.01	25
17.7	53.12	37.96	470.07	335.93	161199.32	25
17.8	53.42	38.17	475.40	339.73	163025.93	25
17.9	53.72	38.39	480.75	343.56	164862.83	25
18	54.02	38.60	486.14	347.41	166710.02	25

Height of water(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
14	42.01	38.46	294.08	269.25	112667.49	20
15	45.01	41.21	337.60	309.09	129337.68	20
16	48.01	43.96	384.11	351.68	147157.54	20
17	51.01	46.71	433.62	397.01	166127.07	20

Height of water(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
14	42.01	52.25	294.08	365.74	131965.13	15
15	45.01	55.98	337.60	419.86	151490.58	15
15.1	45.31	56.35	342.11	425.47	153517.19	15
15.2	45.61	56.73	346.66	431.13	155557.26	15
15.3	45.91	57.10	351.24	436.82	157610.80	15
15.4	46.21	57.47	355.84	442.55	159677.81	15
15.5	46.51	57.85	360.48	448.31	161758.28	15
15.6	46.81	58.22	365.15	454.12	163852.21	15
15.7	47.11	58.59	369.84	459.96	165959.62	15
15.8	47.41	58.97	374.57	465.83	168080.48	15

Height of water(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
13.00	39.01	73.73	253.57	479.22	146559.26	10
13.10	39.31	74.29	257.49	486.62	148822.69	10
13.20	39.61	74.86	261.44	494.08	151103.47	10
13.30	39.91	75.43	265.41	501.60	153401.59	10
13.40	40.21	76.00	269.42	509.17	155717.05	10
13.50	40.51	76.56	273.45	516.80	158049.86	10
13.60	40.81	77.13	277.52	524.48	160400.01	10
13.70	41.11	77.70	281.62	532.22	162767.50	10
13.80	41.41	78.26	285.74	540.02	165152.34	10
13.90	41.71	78.83	289.90	547.87	167554.53	10
14.00	42.01	79.40	294.08	555.79	169974.06	10

Height of water(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
10.00	30.01	114.30	150.04	571.50	144309.16	5
10.10	30.31	115.44	153.06	582.99	147209.78	5
10.20	30.61	116.59	156.10	594.59	150139.25	5
10.30	30.91	117.73	159.18	606.31	153097.59	5
10.40	31.21	118.87	162.29	618.14	156084.79	5
10.50	31.51	120.02	165.42	630.08	159100.85	5
10.60	31.81	121.16	168.59	642.14	162145.78	5
10.70	32.11	122.30	171.78	654.31	165219.56	5
10.80	32.41	123.44	175.01	666.60	168322.21	5

Height of dam(m)	Length of a (m)	Length of b(m)	V1 (m²)	V2 (m²)	Vtotal (m³)	Angle of slope behind upslope
5.00	15.00	145.00	37.51	724.98	152497.84	0(Restricted to a + b =160)
5.10	15.30	144.70	39.03	737.95	155394.75	0(Restricted to a + b =160)
5.20	15.60	144.40	40.57	750.86	158285.66	0(Restricted to a + b =160)
5.30	15.90	144.10	42.15	763.71	161170.57	0(Restricted to a + b =160)
5.40	16.20	143.80	43.75	776.49	164049.48	0(Restricted to a + b =160)
5.50	16.50	143.50	45.39	789.22	166922.39	0(Restricted to a + b =160)
5.60	16.80	143.20	47.05	801.89	169789.29	0(Restricted to a + b =160)

Height of water(m)	Length of a (m)	Length of b(m)	Length of c (m)	V1 (m²)	V2 (m²)	V3 (m²)	Vtotal (m³)	Angle of slope behind upslope
8	24.00691165	50	21.97981936	96.02764661	400	87.91927742	116789.38	0+20°
9	27.00777561	50	24.72729678	121.5349902	450	111.2728355	136561.57	0+20°
10	30.00863956	50	27.47477419	150.0431978	500	137.373871	157483.41	0+20°
10.1	30.30872596	50	27.74952194	153.0590661	505	140.1350858	159638.83	0+20°
10.2	30.60881236	50	28.02426968	156.104943	510	142.9237754	161805.74	0+20°
10.3	30.90889875	50	28.29901742	159.1808286	515	145.7399397	163984.15	0+20°
10.4	31.20898515	50	28.57376516	162.2867228	520	148.5835788	166174.06	0+20°
10.5	31.50907154	50	28.8485129	165.4226256	525	151.4546927	168375.46	0+20°
10.6	31.80915794	50	29.12326065	168.5885371	530	154.3532814	170588.36	0+20°

Height of water(m)	Length of a (m)	Length of b(m)	Length of c (m)	V1 (m²)	V2 (m²)	V3 (m²)	Vtotal (m³)	Angle of slope behind upslope
8	24.00691165	50	29.85640646	96.02764661	400	119.4256258	123090.65	0+15°
9	27.00777561	50	33.58845727	121.5349902	450	151.1480577	144536.61	0+15°
10	30.00863956	50	37.32050808	150.0431978	500	186.6025404	167329.15	0+15°

Embankment calculations

Height of water(m)	Angle of slope	total dam height - H (m)	BC(m)	AC(m)	CD(m)	DE=BH(m)	EF(m)	HF(m)	Angle @ F	d= HI(m)	GI(m)	FG(m)	Embankment width(m)
18.7	20.0	21.7	21.7	65.2	23.7	4.0	1.5	46.9	110.0	383.4	342.8	124.8	412.1
18.0	20.0	21.0	21.0	63.1	23.0	4.0	1.5	45.4	110.0	371.4	332.1	120.9	399.3
17.0	20.0	20.0	20.0	60.1	21.9	4.0	1.5	43.3	110.0	354.3	316.8	115.3	380.9
15.7	20.0	18.7	18.7	56.2	20.5	4.0	1.5	40.6	110.0	332.1	296.9	108.1	357.1
13.9	20.0	16.9	16.9	50.8	18.5	4.0	1.5	36.8	110.0	301.2	269.4	98.0	324.2
10.8	20.0	13.8	13.8	41.5	15.1	4.0	1.5	30.4	110.0	248.2	221.9	80.8	267.4
10.4	20.0	13.4	13.4	40.3	14.7	4.0	1.5	29.5	110.0	241.3	215.8	78.5	260.1
5.5	20.0	8.5	8.5	25.6	9.3	4.0	1.5	19.3	110.0	157.4	140.8	51.2	170.3
10.0	20.0	13.0	13.0	39.1	14.2	4.0	1.5	28.7	110.0	234.5	209.7	76.3	252.7
									90.0				
Height of water(m)	Angle of slope	total dam height - H (m)	BC(m)	AC(m)	CD(m)	DE=BH(m)	EF(m)	HF(m)	Angle @ F	d= HI(m)	GI(m)	FG(m)	Embankment width(m)
18.7	15.0	21.7	21.7	65.2	17.5	4.0	1.1	40.3	105.0	193.4	172.9	46.3	242.1
18.0	15.0	21.0	21.0	63.1	16.9	4.0	1.1	39.0	105.0	187.3	167.5	44.9	234.6
17.0	15.0	20.0	20.0	60.1	16.1	4.0	1.1	37.2	105.0	178.6	159.7	42.8	223.8
15.7	15.0	18.7	18.7	56.2	15.1	4.0	1.1	34.8	105.0	167.3	149.6	40.1	209.8
13.9	15.0	16.9	16.9	50.8	13.6	4.0	1.1	31.6	105.0	151.7	135.7	36.4	190.5
10.8	15.0	13.8	13.8	41.5	11.1	4.0	1.1	26.0	105.0	124.8	111.6	29.9	157.1
10.4	15.0	13.4	13.4	40.3	10.8	4.0	1.1	25.3	105.0	121.4	108.5	29.1	152.8
5.5	15.0	8.5	8.5	25.6	6.8	4.0	1.1	16.4	105.0	78.9	70.5	18.9	100.1
10.0	15.0	13.0	13.0	39.1	10.5	4.0	1.1	24.5	105.0	117.9	105.4	28.2	148.5
									90.0				
Height of water(m)	Angle of slope	total dam height - H (m)	BC(m)	AC(m)	CD(m)	DE=BH(m)	EF(m)	HF(m)	Angle @ F	d= HI(m)	GI(m)	FG(m)	Embankment width(m)
18.7	10.0	21.7	21.7	65.2	11.5	4.0	0.7	33.9	100.0	116.9	104.5	18.4	173.7
18.0	10.0	21.0	21.0	63.1	11.1	4.0	0.7	32.8	100.0	113.2	101.2	17.8	168.3
17.0	10.0	20.0	20.0	60.1	10.6	4.0	0.7	31.3	100.0	107.9	96.5	17.0	160.6
15.7	10.0	18.7	18.7	56.2	9.9	4.0	0.7	29.3	100.0	101.1	90.4	15.9	150.6
13.9	10.0	16.9	16.9	50.8	9.0	4.0	0.7	26.6	100.0	91.6	81.9	14.4	136.7
10.8	10.0	13.8	13.8	41.5	7.3	4.0	0.7	21.8	100.0	75.2	67.3	11.9	112.7
10.4	10.0	13.4	13.4	40.3	7.1	4.0	0.7	21.2	100.0	73.1	65.4	11.5	109.7
5.5	10.0	8.5	8.5	25.6	4.5	4.0	0.7	13.7	100.0	47.3	42.3	7.5	71.8
10.0	10.0	13.0	13.0	39.1	6.9	4.0	0.7	20.6	100.0	71.0	63.5	11.2	106.6
									90.0				
Height of water(m)	Angle of slope	total dam height - H (m)	BC(m)	AC(m)	CD(m)	DE=BH(m)	EF(m)	HF(m)	Angle @ F	d= HI(m)	GI(m)	FG(m)	Embankment width(m)
18.7	5.0	21.7	21.7	65.2	5.7	4.0	0.3	27.8	95.0	75.1	67.2	5.9	136.4
18.0	5.0	21.0	21.0	63.1	5.5	4.0	0.3	26.9	95.0	72.7	65.0	5.7	132.2
17.0	5.0	20.0	20.0	60.1	5.3	4.0	0.3	25.6	95.0	69.3	62.0	5.4	126.1
15.7	5.0	18.7	18.7	56.2	4.9	4.0	0.3	24.0	95.0	64.9	58.0	5.1	118.2
13.9	5.0	16.9	16.9	50.8	4.4	4.0	0.3	21.7	95.0	58.7	52.5	4.6	107.3
10.8	5.0	13.8	13.8	41.5	3.6	4.0	0.3	17.8	95.0	48.1	43.0	3.8	88.5
10.4	5.0	13.4	13.4	40.3	3.5	4.0	0.3	17.3	95.0	46.7	41.8	3.7	86.1
5.5	5.0	8.5	8.5	25.6	2.2	4.0	0.3	11.1	95.0	30.0	26.8	2.3	56.4
10.0	5.0	13.0	13.0	39.1	3.4	4.0	0.3	16.8	95.0	45.4	40.6	3.5	83.7

Soil Balance

Height of water(m)	Area of embankment(m²)	Volume of embankment(m³)	Angle of slope	Available soil from embankment excavation(m³)	Available Soil from Water excavation	Excess soil
18.7	9706.3	1941266.5	20.0	1737770.6	0.0	1737770.6
18	9111.7	1822333.4	20.0	1631322.4	16545.9	-174465.1
17	8294.7	1658949.1	20.0	1485089.2	46550.0	-127309.9
15.7	7290.1	1458017.4	20.0	1305250.1	106222.4	-46545.0
13.9	6006.0	1201205.9	20.0	1075397.0	249207.8	123398.9
10.8	4085.9	817179.0	20.0	731682.9	732718.9	647222.7
10.4	3865.0	772996.7	20.0	692138.6	326729.9	245871.7
5.5	1656.8	331368.9	20.0	296870.1	1188815.4	1154316.6
10	3650.2	730041.7	20.0	653692.731	402901.6319	326552.6

Height of water(m)	Area of embankment(m²)	Volume of embankment(m³)	Angle of slope	Soil available	Available Soil from Water excavation	Excess soil
18.7	4916.2	983238.1	15.0	1162831.3	0.0	179593.2
18	4614.9	922989.6	15.0	1091601.3	16545.9	185157.6
17	4201.1	840223.3	15.0	993749.2	46550.0	200075.9
15.7	3692.2	738436.5	15.0	873409.6	106222.4	241195.6
13.9	3041.7	608342.3	15.0	719603.2	249207.8	360468.7
10.8	2069.0	413804.2	15.0	489606.5	732718.9	808521.2
10.4	1957.1	391422.5	15.0	463145.3	326729.9	398452.7
5.5	838.5	167705.2	15.0	198651.0	1188815.4	1219761.1
10	1848.3	369662.7	15.0	437419.3	402901.6319	470658.3

Height of water(m)	Area of embankment(m²)	Volume of embankment(m³)	Angle of slope	Soil available	Available Soil from Water excavation	Excess soil
18.7	2989.0	597807.4	10.0	875374.8	0.0	277567.4
18	2805.8	561168.8	10.0	821753.2	16545.9	277130.3
17	2554.2	510836.4	10.0	748090.5	46550.0	283804.1
15.7	2244.7	448937.2	10.0	657499.3	106222.4	314784.6
13.9	1849.1	369823.5	10.0	541714.4	249207.8	421098.7
10.8	1257.6	251519.8	10.0	368573.8	732718.9	849772.9
10.4	1189.5	237908.9	10.0	348654.0	326729.9	437475.0
5.5	509.3	101860.6	10.0	149543.7	1188815.4	1236498.5
10	1123.4	224676.2	10.0	329287.5	402901.6	507513.0

Height of water(m)	Area of embankment(m²)	Volume of embankment(m³)	Angle of slope	Soil available	Available Soil from Water excavation	Excess soil
18.7	1936.4	387276.5	5.0	704754.4	0.0	317477.9
18	1817.7	363534.1	5.0	661584.3	16545.9	314596.1
17	1654.6	330917.9	5.0	602279.3	46550.0	317911.4
15.7	1454.0	290806.2	5.0	529345.3	106222.4	344761.6
13.9	1197.7	239539.2	5.0	436128.2	249207.8	445796.8
10.8	814.4	162876.4	5.0	296734.6	732718.9	866577.1
10.4	770.3	154056.4	5.0	280697.4	326729.9	453371.0
5.5	329.5	65894.8	5.0	120395.9	1188815.4	1243316.5
10	727.4065933	145481.3187	5.0	265105.675	402901.6319	522526.0

Appendix G – Slope analysis

Stability of downstream slope-After construction

File Information

File Version: 8.16
Revision Number: 48
Date: 11/5/2016
Time: 6:55:27 PM
Tool Version: 8.16.0.12829
File Name: Downsstream slope 13.4 thesis.gsz
Directory: C:\Users\Ian\Documents\
Last Solved Date: 11/5/2016
Last Solved Time: 6:55:45 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Stability of downstream slope-After construction

Kind: SLOPE/W
Method: Morgenstern-Price
Settings
- Side Function
  - Interslice force function option: Half-Sine
- PWP Conditions Source: (none)

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Resisting Side Maximum Convex Angle: 1 °
- Driving Side Maximum Convex Angle: 5 °
- Optimize Critical Slip Surface Location: No
- Tension Crack
  - Tension Crack Option: (none)

F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 m
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Embankment
Model: Mohr-Coulomb
Unit Weight: 21.6 kN/m³
Cohesion': 8 kPa
Phi': 35 °
Phi-B: 0 °

Slope
Model: Mohr-Coulomb
Unit Weight: 19.8 kN/m³
Cohesion': 8 kPa
Phi': 29 °
Phi-B: 0 °

Core
Model: Mohr-Coulomb
Unit Weight: 19.8 kN/m³
Cohesion': 10 kPa
Phi': 29 °
Phi-B: 0 °

Slip Surface Entry and Exit
Left Projection: Range
Left-Zone Left Coordinate: (50.38856, 53.4) m
Left-Zone Right Coordinate: (54, 53.4) m
Left-Zone Increment: 4
Right Projection: Point
Right Coordinate: (119.39987, 20.80007) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits
Left Coordinate: (0, 40) m
Right Coordinate: (129.7, 18.9) m

Points

	X (m)	Y (m)
Point 1	10	40
Point 2	0	50.4
Point 3	0	40
Point 4	41.2	50.4
Point 5	50.2	53.4
Point 6	54.2	53.4
Point 7	119.6	20.7
Point 8	50.8	50.9
Point 9	53.8	50.9
Point 10	42.2	33.6
Point 11	50.8	32.1
Point 12	51.3	30.6
Point 13	53.3	30.6
Point 14	53.8	31.6
Point 15	64.4	29.7
Point 16	129.7	18.9
Point 17	0	0
Point 18	129.7	0
Point 19	42.27491	33.75069

Regions

	Material	Points	Area (m^2)
Region 1	Slope	3,17,18,16,7,15,14,13,12,11,10,19,1	3,878.5
Region 2	Core	8,19,10,11,12,13,14,15,9	243.4
Region 3	Embankment	1,4,5,6,7,15,9,8,19	992.11

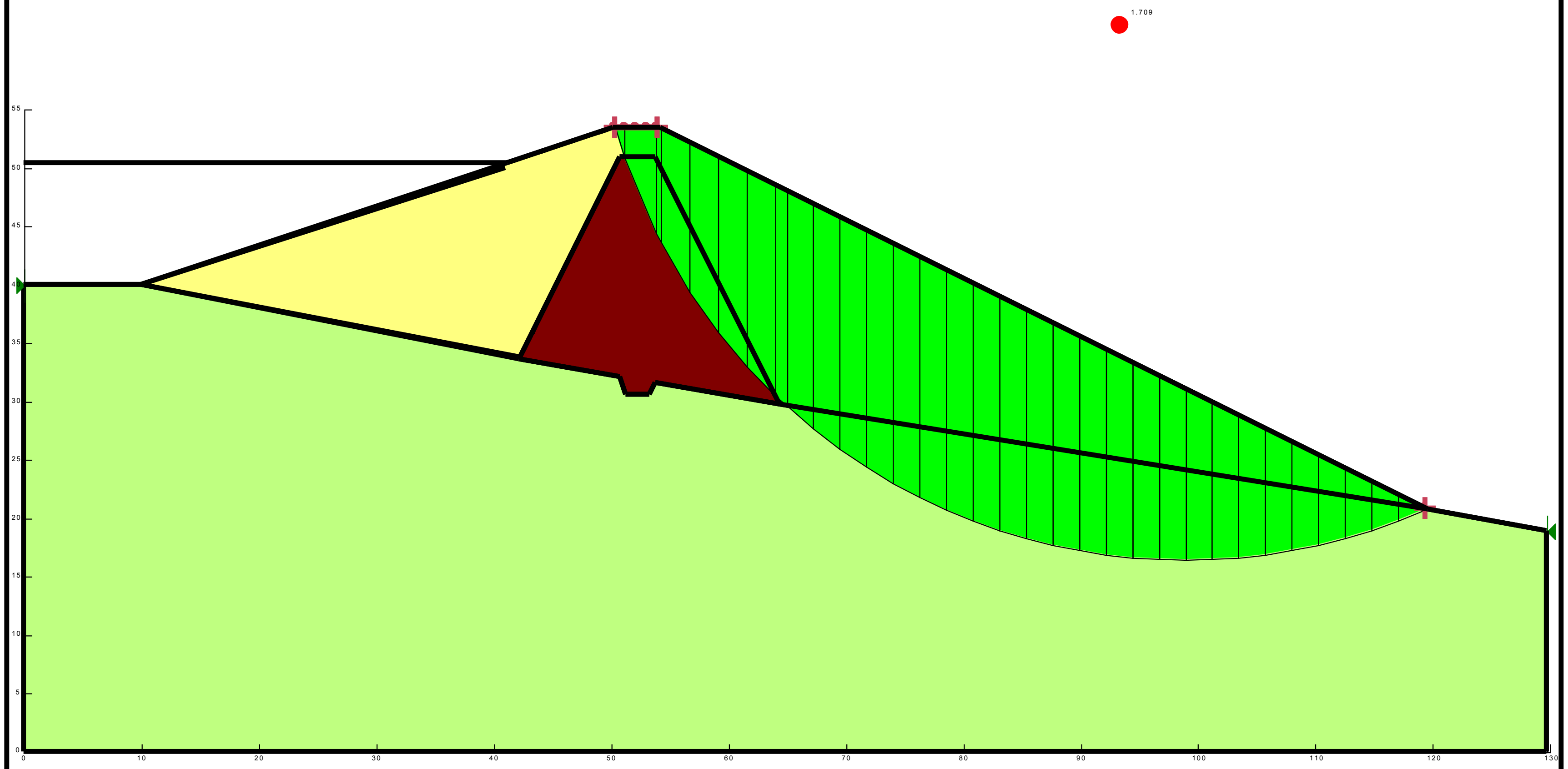
Current Slip Surface

Slip Surface: 4
F of S: 1.709
Volume: 988.97645 m^3
Weight: 20,722.178 kN
Resisting Moment: 624,488.28 kN-m
Activating Moment: 365,409.44 kN-m
Resisting Force: 10,848.13 kN
Activating Force: 6,347.0645 kN
F of S Rank (Analysis): 15 of 25 slip surfaces
F of S Rank (Query): 15 of 25 slip surfaces
Exit: (119.39987, 20.800066) m
Entry: (50.38856, 53.4) m
Radius: 50.30442 m
Center: (98.893332, 66.734984) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	50.767383	52.15	0	4.8419016	3.390336	8
Slice 2	52.473103	47.669329	0	53.334727	29.563922	10
Slice 3	54	44.042983	0	97.097896	53.822243	10
Slice 4	55.425027	41.534279	0	122.74066	68.036259	10
Slice 5	57.87508	37.691158	0	160.68342	89.068272	10
Slice 6	60.325133	34.495562	0	192.35933	106.62652	10
Slice 7	62.775187	31.764815	0	220.34411	122.13873	10
Slice 8	64.47381	30.055161	0	230.34526	161.28948	8
Slice 9	66.079522	28.635691	0	251.88787	139.62373	8
Slice 10	68.343753	26.794698	0	269.09388	149.16117	8
Slice 11	70.607984	25.158561	0	285.75516	158.39667	8
Slice 12	72.872216	23.703764	0	302.10091	167.45727	8
Slice 13	75.136447	22.412357	0	318.21452	176.38919	8
Slice 14	77.400678	21.270356	0	334.0441	185.16367	8
Slice 15	79.66491	20.266701	0	349.40381	193.67769	8
Slice 16	81.929141	19.392565	0	363.97042	201.7521	8
Slice 17	84.193372	18.640863	0	377.27951	209.12944	8
Slice 18	86.457604	18.005915	0	388.72552	215.47407	8
Slice 19	88.721835	17.483192	0	397.57087	220.37713	8
Slice 20	90.986066	17.069144	0	402.96934	223.36955	8
Slice 21	93.250298	16.761062	0	404.00871	223.94569	8
Slice 22	95.514529	16.556992	0	399.77574	221.59931	8

Slice 23	97.77876	16.455663	0	389.44273	215.87163	8
Slice 24	100.04299	16.456453	0	372.36928	206.40766	8
Slice 25	102.30722	16.559368	0	348.20622	193.01386	8
Slice 26	104.57145	16.765039	0	316.98258	175.70631	8
Slice 27	106.83569	17.074746	0	279.15443	154.73783	8
Slice 28	109.09992	17.490456	0	235.59821	130.59422	8
Slice 29	111.36415	18.014888	0	187.54107	103.95571	8
Slice 30	113.62838	18.651608	0	136.43559	75.627483	8
Slice 31	115.89261	19.40516	0	83.800295	46.451262	8
Slice 32	118.15684	20.281246	0	31.055733	17.214474	8
Slice 33	119.34441	20.77539	0	3.9574758	2.7710544	8



Stability of downstream slope-After construction

Downsstream slope 13.4 thesis.gsz

11/5/2016 1:496

Stability of downstream slope-Full supply

File Information

File Version: 8.16
Revision Number: 53
Date: 11/5/2016
Time: 7:23:46 PM
Tool Version: 8.16.0.12829
File Name: Downsstream slope 13.4 thesis.gsz
Directory: C:\Users\Ian\Documents\
Last Solved Date: 11/5/2016
Last Solved Time: 7:23:56 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Stability of downstream slope-Full supply

Kind: SLOPE/W
Method: Morgenstern-Price
Settings
  Side Function
    Interslice force function option: Half-Sine
  PWP Conditions Source: Piezometric Line with Ru
  Apply Phreatic Correction: No
Slip Surface
  Direction of movement: Left to Right
  Use Passive Mode: No
  Slip Surface Option: Entry and Exit
  Critical slip surfaces saved: 1
  Resisting Side Maximum Convex Angle: 1 °
  Driving Side Maximum Convex Angle: 5 °
  Optimize Critical Slip Surface Location: No
  Tension Crack
    Tension Crack Option: (none)

F of S Distribution
    F of S Calculation Option: Constant
Advanced
    Number of Slices: 30
    F of S Tolerance: 0.001
    Minimum Slip Surface Depth: 0.1 m
    Search Method: Root Finder
    Tolerable difference between starting and converged F of S: 3
    Maximum iterations to calculate converged lambda: 20
    Max Absolute Lambda: 2

Materials

Embankment

Model: Mohr-Coulomb
Unit Weight: 22.1 kN/m³
Cohesion': 8 kPa
Phi': 35 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1
    Include Ru in PWP: No

Slope

Model: Mohr-Coulomb
Unit Weight: 22.1 kN/m³
Cohesion': 8 kPa
Phi': 29 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1
    Include Ru in PWP: No

Core

Model: Mohr-Coulomb
Unit Weight: 20.5 kN/m³
Cohesion': 10 kPa
Phi': 29 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1
    Include Ru in PWP: No

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (50.38856, 53.4) m
Left-Zone Right Coordinate: (54, 53.4) m
Left-Zone Increment: 4
Right Projection: Point

Right Coordinate: (119.08876, 20.95562) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 40) m
Right Coordinate: (129.7, 18.9) m

Piezometric Lines

Piezometric Line 1

Coordinates

	X (m)	Y (m)
Coordinate 1	0	50.4
Coordinate 2	41.2	50.4
Coordinate 3	48	50
Coordinate 4	50	49.2907
Coordinate 5	62	34.5
Coordinate 6	91	25.36304

Points

	X (m)	Y (m)
Point 1	10	40
Point 2	0	50.4
Point 3	0	40
Point 4	41.2	50.4
Point 5	50.2	53.4
Point 6	54.2	53.4
Point 7	119.6	20.7
Point 8	50.8	50.9
Point 9	53.8	50.9
Point 10	42.2	33.6
Point 11	50.8	32.1
Point 12	51.3	30.6
Point 13	53.3	30.6
Point 14	53.8	31.6
Point 15	64.4	29.7
Point 16	129.7	18.9
Point 17	0	0
Point 18	129.7	0
Point 19	42.27491	33.75069

Regions

	Material	Points	Area (m²)
Region 1	Slope	3,17,18,16,7,15,14,13,12,11,10,19,1	3,878.5
Region 2	Core	8,19,10,11,12,13,14,15,9	243.4
Region 3	Embankment	1,4,5,6,7,15,9,8,19	992.11

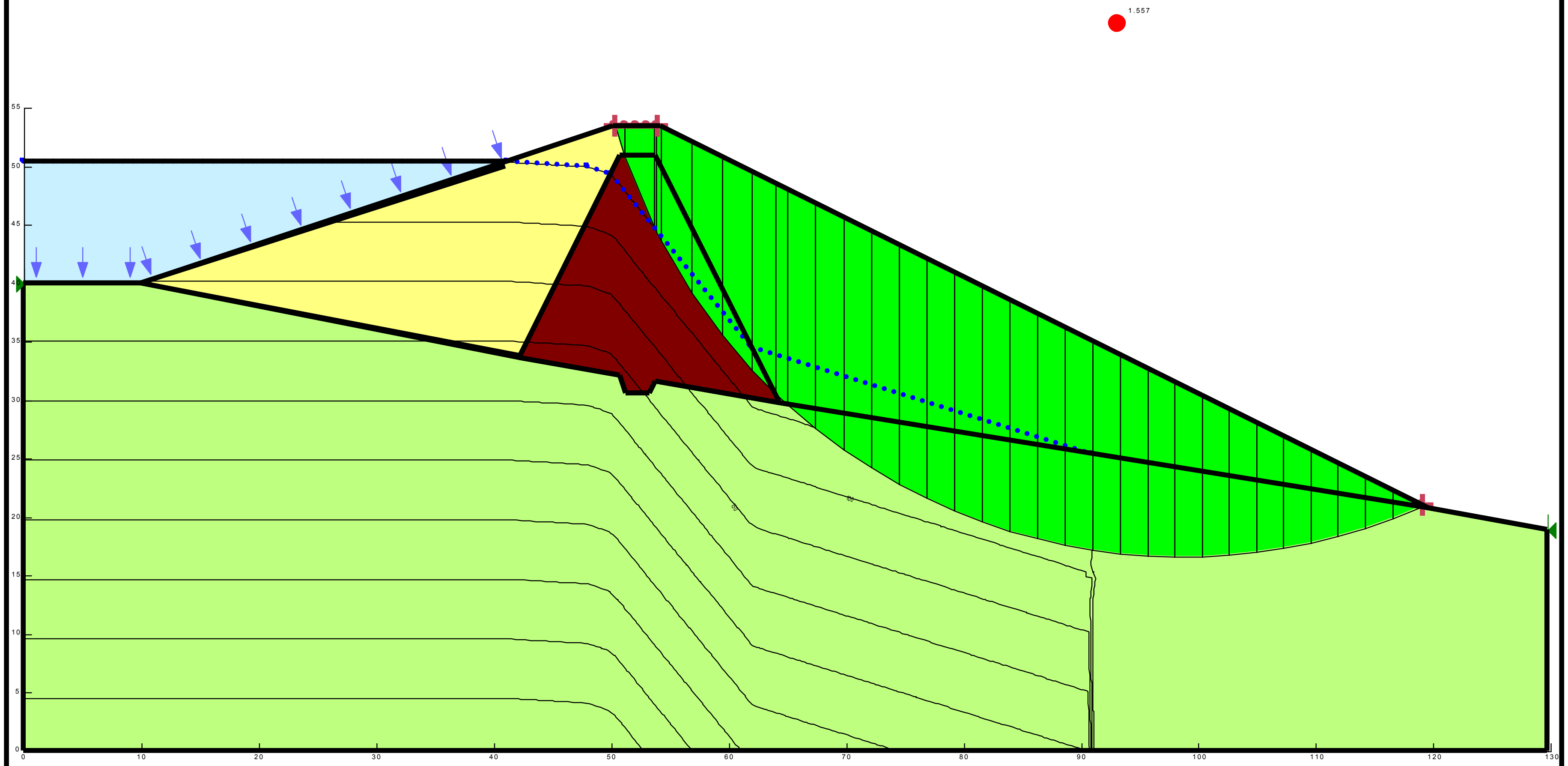
Current Slip Surface

Slip Surface: 4
F of S: 1.557
Volume: 980.3823 m^3
Weight: 21,587.373 kN
Resisting Moment: 581,544.35 kN-m
Activating Moment: 373,408.3 kN-m
Resisting Force: 10,190.598 kN
Activating Force: 6,542.7584 kN
F of S Rank (Analysis): 6 of 25 slip surfaces
F of S Rank (Query): 6 of 25 slip surfaces
Exit: (119.08876, 20.95562) m
Entry: (50.38856, 53.4) m
Radius: 50.071537 m
Center: (98.668562, 66.674045) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	50.767572	52.15	-37.319336	4.2609846	2.9835736	8
Slice 2	52.375398	47.874149	-14.820986	50.399737	27.93703	10
Slice 3	53.702106	44.648443	0.77667512	89.831162	49.363708	10
Slice 4	54	44.053646	3.0089921	98.320204	52.831867	10
Slice 5	55.5	41.435184	10.556693	127.93237	65.062403	10
Slice 6	58.1	37.409512	18.608445	170.31396	84.091742	10
Slice 7	60.7	34.092422	19.711124	203.72705	102.00169	10
Slice 8	62.977962	31.582819	25.587024	230.9005	113.80712	10
Slice 9	64.484632	30.094076	35.531714	240.261	143.35299	8
Slice 10	66.194551	28.592717	44.972122	264.57346	121.72701	8

Slice 11	68.556972	26.695748	56.276122	285.08055	126.82836	8
Slice 12	70.919394	25.019176	65.418691	304.07959	132.2919	8
Slice 13	73.281815	23.53713	72.653551	322.09709	138.26881	8
Slice 14	75.644237	22.23004	78.172613	339.41981	144.81168	8
Slice 15	78.006658	21.082787	82.124147	356.12622	151.88183	8
Slice 16	80.36908	20.083517	84.624423	372.09819	159.34931	8
Slice 17	82.731502	19.222845	85.765455	387.02247	166.98949	8
Slice 18	85.093923	18.493322	85.620319	400.38983	174.47959	8
Slice 19	87.456345	17.889046	84.246883	411.49928	181.39897	8
Slice 20	89.818778	17.405395	81.690447	419.47545	187.23729	8
Slice 21	92.15852	17.041264	0	432.05499	239.49199	8
Slice 22	94.47556	16.791924	0	430.59561	238.68304	8
Slice 23	96.7926	16.651094	0	422.4172	234.14968	8
Slice 24	99.109641	16.617856	0	406.69538	225.43493	8
Slice 25	101.42668	16.691994	0	382.91203	212.2516	8
Slice 26	103.74372	16.873989	0	350.96662	194.54398	8
Slice 27	106.06076	17.165034	0	311.24777	172.52746	8
Slice 28	108.3778	17.567071	0	264.6419	146.6934	8
Slice 29	110.69484	18.082862	0	212.4668	117.77227	8
Slice 30	113.01188	18.716086	0	156.33487	86.657832	8
Slice 31	115.32892	19.471482	0	97.968813	54.305	8
Slice 32	117.64596	20.355046	0	39.005164	21.620915	8
Slice 33	118.94662	20.892662	0	6.6213107	4.6362917	8



Stability of downstream slope-Full supply
Downsstream slope 13.4 thesis.gsz
11/5/2016 1:496

Stability of upstream slope-After construction

File Information

File Version: 8.16
Revision Number: 49
Date: 11/5/2016
Time: 7:09:26 PM
Tool Version: 8.16.0.12829
File Name: Downsstream slope 13.4 thesis.gsz
Directory: C:\Users\Ian\Documents\
Last Solved Date: 11/5/2016
Last Solved Time: 7:09:39 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Stability of upstream slope-After construction

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - PWP Conditions Source: (none)
- Slip Surface
  - Direction of movement: Right to Left
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Resisting Side Maximum Convex Angle: 1 °
  - Driving Side Maximum Convex Angle: 5 °
  - Optimize Critical Slip Surface Location: No
  - Tension Crack
    - Tension Crack Option: (none)
- F of S Distribution

F of S Calculation Option: Constant

Advanced

Number of Slices: 30

F of S Tolerance: 0.001

Minimum Slip Surface Depth: 0.1 m

Search Method: Root Finder

Tolerable difference between starting and converged F of S: 3

Maximum iterations to calculate converged lambda: 20

Max Absolute Lambda: 2

Materials

Embankment

Model: Mohr-Coulomb

Unit Weight: 21.6 kN/m³

Cohesion': 8 kPa

Phi': 35 °

Phi-B: 0 °

Slope

Model: Mohr-Coulomb

Unit Weight: 19.8 kN/m³

Cohesion': 8 kPa

Phi': 29 °

Phi-B: 0 °

Core

Model: Mohr-Coulomb

Unit Weight: 19.8 kN/m³

Cohesion': 10 kPa

Phi': 29 °

Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Point

Left Coordinate: (10, 40) m

Left-Zone Increment: 4

Right Projection: Range

Right-Zone Left Coordinate: (50.38856, 53.4) m

Right-Zone Right Coordinate: (54, 53.4) m

Right-Zone Increment: 4

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 40) m

Right Coordinate: (129.7, 18.9) m

Points

	X (m)	Y (m)
Point 1	10	40
Point 2	0	50.4
Point 3	0	40
Point 4	41.2	50.4
Point 5	50.2	53.4
Point 6	54.2	53.4
Point 7	119.6	20.7
Point 8	50.8	50.9
Point 9	53.8	50.9
Point 10	42.2	33.6
Point 11	50.8	32.1
Point 12	51.3	30.6
Point 13	53.3	30.6
Point 14	53.8	31.6
Point 15	64.4	29.7
Point 16	129.7	18.9
Point 17	0	0
Point 18	129.7	0
Point 19	42.27491	33.75069

Regions

	Material	Points	Area (m^2)
Region 1	Slope	3,17,18,16,7,15,14,13,12,11,10,19,1	3,878.5
Region 2	Core	8,19,10,11,12,13,14,15,9	243.4
Region 3	Embankment	1,4,5,6,7,15,9,8,19	992.11

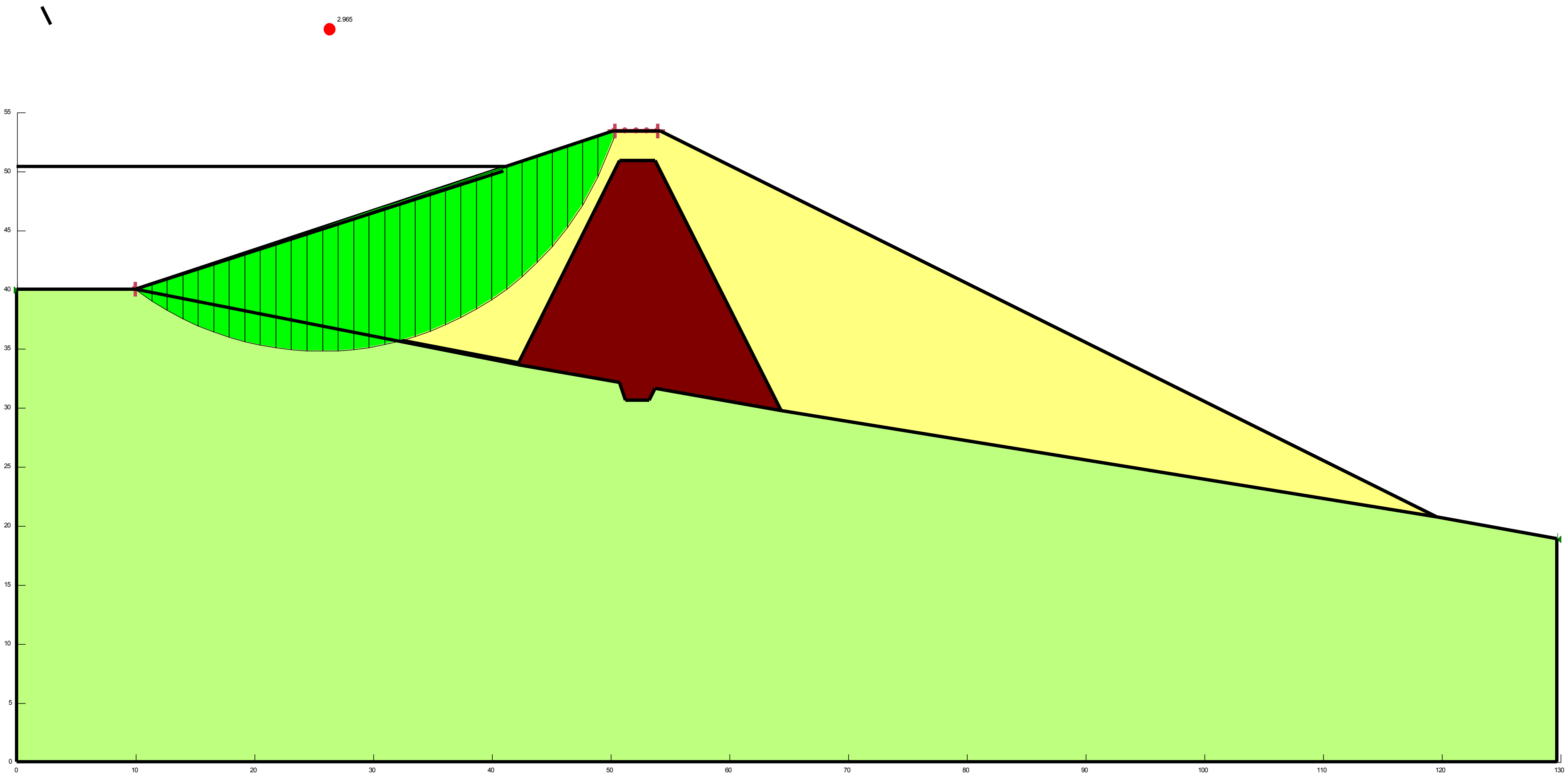
Current Slip Surface

Slip Surface: 4
F of S: 2.965
Volume: 333.28787 m^3
Weight: 7,127.645 kN
Resisting Moment: 130,928 kN-m
Activating Moment: 44,159.902 kN-m
Resisting Force: 4,434.7995 kN
Activating Force: 1,495.4152 kN
F of S Rank (Analysis): 11 of 25 slip surfaces
F of S Rank (Query): 11 of 25 slip surfaces
Exit: (10, 40) m
Entry: (50.38856, 53.4) m
Radius: 25.89579 m
Center: (25.546006, 60.71023) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	10.654063	39.539909	0	19.120065	10.598425	8
Slice 2	11.962188	38.676587	0	52.347119	29.016482	8
Slice 3	13.270313	37.921036	0	84.19222	46.66851	8
Slice 4	14.578438	37.262778	0	114.13124	63.263978	8
Slice 5	15.886564	36.693775	0	141.64981	78.517771	8
Slice 6	17.194689	36.207782	0	166.29488	92.178757	8
Slice 7	18.502814	35.799923	0	187.71631	104.05285	8
Slice 8	19.81094	35.466398	0	205.6941	114.0181	8
Slice 9	21.119065	35.204281	0	220.14916	122.03067	8
Slice 10	22.42719	35.011381	0	231.13846	128.12214	8
Slice 11	23.735315	34.886144	0	238.8373	132.38967	8
Slice 12	25.043441	34.827583	0	243.51313	134.98153	8
Slice 13	26.351566	34.835246	0	245.49555	136.08041	8
Slice 14	27.659691	34.909191	0	245.14636	135.88685	8
Slice 15	28.967817	35.049993	0	242.83279	134.60441	8
Slice 16	30.275942	35.258765	0	238.90554	132.4275	8
Slice 17	31.584067	35.537202	0	233.6821	129.53211	8
Slice 18	32.878263	35.883139	0	227.2347	159.11145	8
Slice 19	34.158531	36.298028	0	218.53043	153.01665	8
Slice 20	35.438798	36.788592	0	209.11501	146.4239	8
Slice 21	36.719065	37.359617	0	199.09315	139.40652	8
Slice	37.999332	38.017204	0	188.50585	131.99322	8

22						
Slice 23	39.279599	38.769171	0	177.32671	124.1655	8
Slice 24	40.559866	39.625676	0	165.45499	115.85283	8
Slice 25	41.842857	40.602523	0	152.67245	106.9024	8
Slice 26	43.128571	41.718709	0	138.67741	97.102966	8
Slice 27	44.414286	42.998717	0	123.04042	86.15383	8
Slice 28	45.7	44.481927	0	105.07894	73.577065	8
Slice 29	46.985714	46.23227	0	83.728652	58.627433	8
Slice 30	48.271429	48.367913	0	57.196737	40.049586	8
Slice 31	49.557143	51.169449	0	21.954058	15.372397	8
Slice 32	50.29428	53.093659	0	-1.1997595	-0.84008061	8


2.965
0
5
10
15
20
25
30
35
40
45
50
55
0
10
20
30
40
50
60
70
80
90
100
110
120
130

Slope Stability –upstreamslope draw down

File Information

File Version: 8.16
Revision Number: 65
Date: 11/5/2016
Time: 11:41:44 PM
Tool Version: 8.16.0.12829
File Name: Downsstream slope 13.4 thesis.gsz
Directory: C:\Users\Ian\Documents\
Last Solved Date: 11/5/2016
Last Solved Time: 11:41:54 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Slope Stability (2)

Kind: SLOPE/W
Method: Morgenstern-Price
Settings
  Side Function
    Interslice force function option: Half-Sine
  PWP Conditions Source: Piezometric Line
  Apply Phreatic Correction: No
  Use Staged Rapid Drawdown: Yes
Slip Surface
  Direction of movement: Right to Left
  Use Passive Mode: No
  Slip Surface Option: Entry and Exit
  Critical slip surfaces saved: 1
  Resisting Side Maximum Convex Angle: 1 °
  Driving Side Maximum Convex Angle: 5 °
  Optimize Critical Slip Surface Location: No
  Tension Crack

Tension Crack Option: (none)

F of S Distribution

F of S Calculation Option: Constant

Advanced

Number of Slices: 30

F of S Tolerance: 0.001

Minimum Slip Surface Depth: 0.1 m

Search Method: Root Finder

Tolerable difference between starting and converged F of S: 3

Maximum iterations to calculate converged lambda: 20

Max Absolute Lambda: 2

Materials

Embankment

Model: Mohr-Coulomb

Unit Weight: 22.1 kN/m³

Cohesion': 0 kPa

Phi': 35 °

Phi-B: 0 °

Cohesion R: 8 kPa

Phi R: 33 °

Pore Water Pressure

Piezometric Line: 1

Piezometric Line After Drawdown: 2

Slope

Model: Mohr-Coulomb

Unit Weight: 22.1 kN/m³

Cohesion': 0 kPa

Phi': 29 °

Phi-B: 0 °

Cohesion R: 8 kPa

Phi R: 24 °

Pore Water Pressure

Piezometric Line: 1

Piezometric Line After Drawdown: 2

Core

Model: Mohr-Coulomb

Unit Weight: 20.5 kN/m³

Cohesion': 0 kPa

Phi': 29 °

Phi-B: 0 °

Cohesion R: 10 kPa

Phi R: 28 °

Pore Water Pressure

Piezometric Line: 1

Piezometric Line After Drawdown: 2

Slip Surface Entry and Exit

Left Projection: Point
Left Coordinate: (10, 40) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (49.58293, 53.19431) m
Right-Zone Right Coordinate: (54, 53.4) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 40) m
Right Coordinate: (129.7, 18.9) m

Piezometric Lines

Piezometric Line 1

Coordinates

	X (m)	Y (m)
Coordinate 1	0	50.4
Coordinate 2	41.2	50.4
Coordinate 3	50	49.2907
Coordinate 4	58.75	41
Coordinate 5	97	24.38478

Piezometric Line 2

Coordinates

	X (m)	Y (m)
Coordinate 1	0	42
Coordinate 2	16	42
Coordinate 3	25	42
Coordinate 4	44	37.22093
Coordinate 5	63.75	31
Coordinate 6	94	24.87391

Points

	X (m)	Y (m)
Point 1	10	40
Point 2	0	50.4
Point 3	0	40
Point 4	41.2	50.4
Point 5	50.2	53.4
Point 6	54.2	53.4
Point 7	119.6	20.7
Point 8	50.8	50.9
Point 9	53.8	50.9
Point 10	42.2	33.6
Point 11	50.8	32.1
Point 12	51.3	30.6
Point 13	53.3	30.6
Point 14	53.8	31.6
Point 15	64.4	29.7
Point 16	129.7	18.9
Point 17	0	0
Point 18	129.7	0
Point 19	42.27491	33.75069
Point 20	94	24.87391
Point 21	94	24
Point 22	123.52778	20
Point 23	123	19

Regions

	Material	Points	Area (m^2)

Region 1	Core	8,19,10,11,12,13,14,15,9	243.4
Region 2	Embankment	1,4,5,6,7,20,15,9,8,19	992.11
Region 3	Slope	20,21,23,22	28.722
Region 4	Slope	3,17,18,16,22,23,21,20,15,14,13,12,11,10,19,1	3,849
Region 5	Slope	7,20,22	0.7629

Current Slip Surface

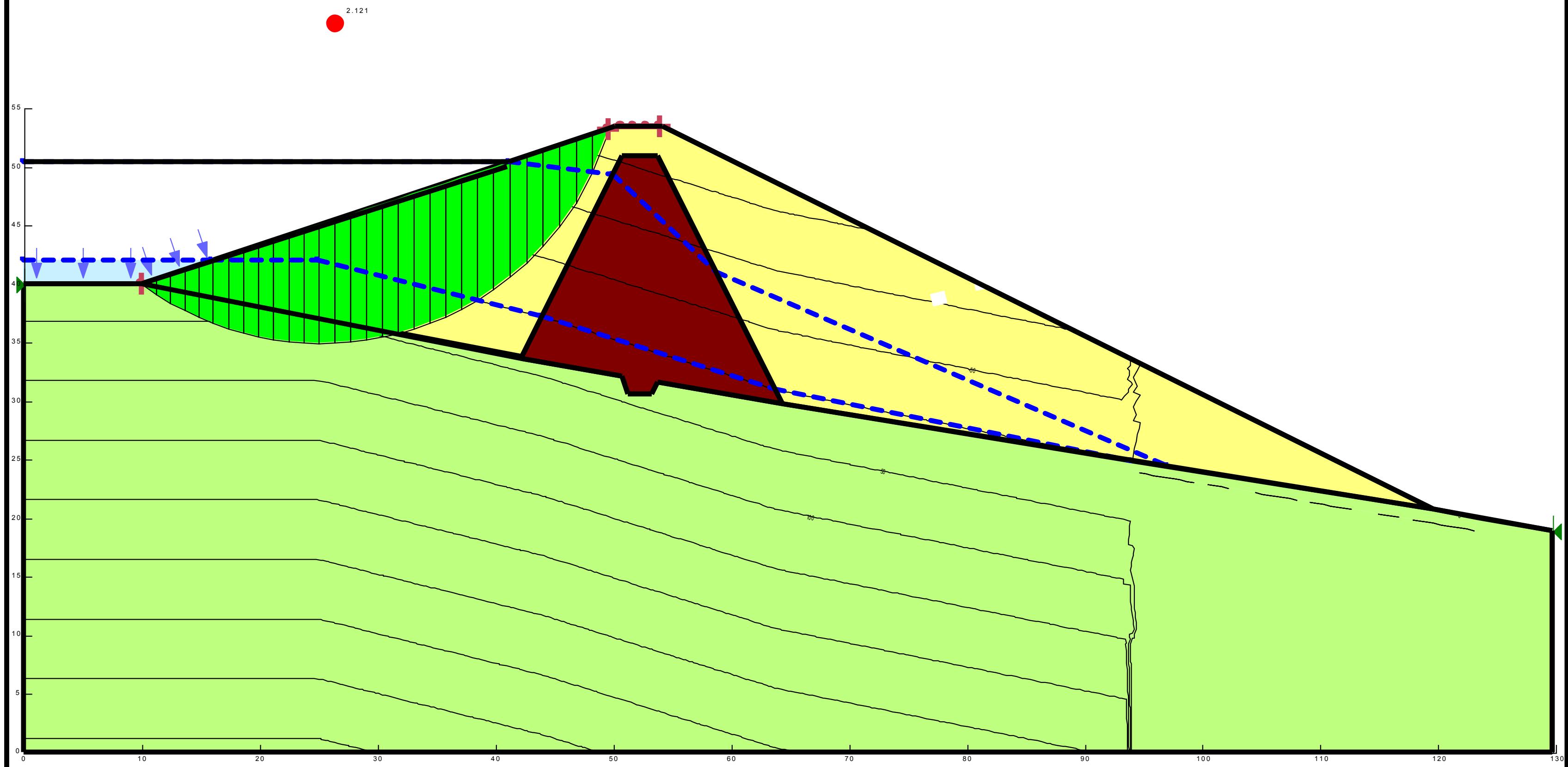
Slip Surface: 4
F of S: 2.121
Volume: 318.57079 m^3
Weight: 7,040.4145 kN
Resisting Moment: 86,856.603 kN-m
Activating Moment: 40,942.526 kN-m
Resisting Force: 2,961.3568 kN
Activating Force: 1,395.908 kN
F of S Rank (Analysis): 13 of 25 slip surfaces
F of S Rank (Query): 13 of 25 slip surfaces
Exit: (10, 40) m
Entry: (49.58293, 53.19431) m
Radius: 25.412477 m
Center: (25.20268, 60.363509) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	10.6	39.578466	23.747987	34.291721	5.8444872	0
Slice 2	11.8	38.784276	31.536601	62.972695	17.425312	0
Slice 3	13	38.083011	38.413913	89.841241	28.506634	0
Slice 4	14.2	37.466088	44.464076	114.5365	38.841778	0
Slice 5	15.4	36.926818	49.752693	136.7246	48.209315	0
Slice 6	16.642857	36.445791	54.470125	159.50506	58.221816	0
Slice	17.928571	36.0236	58.610552	182.44283	68.641351	0

7						
Slice 8	19.214286	35.675545	62.023925	201.84114	77.501946	0
Slice 9	20.5	35.398516	64.740751	217.60359	84.733254	0
Slice 10	21.785714	35.190161	66.784092	229.76897	90.34399	0
Slice 11	23.071429	35.048779	68.170626	238.49624	94.413031	0
Slice 12	24.357143	34.973248	68.911354	244.03994	97.075361	0
Slice 13	25.670571	34.964193	67.346026	246.85969	99.506046	0
Slice 14	27.011713	35.02442	63.447113	245.62228	0	87.923368
Slice 15	28.352854	35.156095	58.847505	243.57399	0	86.635944
Slice 16	29.693996	35.360351	53.5361	239.6994	0	84.783402
Slice 17	31.035138	35.638992	47.495202	234.33942	0	82.487004
Slice 18	32.373148	35.993561	40.717399	227.91086	0	118.98347
Slice 19	33.708025	36.427106	33.172807	219.11999	0	114.24013
Slice 20	35.042903	36.944716	24.803795	209.47159	0	109.21395
Slice 21	36.377781	37.552136	15.554006	199.06699	0	103.96616
Slice 22	37.712659	38.256806	5.3504878	187.92562	0	98.515017
Slice 23	39.085073	39.094648	-6.2516379	175.60064	0	92.653318
Slice 24	40.495024	40.086464	-19.456381	161.88042	0	86.287501

Slice 25	41.9	41.228975	-34.126714	146.0957	0	81.719782
Slice 26	43.3	42.551007	-50.545339	124.12342	86.912155	0
Slice 27	44.697549	44.098254	-69.600674	104.65482	73.280096	0
Slice 28	46.092647	45.944932	-92.020572	81.927157	57.366013	0
Slice 29	47.487745	48.237507	-118.81337	54.213051	37.960387	0
Slice 30	48.884112	51.356883	-153.71854	18.211784	12.752029	0



Slope Stability (2)

Downsstream slope 13.4 thesis.gsz

11/5/2016 1:496

Stability of cutting -after construction

File Information

File Version: 8.16
Revision Number: 16
Date: 11/5/2016
Time: 7:45:44 PM
Tool Version: 8.16.0.12829
File Name: Cutting slope.gsz
Directory: C:\Users\Ian\Documents\MAsters analysis\New folder\
Last Solved Date: 11/5/2016
Last Solved Time: 7:45:59 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Stability of cutting -after construction

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - PWP Conditions Source: (none)
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Resisting Side Maximum Convex Angle: 1 °
  - Driving Side Maximum Convex Angle: 5 °
  - Optimize Critical Slip Surface Location: No
  - Tension Crack
    - Tension Crack Option: (none)
- F of S Distribution
  - F of S Calculation Option: Constant
- Advanced

Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 m
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Slope

Model: Mohr-Coulomb
Unit Weight: 21.6 kN/m³
Cohesion': 8 kPa
Phi': 28 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (0.23899, 17.75698) m
Left-Zone Right Coordinate: (4.44444, 17) m
Left-Zone Increment: 4
Right Projection: Point
Right Coordinate: (37.58185, 5.0066) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 17.8) m
Right Coordinate: (47.6, 5) m

Points

	X (m)	Y (m)
Point 1	0	17.8
Point 2	5	16.9
Point 3	9	15.4
Point 4	37.6	5
Point 5	47.6	5
Point 6	0	0
Point 7	47.6	0

Regions

	Material	Points	Area (m^2)

Region 1	Slope	1,6,7,5,4,3,2	493.07

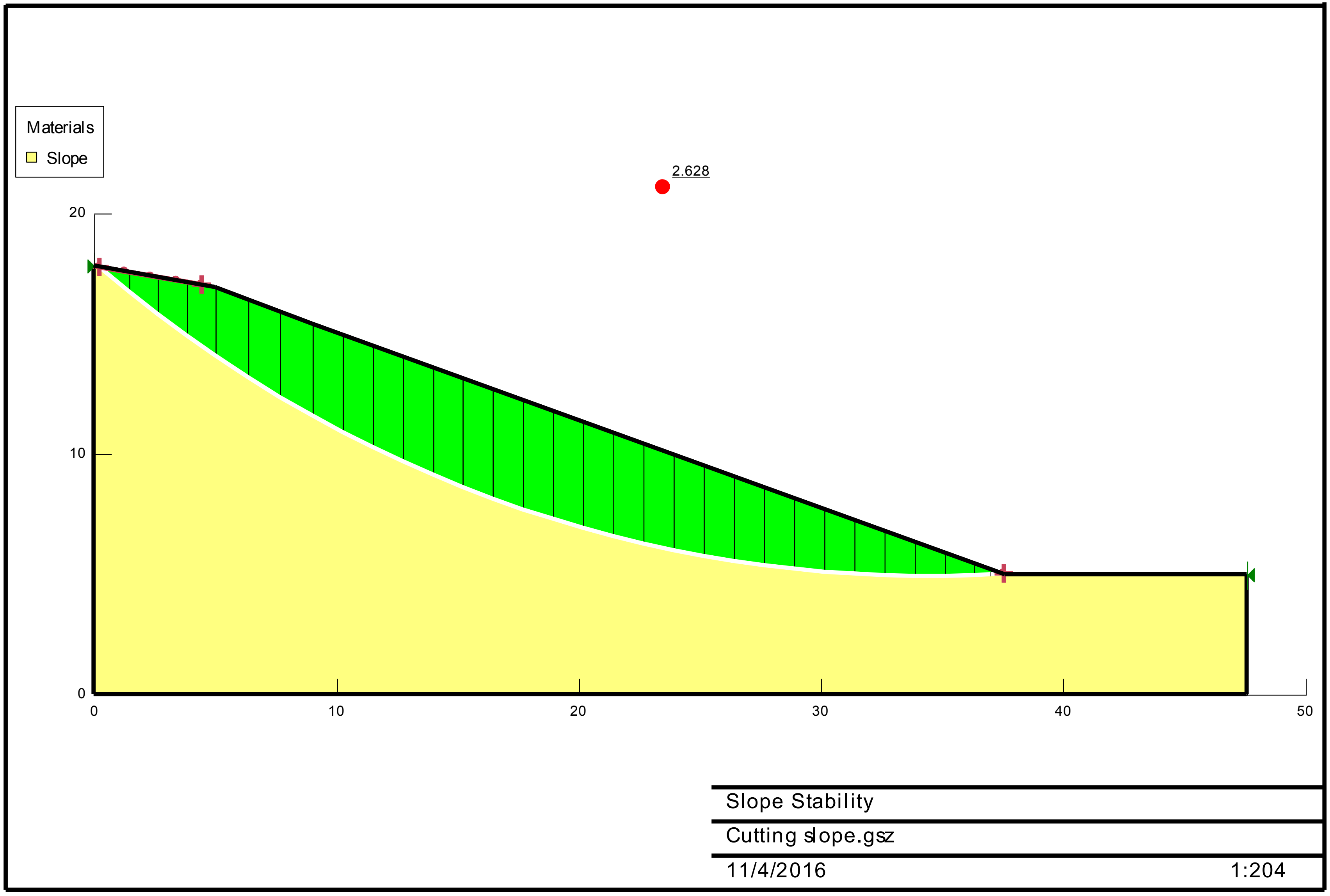
Current Slip Surface

Slip Surface: 2
F of S: 2.014
Volume: 116.45402 m^3
Weight: 2,515.4068 kN
Resisting Moment: 84,017.246 kN-m
Activating Moment: 41,712.184 kN-m
Resisting Force: 1,504.4161 kN
Activating Force: 746.90403 kN
F of S Rank (Analysis): 1 of 25 slip surfaces
F of S Rank (Query): 1 of 25 slip surfaces
Exit: (37.58185, 5.0066) m
Entry: (0.23899031, 17.756982) m
Radius: 52.094431 m
Center: (34.489475, 57.009166) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	0.83411652	17.253179	0	4.2935447	2.2829182	8
Slice 2	2.0243689	16.27506	0	17.777273	9.4523439	8
Slice 3	3.2146214	15.353922	0	30.021304	15.962611	8
Slice 4	4.4048738	14.486055	0	41.248058	21.931982	8
Slice 5	5.6666667	13.622131	0	49.974397	26.571858	8
Slice 6	7	12.765043	0	56.289121	29.929457	8
Slice 7	8.3333333	11.963727	0	61.905006	32.915476	8
Slice 8	9.6213446	11.239026	0	66.884149	35.562933	8
Slice 9	10.864034	10.585227	0	71.309038	37.915688	8
Slice 10	12.106723	9.9733438	0	75.274092	40.023945	8
Slice 11	13.349412	9.401759	0	78.786136	41.891331	8
Slice 12	14.592101	8.8690357	0	81.835404	43.512656	8
Slice 13	15.83479	8.3738956	0	84.396757	44.874551	8

Slice 14	17.077479	7.9152006	0	86.43085	45.956098	8
Slice 15	18.320168	7.4919372	0	87.885615	46.72961	8
Slice 16	19.562858	7.103203	0	88.698322	47.161734	8
Slice 17	20.805547	6.7481963	0	88.798457	47.214977	8
Slice 18	22.048236	6.4262064	0	88.11148	46.849705	8
Slice 19	23.290925	6.136606	0	86.563401	46.026577	8
Slice 20	24.533614	5.8788444	0	84.085968	44.709303	8
Slice 21	25.776303	5.6524419	0	80.622112	42.867538	8
Slice 22	27.018992	5.456985	0	76.131201	40.479678	8
Slice 23	28.261682	5.2921226	0	70.593618	37.535292	8
Slice 24	29.504371	5.1575628	0	64.014158	34.036931	8
Slice 25	30.74706	5.05307	0	56.423812	30.001073	8
Slice 26	31.989749	4.9784629	0	47.87962	25.458046	8
Slice 27	33.232438	4.933613	0	38.462431	20.450837	8
Slice 28	34.475127	4.9184434	0	28.272605	15.032811	8
Slice 29	35.717816	4.9329282	0	17.423886	9.2644443	8
Slice 30	36.960505	4.977092	0	6.0358685	3.2093282	8



Materials
Slope
2.628
20
10
0
0
10
20
30
40
50
Slope Stability
Cutting slope.gsz
11/4/2016
1:204

Stability of cutting-Drawdown

File Information

- File Version: 8.16
- Revision Number: 18
- Date: 11/5/2016
- Time: 11:50:51 PM
- Tool Version: 8.16.0.12829
- File Name: Cutting slope.gsz
- Directory: C:\Users\Ian\Documents\MAsters analysis\New folder\
- Last Solved Date: 11/6/2016
- Last Solved Time: 12:01:22 AM

Project Settings

- Length(L) Units: Meters
- Time(t) Units: Seconds
- Force(F) Units: Kilonewtons
- Pressure(p) Units: kPa
- Strength Units: kPa
- Unit Weight of Water: 9.807 kN/m³
- View: 2D
- Element Thickness: 1

Analysis Settings

Slope Stability (2)

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - PWP Conditions Source: Piezometric Line
  - Apply Phreatic Correction: No
  - Use Staged Rapid Drawdown: Yes
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Resisting Side Maximum Convex Angle: 1 °
  - Driving Side Maximum Convex Angle: 5 °
  - Optimize Critical Slip Surface Location: No
  - Tension Crack
    - Tension Crack Option: (none)
- F of S Distribution

F of S Calculation Option: Constant

Advanced

Number of Slices: 30

F of S Tolerance: 0.001

Minimum Slip Surface Depth: 0.1 m

Search Method: Root Finder

Tolerable difference between starting and converged F of S: 3

Maximum iterations to calculate converged lambda: 20

Max Absolute Lambda: 2

Materials

Slope

Model: Mohr-Coulomb

Unit Weight: 21.1 kN/m³

Cohesion': 0 kPa

Phi': 28 °

Phi-B: 0 °

Cohesion R: 8 kPa

Phi R: 27 °

Pore Water Pressure

Piezometric Line: 1

Piezometric Line After Drawdown: 2

Slip Surface Entry and Exit

Left Projection: Range

Left-Zone Left Coordinate: (0.23899, 17.75698) m

Left-Zone Right Coordinate: (4.44444, 17) m

Left-Zone Increment: 4

Right Projection: Point

Right Coordinate: (37.58185, 5.0066) m

Right-Zone Increment: 4

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 17.8) m

Right Coordinate: (47.6, 5) m

Piezometric Lines

Piezometric Line 1

Coordinates

	X (m)	Y (m)

Coordinate 1	0	15.4
Coordinate 2	9	15.4
Coordinate 3	47.6	15.4

Piezometric Line 2

Coordinates

	X (m)	Y (m)
Coordinate 1	0	10.2
Coordinate 2	47.6	10.2

Points

	X (m)	Y (m)
Point 1	0	17.8
Point 2	5	16.9
Point 3	9	15.4
Point 4	37.6	5
Point 5	47.6	5
Point 6	0	0
Point 7	47.6	0

Regions

	Material	Points	Area (m^2)
Region 1	Slope	1,6,7,5,4,3,2	493.07

Current Slip Surface

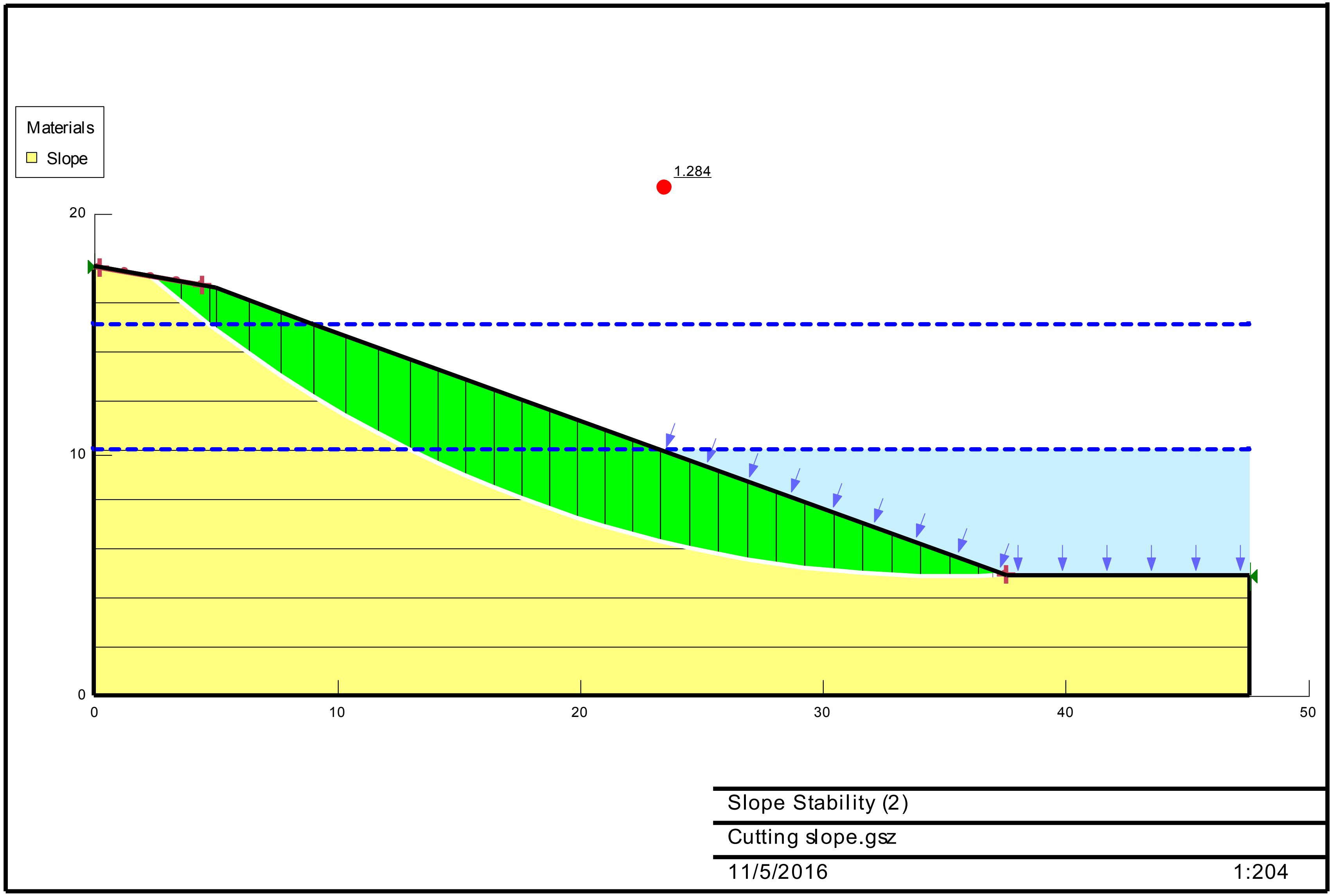
Slip Surface: 12
F of S: 1.284
Volume: 98.562495 m^3
Weight: 2,079.6686 kN
Resisting Moment: 35,981.009 kN-m
Activating Moment: 28,021.193 kN-m

Resisting Force: 667.42523 kN
Activating Force: 519.82654 kN
F of S Rank (Analysis): 1 of 25 slip surfaces
F of S Rank (Query): 1 of 25 slip surfaces
Exit: (37.58185, 5.0066) m
Entry: (2.3417152, 17.378491) m
Radius: 49.725147 m
Center: (35.227684, 54.675988) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	2.9392855	16.868158	-65.394627	6.2158536	3.305028	0
Slice 2	4.134426	15.878913	-55.693095	18.253635	9.70563	0
Slice 3	4.8659981	15.296684	-49.983177	25.332138	13.469337	0
Slice 4	5.6666667	14.700383	-44.135258	30.449297	16.190178	0
Slice 5	7	13.747609	-34.791401	37.835065	20.117261	0
Slice 6	8.3333333	12.859056	-26.07736	45.323057	0	21.048879
Slice 7	9.665142	12.031583	-17.962336	51.929162	0	21.60211
Slice 8	10.995426	11.261617	-10.411275	57.434924	0	23.218973
Slice 9	12.32571	10.54518	-3.3851786	62.24833	0	24.646328
Slice 10	13.563583	9.9227532	2.7189592	66.176348	0	25.83397
Slice 11	14.709043	9.3860306	7.982598	69.308697	0	26.808059
Slice 12	15.854504	8.8842488	12.903572	71.98603	0	27.673557
Slice 13	16.999965	8.4162699	17.493041	74.199461	0	28.42882

Slice 14	18.145426	7.9810767	21.760981	75.942372	28.808756	0
Slice 15	19.290887	7.5777596	25.716312	77.213508	27.381545	0
Slice 16	20.436348	7.2055055	29.367007	77.867863	25.788362	0
Slice 17	21.581809	6.8635885	32.720188	77.886727	24.015475	0
Slice 18	22.72727	6.5513614	35.782199	77.247675	22.047585	0
Slice 19	23.895077	6.2632965	38.607252	78.111988	21.005041	0
Slice 20	25.085231	6.0000046	41.189355	80.474936	20.888514	0
Slice 21	26.275385	5.7670873	43.473575	82.068376	20.521219	0
Slice 22	27.46554	5.5641121	45.464152	82.81945	19.862164	0
Slice 23	28.655694	5.3907089	47.164717	82.664052	18.875331	0
Slice 24	29.845848	5.2465663	48.578324	81.552019	17.532425	0
Slice 25	31.036002	5.1314287	49.707478	79.451508	15.815181	0
Slice 26	32.226156	5.0450942	50.554161	76.352036	13.716973	0
Slice 27	33.41631	4.9874125	51.119846	72.265831	11.243519	0
Slice 28	34.606465	4.9582837	51.405511	67.227285	8.4125866	0
Slice 29	35.796619	4.9576579	51.411649	61.290559	5.2527097	0
Slice 30	36.986773	4.9855337	51.138271	54.525567	1.8010574	0


Materials
Slope
1.284
20
10
0
0
10
20
30
40
50
Slope Stability (2)
Cutting slope.gsz
11/5/2016
1:204

Steady-State Seepage without filter

File Information

File Version: 8.16
Revision Number: 55
Date: 11/5/2016
Time: 10:15:15 PM
Tool Version: 8.16.0.12829
File Name: Downsstream slope 13.4 thesis.gsz
Directory: C:\Users\Ian\Documents\
Last Solved Date: 11/5/2016
Last Solved Time: 10:15:23 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Mass(M) Units: Grams
Mass Flux Units: g/sec
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Steady-State Seepage

Kind: SEEP/W
Method: Steady-State
Settings
 Include Air Flow: No
Control
 Apply Runoff: Yes
Convergence
 Maximum Number of Iterations: 500
 Minimum Pressure Head Difference: 0.005
 Significant Digits: 2
 Max # of Reviews: 10
 Hydraulic Under-Relaxation Criteria
  Under-Relaxation Initial Rate: 1
  Under-Relaxation Min. Rate: 0.1
  Under-Relaxation Reduction Rate: 0.65
  Under-Relaxation Iterations: 10
 Equation Solver: Parallel Direct
Time

Starting Time: 0 sec
Duration: 0 sec
Ending Time: 0 sec

Materials

Embankment

Model: Saturated / Unsaturated
Hydraulic
K-Function: embankment
Ky'/Kx' Ratio: 1
Rotation: 0 °

Slope

Model: Saturated / Unsaturated
Hydraulic
K-Function: embankment
Ky'/Kx' Ratio: 1
Rotation: 0 °

Core

Model: Saturated / Unsaturated
Hydraulic
K-Function: core
Ky'/Kx' Ratio: 1
Rotation: 0 °

Boundary Conditions

Zero Pressure

Type: Pressure Head 0
Review: No

potential seeage face

Type: Total Flux (Q) 0
Review: No

water head

Type: Head (H) 10.4
Review: No

K Functions

embankment

Model: Hyd K Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %

Saturated Kx: 4e-006
Data Points: Matric Suction (kPa), X-Conductivity (m/sec)
Data Point: (0.01, 4e-006)
Data Point: (0.018329807, 3.9497379e-006)
Data Point: (0.033598183, 3.8821396e-006)
Data Point: (0.061584821, 3.791684e-006)
Data Point: (0.11288379, 3.6710212e-006)
Data Point: (0.20691381, 3.5108821e-006)
Data Point: (0.37926902, 3.2999338e-006)
Data Point: (0.6951928, 3.0250659e-006)
Data Point: (1.274275, 2.6728766e-006)
Data Point: (2.3357215, 2.2338578e-006)
Data Point: (4.2813324, 1.7120358e-006)
Data Point: (7.8475997, 1.1436488e-006)
Data Point: (14.384499, 6.1629223e-007)
Data Point: (26.366509, 2.4451574e-007)
Data Point: (48.329302, 6.7938854e-008)
Data Point: (88.586679, 1.3866329e-008)
Data Point: (162.37767, 2.321804e-009)
Data Point: (297.63514, 3.5125928e-010)
Data Point: (545.55948, 5.076797e-011)
Data Point: (1,000, 7.1973189e-012)
Estimation Properties
Hyd. K-Function Estimation Method: Van Genuchten Function
Volume Water Content Function: core
Saturated Kx: 4e-006 m/sec
Residual Water Content: 0.005 m^3/m^3
Maximum: 1,000
Minimum: 0.01
Num. Points: 20

core

Model: Hyd K Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Saturated Kx: 4e-007
Data Points: Matric Suction (kPa), X-Conductivity (m/sec)
Data Point: (0.01, 4e-007)
Data Point: (0.018329807, 3.9990914e-007)
Data Point: (0.033598183, 3.9969081e-007)
Data Point: (0.061584821, 3.9916813e-007)
Data Point: (0.11288379, 3.9791655e-007)
Data Point: (0.20691381, 3.9491841e-007)
Data Point: (0.37926902, 3.877576e-007)
Data Point: (0.6951928, 3.7078233e-007)
Data Point: (1.274275, 3.3158662e-007)
Data Point: (2.3357215, 2.4908693e-007)
Data Point: (4.2813324, 1.1965466e-007)
Data Point: (7.8475997, 2.3303713e-008)
Data Point: (14.384499, 1.6593044e-009)
Data Point: (26.366509, 6.9251805e-011)

Data Point: (48.329302, 2.449839e-012)
Data Point: (88.586679, 8.317781e-014)
Data Point: (162.37767, 2.7973455e-015)
Data Point: (297.63514, 9.3871997e-017)
Data Point: (545.55948, 3.1485459e-018)
Data Point: (1,000, 1.0559293e-019)
Estimation Properties
Hyd. K-Function Estimation Method: Van Genuchten Function
Volume Water Content Function: embankment
Saturated Kx: 4e-007 m/sec
Residual Water Content: 0.04 m³/m³
Maximum: 1,000
Minimum: 0.01
Num. Points: 20

Points

	X (m)	Y (m)	Hydraulic Boundary
Point 1	10	40	
Point 2	0	50.4	
Point 3	0	40	
Point 4	41.2	50.4	
Point 5	50.2	53.4	
Point 6	54.2	53.4	
Point 7	119.6	20.7	Zero Pressure
Point 8	50.8	50.9	
Point 9	53.8	50.9	
Point 10	42.2	33.6	
Point 11	50.8	32.1	
Point 12	51.3	30.6	
Point 13	53.3	30.6	
Point 14	53.8	31.6	
Point 15	64.4	29.7	
Point 16	129.7	18.9	
Point 17	0	0	
Point 18	129.7	0	
Point 19	42.27491	33.75069	

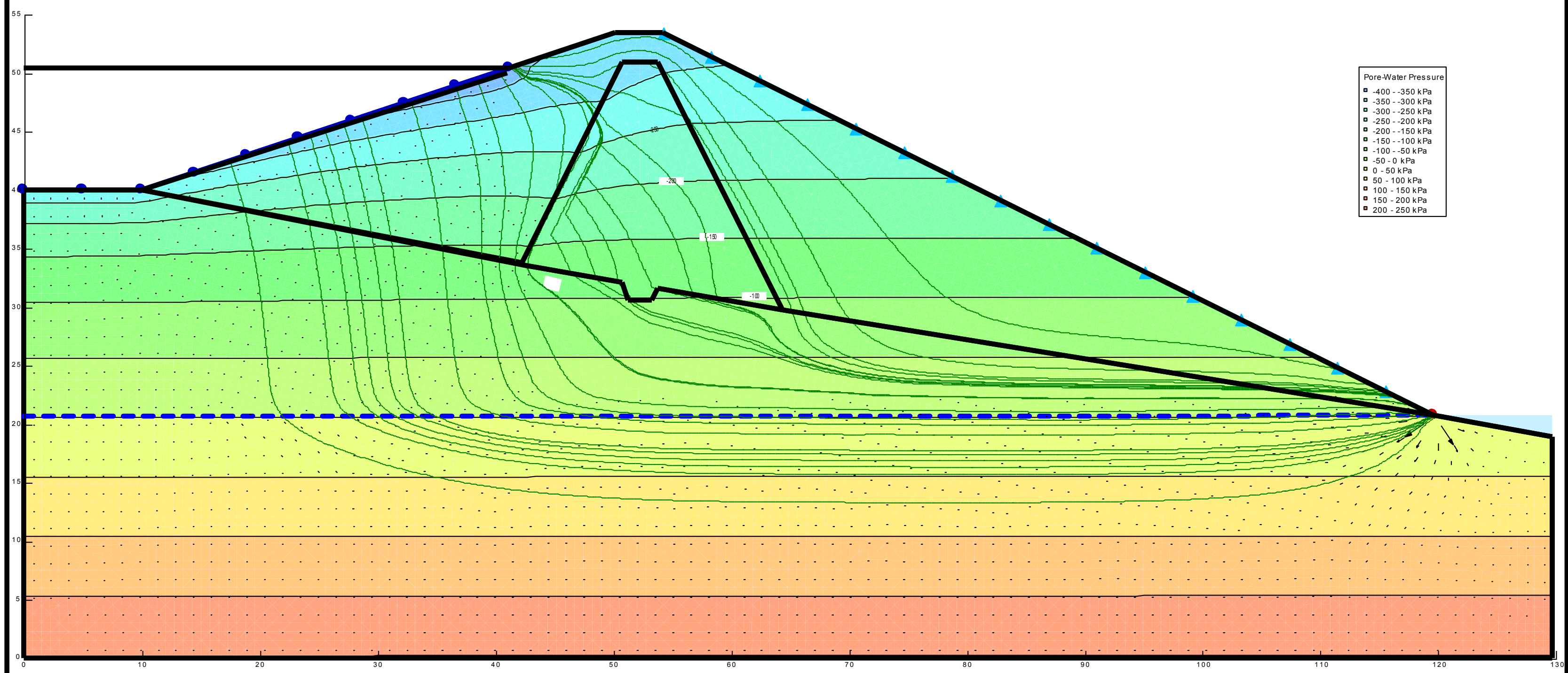
Lines

	Start Point	End Point	Hydraulic Boundary	Length (m)	Angle (°)
Line 1	3	1	water head	10	0
Line 2	11	12		1.5811	-71.6
Line 3	12	13		2	0
Line 4	13	14		1.118	63.4
Line 5	14	15		10.769	-10.2
Line 6	15	7		55.929	-9.26

Line 7	7	16		10.259	-10.1
Line 8	16	18		18.9	90
Line 9	18	17		129.7	0
Line 10	17	3		40	90
Line 11	1	19		32.874	-11
Line 12	11	10		8.7298	-9.89
Line 13	10	19		0.16828	63.6
Line 14	8	19		19.151	63.6
Line 15	15	9		23.702	-63.4
Line 16	9	8		3	0
Line 17	1	4	water head	32.888	18.4
Line 18	4	5		9.4868	18.4
Line 19	5	6		4	0
Line 20	6	7	potential seeage face	73.119	-26.6

Regions

	Material	Points	Area (m^2)
Region 1	Slope	3,17,18,16,7,15,14,13,12,11,10,19,1	3,878.5
Region 2	Core	8,19,10,11,12,13,14,15,9	243.4
Region 3	Embankment	1,4,5,6,7,15,9,8,19	992.11



Steady-State Seepage

Downsstream slope 13.4 thesis.gsz

11/5/2016 1:496

Steady-State Seepage with filter longer

File Information

File Version: 8.16
Revision Number: 70
Date: 11/6/2016
Time: 8:06:16 PM
Tool Version: 8.16.0.12829
File Name: Downsstream slope 13.4 thesis.gsz
Directory: C:\Users\Ian\Documents\
Last Solved Date: 11/6/2016
Last Solved Time: 8:06:24 PM

Project Settings

Length(L) Units: Meters
Time(t) Units: Seconds
Force(F) Units: Kilonewtons
Pressure(p) Units: kPa
Mass(M) Units: Grams
Mass Flux Units: g/sec
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Steady-State Seepage with filter longer

Kind: SEEP/W
Method: Steady-State
Settings
- Include Air Flow: No

Control
- Apply Runoff: Yes

Convergence
- Maximum Number of Iterations: 500
- Minimum Pressure Head Difference: 0.005
- Significant Digits: 2
- Max # of Reviews: 10
- Hydraulic Under-Relaxation Criteria
  - Under-Relaxation Initial Rate: 1
  - Under-Relaxation Min. Rate: 0.1
  - Under-Relaxation Reduction Rate: 0.65
  - Under-Relaxation Iterations: 10

Equation Solver: Parallel Direct
Time
Starting Time: 0 sec
Duration: 0 sec
Ending Time: 0 sec

Materials

Embankment
Model: Saturated / Unsaturated
Hydraulic
K-Function: embankment
Ky'/Kx' Ratio: 1
Rotation: 0 °

Slope
Model: Saturated / Unsaturated
Hydraulic
K-Function: embankment
Ky'/Kx' Ratio: 1
Rotation: 0 °

Core
Model: Saturated / Unsaturated
Hydraulic
K-Function: core
Ky'/Kx' Ratio: 1
Rotation: 0 °

filter
Model: Saturated Only
Hydraulic
Sat Kx: 0.001 m/sec
Ky'/Kx' Ratio: 1
Rotation: 0 °
Volumetric Water Content: 0 m^3/m^3
Mv: 0 /kPa

Boundary Conditions

Zero Pressure
Type: Pressure Head 0
Review: No

water head
Type: Head (H) 10.4
Review: No

K Functions

embankment

Model: Hyd K Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Saturated Kx: 4e-006
Data Points: Matric Suction (kPa), X-Conductivity (m/sec)
Data Point: (0.01, 4e-006)
Data Point: (0.018329807, 3.9497379e-006)
Data Point: (0.033598183, 3.8821396e-006)
Data Point: (0.061584821, 3.791684e-006)
Data Point: (0.11288379, 3.6710212e-006)
Data Point: (0.20691381, 3.5108821e-006)
Data Point: (0.37926902, 3.2999338e-006)
Data Point: (0.6951928, 3.0250659e-006)
Data Point: (1.274275, 2.6728766e-006)
Data Point: (2.3357215, 2.2338578e-006)
Data Point: (4.2813324, 1.7120358e-006)
Data Point: (7.8475997, 1.1436488e-006)
Data Point: (14.384499, 6.1629223e-007)
Data Point: (26.366509, 2.4451574e-007)
Data Point: (48.329302, 6.7938854e-008)
Data Point: (88.586679, 1.3866329e-008)
Data Point: (162.37767, 2.321804e-009)
Data Point: (297.63514, 3.5125928e-010)
Data Point: (545.55948, 5.076797e-011)
Data Point: (1,000, 7.1973189e-012)
Estimation Properties
Hyd. K-Function Estimation Method: Van Genuchten Function
Volume Water Content Function: core
Saturated Kx: 4e-006 m/sec
Residual Water Content: 0.005 m^3/m^3
Maximum: 1,000
Minimum: 0.01
Num. Points: 20

core

Model: Hyd K Data Point Function
Function: X-Conductivity vs. Pore-Water Pressure
Curve Fit to Data: 100 %
Segment Curvature: 100 %
Saturated Kx: 4e-007
Data Points: Matric Suction (kPa), X-Conductivity (m/sec)
Data Point: (0.01, 4e-007)
Data Point: (0.018329807, 3.9990914e-007)
Data Point: (0.033598183, 3.9969081e-007)
Data Point: (0.061584821, 3.9916813e-007)
Data Point: (0.11288379, 3.9791655e-007)
Data Point: (0.20691381, 3.9491841e-007)

Data Point: (0.37926902, 3.877576e-007)
Data Point: (0.6951928, 3.7078233e-007)
Data Point: (1.274275, 3.3158662e-007)
Data Point: (2.3357215, 2.4908693e-007)
Data Point: (4.2813324, 1.1965466e-007)
Data Point: (7.8475997, 2.3303713e-008)
Data Point: (14.384499, 1.6593044e-009)
Data Point: (26.366509, 6.9251805e-011)
Data Point: (48.329302, 2.449839e-012)
Data Point: (88.586679, 8.317781e-014)
Data Point: (162.37767, 2.7973455e-015)
Data Point: (297.63514, 9.3871997e-017)
Data Point: (545.55948, 3.1485459e-018)
Data Point: (1,000, 1.0559293e-019)

Estimation Properties

Hyd. K-Function Estimation Method: Van Genuchten Function
Volume Water Content Function: embankment
Saturated Kx: 4e-007 m/sec
Residual Water Content: 0.04 m^3/m^3
Maximum: 1,000
Minimum: 0.01
Num. Points: 20

Points

	X (m)	Y (m)	Hydraulic Boundary
Point 1	10	40	
Point 2	0	50.4	
Point 3	0	40	
Point 4	41.2	50.4	
Point 5	50.2	53.4	
Point 6	54.2	53.4	
Point 7	119.6	20.7	Zero Pressure
Point 8	50.8	50.9	
Point 9	53.8	50.9	
Point 10	42.2	33.6	
Point 11	50.8	32.1	
Point 12	51.3	30.6	
Point 13	53.3	30.6	
Point 14	53.8	31.6	
Point 15	64.4	29.7	
Point 16	129.7	18.9	
Point 17	0	0	
Point 18	129.7	0	
Point 19	42.27491	33.75069	
Point 20	94	24.87391	
Point 21	94	24	

Point 22	123.52778	20	
Point 23	123	19	
Point 24	80.64152	27.05192	
Point 25	80.5	26	
Point 26	94	24.85534	

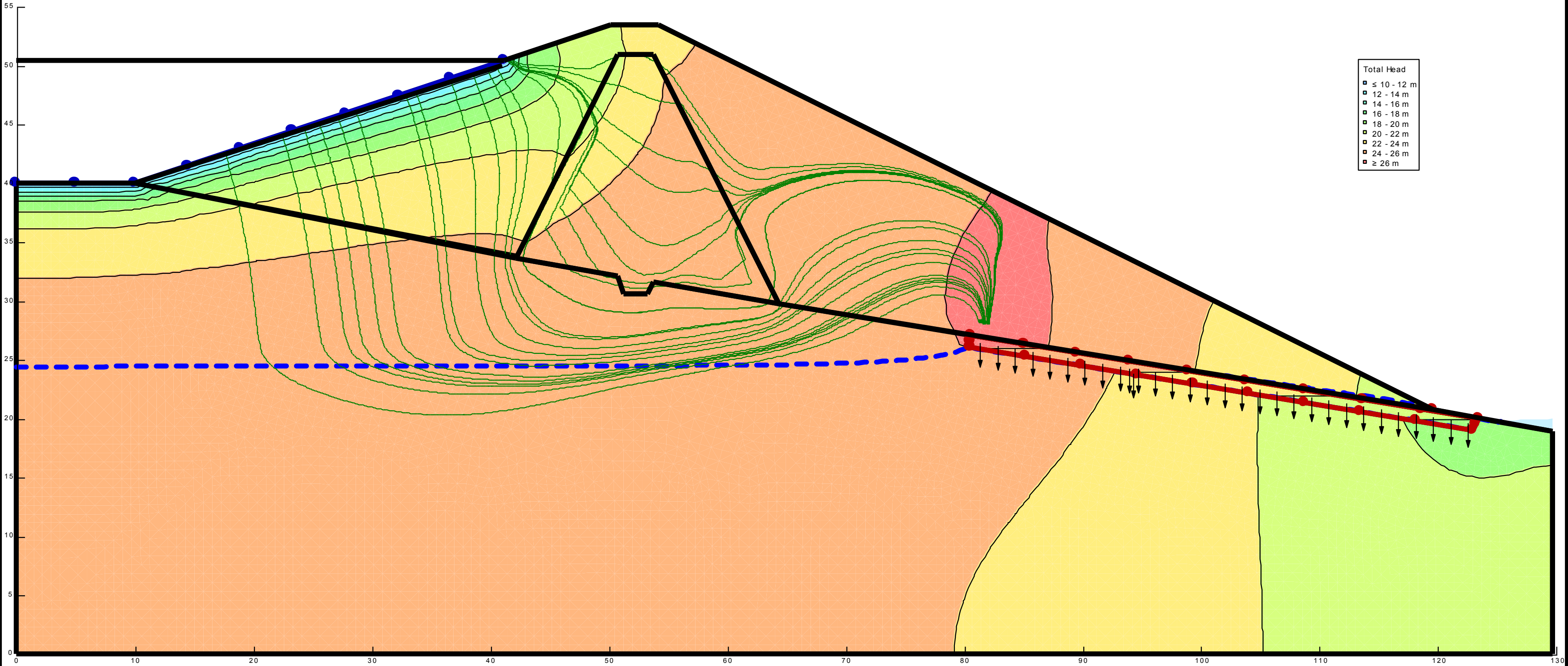
Lines

	Start Point	End Point	Hydraulic Boundary	Length (m)	Angle (°)
Line 1	3	1	water head	10	0
Line 2	11	12		1.5811	-71.6
Line 3	12	13		2	0
Line 4	13	14		1.118	63.4
Line 5	14	15		10.769	-10.2
Line 6	16	18		18.9	90
Line 7	18	17		129.7	0
Line 8	17	3		40	90
Line 9	1	19		32.874	-11
Line 10	11	10		8.7298	-9.89
Line 11	10	19		0.16828	63.6
Line 12	8	19		19.151	63.6
Line 13	15	9		23.702	-63.4
Line 14	9	8		3	0
Line 15	1	4	water head	32.888	18.4
Line 16	4	5		9.4868	18.4
Line 17	5	6		4	0
Line 18	6	7		73.119	-26.6
Line 19	20	7		25.938	-9.26
Line 20	7	22		3.9897	-10.1
Line 21	22	16		6.2695	-10.1
Line 22	23	22		1.1307	62.2
Line 23	22	20		29.927	-9.37
Line 24	15	24		16.456	-9.26
Line 25	24	20		13.535	-9.26
Line 26	25	24		1.0614	82.3
Line 27	25	21		13.647	-8.43
Line 28	20	26		0.01857	90
Line 29	25	23		43.073	-9.35
Line 30	22	26		29.924	-9.34
Line 31	26	24		13.538	-9.34

Regions

	Material	Points	Area	Hydraulic

			(m²)	Boundary
Region 1	Core	8,19,10,11,12,13,14,15,9	243.4	
Region 2	Embankment	1,4,5,6,7,20,24,15,9,8,19	992.11	
Region 3	Slope	7,20,22	0.7629	
Region 4	filter	24,25,23,22,26	46.153	Zero Pressure
Region 5	Slope	3,17,18,16,22,23,25,24,15,14,13,12,11,10,19,1	3,831.2	
Region 6	Slope	20,24,26	0.12403	



Steady-State Seepage with filter longer

Downsstream slope 13.4 thesis.gsz

11/6/2016 1:496