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School of Architecture and Built Environment

**MSc in Geo-Information Science for Environment and
Sustainable Development**

**AN ENGINEERING AND GIS-BASED EVALUATION OF THE FACTORS
CONTRIBUTING TO ROAD CRACKS AND BUILDING SETTLEMENTS: CASE STUDY
OF GATUNA ONE STOP BORDER POST**

**Thesis submitted to the University of Rwanda: College of Science and Technology in
partial fulfillment of the requirements for the award of the Degree of Master of Science in
Geo-Information Science for Environment and Sustainable Development.**

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Kigali, 2nd September 2024

DECLARATION

I declare that this thesis entitled “**An Engineering And GIS-Based Evaluation of The Factors Contributing To Road Cracks and Building Settlements: Case Study of Gatuna One Stop Border Post** ” Case study of Gatuna one stop border post is submitted for the award of the Degree of Master of Science in Geo-Information Science for Environment and Sustainable Development as my original work that has never been submitted to any University or other higher learning institution.

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APPROVAL

It is hereby confirmed that this thesis entitled “**An Engineering And GIS-Based Evaluation of The Factors Contributing to Road Cracks and Building Settlements: Case Study of Gatuna One Stop Border Post**” submitted by **Eng. NSENGIYUMVA Noel** was assessed in relation to the fulfilment of the requirements for the award of the Degree of Master of Science in Geo-Information Science for Environment and Sustainable Development, in the School of Architecture and Built Environment.

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DEDICATION

I gratefully dedicate this work:

To the Almighty God who leads me in every aspect of life,

To all my classmates, all engineers who contribute in this research, especially Eng. Juliette MUKASHYAKA for her encouragement & investment in this research, and whole team who encouraged me,

To My teachers and Lecturers who have given me the knowledge, family and Friends for their encouragement and unconditional technical support.

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LIST OF ABBREVIATIONS AND ACRONYMS

BH: Bore Hole

DEM: Digital Elevation Model

DGPS: Differential Global Positioning System

DPC: Dynamic cone penetration

GIS: Geographical Information System

GNSS: Global Navigation Satellite System

GoR: Government of Rwanda

IDF: Intensity-Duration-Frequency

LDC: Least Developed Country

MINIFRA: Ministry of infrastructure

OSBP: One Stop Border Post

RTDA: Rwanda Transport Development Agency

SMI: Soil Moisture Index

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ABSTRACT

Road infrastructure is a critical component of national development, significantly contributing to trade facilitation, market accessibility, and economic growth. Despite its importance, challenges such as road cracks and building settlements hinder the efficient operation of key transportation hubs, such as the Gatuna One Stop Border Post (OSBP) in Rwanda. This study investigated the underlying causes of these challenges and proposes sustainable solutions. By leveraging Engineering and Geographic Information System (GIS) technology, the research examines various factors contributing to road deterioration and building settlements. The analysis incorporated satellite imagery, topographic maps, and geospatial data to develop detailed maps that highlight areas of concern, with particular attention to the geological framework. The results indicate that roads built on shale rock suffer from slope instability and inadequate load-bearing capacity, while infrastructure on organic material soil is prone to settlement and subsidence due to the compressible nature of the soil and its vulnerability to waterlogging. Structural assessments of buildings at Gatuna OSBP reveal that the predominant cracking in external masonry walls is likely due to ground movement and foundation settlement. To address these issues, the study proposes a series of interventions, including the design of drainage systems to reduce water table levels, the application of soil stabilization techniques, and the adoption of foundation designs tailored to the unique challenges posed by shale rock and organic material soils. Furthermore, the research emphasizes the importance of continuous monitoring and maintenance to ensure the long-term stability and safety of the infrastructure. In conclusion, this thesis highlights the necessity of interdisciplinary collaboration among geotechnical engineers, structural designers, construction managers, geo-spatial experts and environmentalist to develop effective strategies for mitigating the risks of road cracks and building settlements. Addressing these challenges will enhance the sustainability and resilience of Gatuna OSBP transportation hubs, thereby improving regional trade and connectivity.

Keywords: Road infrastructure development, Gatuna One Stop Border Post (OSBP), road cracks and building settlements.

CHAPTER 1. INTRODUCTION

1.1. BACKGROUND

Road construction and maintenance is an important desire for development especially in developing countries. Market accessibility, economic growth, natural resource exploitation, habitat fragmentation, deforestation, and the disappearance of wild lands and wildlife are all related to road existence and status Wilkie, et.al.2000. The priority of investment is addressed when decision usually taken for new construction of road and for maintenance. whether for political needs, industrial stations, or agricultural zones, during the decision-making process for new road construction and maintenance. Road transport is the moving engine for other sectors and activities in development countries. It provides access to industry, agriculture, investment, health services, and education through providing of goods and passengers. The lack for good roads or the existence of poorly maintained or poor conditions of roads are barriers to development and investment in developing countries. Climate conditions were seen to have an effect on road deterioration, vehicle operating costs, road safety and the environment. (Anyala et.al. 2011). In 2005, Transport Canada indicated that climate factors are a major cause for pavement deterioration. It is a fact that temperature, frost and thaw action as well as moisture are factors that can cause certain types of pavement deterioration. These factors can also intensify pavement deterioration caused by heavy vehicles. When a road gets damaged, cracks begin to appear on its surface. GIS facilitates the identification and mapping of road cracks and building settlements by integrating spatial data such as satellite imagery, environmental data (such as temperature, precipitation, and humidity) and field surveys to to assess their impact on road infrastructure. For instance, road cracks in Rwanda, including those on the Kigali-Musanze Road, were attributed to heavy rain weakening the soil, as explained by Mukantabana Seraphine. Similarly, the eruption of the Nyiragongo volcano in DR Congo caused road cracks and damaged houses in Rubavu, Volcanic earthquake also caused the road crack and damaged more than 40 houses in Rubavu (Newtime, 2021).



Figure 1: Deep crack caused by heavy rain in road of Kigali-Musanze, at Gakenke, Source: Newtimes, 2013), left and road crack in Rubavu, 2021(right, Newtimes, 2021).

On May 8, 2018, deep cracks appeared at the frontline of the Kigali – Gatuna road, 5km from the Border Post at the adjacent swamp area, followed by a mass movement of soil and deep settlement of houses nearby the swamp area on July 17, 2016, at the side of the Republic of Uganda, and on June 22, 2019, at the Gatuna site, a deep settlement appeared at the new constructed parking yard going through the edge of the building which followed by differential settlement caused by different assumption matrix . Modern infrastructure and livelihoods, Vision 2050 focuses on reliable infrastructure where Upgrade 440 km national roads, rehabilitate (paved) 453 km national roads, and to ensure the riding quality is kept to 97% for paved roads, and four operational weighbridges and Construct two one-stop border post (OSBP) were identified as key interventions of transport in NST1, priority 4 which intended to improve the hard infrastructure developed for trade competitiveness. (JICA, 2019).

Rwanda Vision 2020-2050 is a strategic plan aimed at transforming Rwanda into a middle-income country by 2035 and a high-income country by 2050. The period from 2017 to 2024 is crucial in laying the groundwork for achieving these long-term goals. Here are some of the key interventions during this period: **Economic Diversification:** Rwanda aims to diversify its economy beyond agriculture and increase the contribution of manufacturing, services, and information communication technology (ICT) sectors. This involves implementing policies to attract investment, support small and medium-sized enterprises (SMEs), and improve the business environment. **Infrastructure Development:** Significant investments are made in infrastructure, including roads, railways, airports, and energy. Improving infrastructure is essential for facilitating trade, reducing transportation costs, and enhancing connectivity within the country and with neighboring regions(REPUBLIC OF RWANDA, 2020).

Rwanda, a landlocked Least Developed Country (LDC), is also prioritizing regional initiatives to facilitate the transfer of goods and to provide alternative routings in order to ensure efficient in-country transportation and connections to foreign markets. As a result, One Border Post has emerged as a significant priority for national economic development. In the same footing, a One Border Post is seen as a vital component of Rwanda's trade with its regional neighbour, (MINIFRA,2017).

In 2004, the East African Community countries entered into an agreement with the World Bank to support the East African Trade Facilitation Project. The improvement of border crossings in East Africa was one of its many components. In this regard, the Government of Rwanda (GoR) through the Rwanda Transport Development Agency (RTDA) planned to set up a One Stop Border Post (OSBP) at Gatuna. The Gatuna OSBP Project was implemented at the existing Gatuna Border Post. The operation of the existing border post was suffering from traffic congestions, delays in cargo clearance, poor infrastructures and uncoordinated activities among other challenges. (Newtimes, 2017). It has therefore become imperative to attempt to improve the services by eliminate the observed challenges and smoothen the clearance process at the border post as well as systems to respond to the mentioned challenges.

The road and building construction, the road cracks and building settlements were identified as major snags in provisional handover meeting of 2019 by the end users' technical team (MINIFRA, 2019). The challenge remains to insufficient information on cause of those road cracks and building settlements, this may be linked to insufficiency of research conducted before on this subject. This is highlighting the need for further research. An engineering and GIS-based evaluation serves as a valuable tool for addressing these challenges by integrating spatial data, conducting spatial analysis, and informing evidence-based decision-making in infrastructure development and maintenance.

1.2. PROBLEM STATEMENT

Road infrastructure is pivotal for economic growth and social development, especially in developing countries. Efficient road networks enhance trade, investment, and accessibility to essential services like healthcare and education. However, the Gatuna One Stop Border Post (OSBP), a critical transit point between Rwanda and Uganda within the East African Community, faces significant challenges, primarily due to persistent road cracks. These cracks severely impact the entire infrastructure system, hindering operational efficiency and economic potential. The issue of road settlement and cracking at Gatuna OSBP has been documented over several times, for instances: on 17th July 2016, 8th May 2018, and most recently on 22nd June 2019 at Gatuna One Stop Border Point (OSBP). Despite multiple efforts such as road maintenance and construction to address these issues through, the problem of road cracks and building settlement persists. Previous rehabilitations have often tackled symptoms rather than addressing root causes, exacerbated by inadequate studies into the underlying issues before intervention decisions were made.

In Rwanda as well as Gatuna OSBP, the major contributing factors to road cracks is heavy vehicle traffic compounded by climatic conditions such as heavy rainfall (BONISHULI, 2014). These factors weaken soil structures, leading to pavement degradation and subsequent cracks. Moreover, suboptimal construction practices and the use of substandard materials may further accelerate road deterioration, undermining the OSBP's functionality. Studies indicate that a significant portion of utility service infrastructures lie beneath road pavements, influencing their stability and longevity. The quality of road structural pavements is crucial; however, the frequent destruction of these layers and the use of inferior materials during replacements pose ongoing challenges (Croney, 1972; BONISHULI, 2014). Additionally, approximately 80% of roads in Rwanda remain unpaved, causing problems such as rough rides, cargo damage, and increased travel times during wet and dry seasons. The absence of sufficient engineering practices exacerbates these issues, as contractors often lack mechanisms to evaluate soil excavation impacts or incorporate effective design and construction specifications (Walaa and Elhoussein, 1999).

The construction history of Gatuna OSBP reveals that despite meticulous planning and phased implementation of building, retaining wall, and access road construction, issues persist. Notably, in June 2019, a major crack emerged along a newly constructed road, accompanied by soil settlement near critical infrastructure components like underground water tanks and powerhouses (Backyard). These incidents highlight the complex interplay of construction

methodologies and environmental factors contributing to infrastructure instability. The challenge remains to insufficient information on cause of those road cracks and building settlements, this may be linked to insufficiency of research conducted before on this subject.

Despite the intervention has been done to maintain the road and building, the problem continued to be appeared, and the root cause were not yet known, in this regard this study investigated the major cause of the road cracks and building settlement, which provided information for better planning and policy making.

Civil Engineering, Geographical Information System (GIS) and Remote Sensing technology offers a robust approach. By integrating spatial data and conducting detailed analyses, GIS can monitor road cracks, assess climatic impacts, and inform targeted maintenance strategies. This research aimed to comprehensively assess the root causes of road cracks and building settlement at Gatuna OSBP. By identifying these causes and their interdependencies, the study developed sustainable strategies that enhance the resilience and operational efficiency of the OSBP and serve as a model for similar transit points across Rwanda and the broader East African region.

1.3. MAIN OBJECTIVE

The main objective of this study is to evaluate the factors contributing to road cracks and building settlements at the Gatuna One Stop Border Post using an integrated engineering and GIS-based approach.

1.4. SPECIFIC OBJECTIVES.

1. To assess the main causes of road cracks of Kigali- Gatuna using engineering and GIS-based technology
2. Evaluate the factors contributing to the settlement of buildings at Gatuna One Stop Boarder Point (OSBP).
3. To determine the stability of the existing building at Gatuna OSBP through structure analysis, soil condition and environmental variables
4. To provide recommendations that guide the management and prevention of roads cracks and building settlement.

1.5. RESEARCH QUESTIONS

1. What are the potential causes of road cracks that are affecting the road of Kigali-Gatuna ?
2. What are the contributing factors of building settlement at Gatuna One Stop Boarder Point (OSBP).
3. Is the type of foundation used in existing Gatuna building able to support the load of the building?
4. What are the recommendation to be taken to guide planners, decision makers in order to prevent the road cracks and building settlement for sustainable development of Rwanda?

1.5. HYPOTHESES

The followings are the hypotheses of my research

- 1) The occurrence of road cracks is associated with soil types and dry season (summer).
- 2) Building settlement at the Gatuna One Stop Border Post (OSBP) is likely caused by a combination of factors, including fragile wetland soils surrounding the building, inadequate compaction and poor quality of the sub-base during road construction, unsuitable foundation types for the building's structural frame. and the influence of soil types and seasonal variations such as the dry season (summer).

1.6. JUSTIFICATION OF THE STUDY

The justification for this research is grounded in several significant development frameworks and goals, emphasizing the importance of addressing road cracking and soil settlement issues along the Kigali-Gatuna Road for sustainable development, improved infrastructure resilience, and economic growth. This research is crucial for supporting Rwanda's Vision 2050, NST1, the East African Community's Vision 2050, the SDGs, and Agenda 2063. By addressing the root causes of road cracking and soil settlement, the study aims to develop sustainable solutions that enhance the resilience and functionality of critical infrastructure, thus fostering economic growth and regional integration.

Rwanda's Vision 2050 aims to transform the country into an upper-middle-income nation by 2035 and a high-income country by 2050. This vision focuses on achieving high standards of living, economic growth, and sustainable resource use. Ensuring the integrity of critical infrastructure, such as the Kigali-Gatuna Road, is vital for facilitating trade, enhancing regional

connectivity, and contributing to economic prosperity, directly supporting Vision 2050's objectives (Wikipedia) (Sustainable Development Platform).

The National Strategy for Transformation (NST1) operationalizes Vision 2050 through a medium-term plan from 2017 to 2024. NST1 focuses on creating 1.5 million productive jobs and accelerating economic growth. A reliable road network is essential for achieving these goals, as it enables efficient movement of goods and services, reduces transportation costs, and enhances market accessibility (Sustainable Development Platform). The East African Community's Vision 2050 seeks to promote a prosperous, competitive, secure, and politically united East Africa. Improved infrastructure within member states, including resilient road networks, is fundamental to achieving seamless regional integration, fostering economic development, and enhancing cross-border trade (Sustainable Development Platform).

The United Nations' Sustainable Development Goals (SDGs) outline a global agenda for sustainable development. This research supports several SDGs, including:

Goal 9: Industry, Innovation, and Infrastructure: By identifying and addressing infrastructure vulnerabilities, this study contributes to building resilient infrastructure and promoting inclusive and sustainable industrialization.

Goal 11: Sustainable Cities and Communities: Enhancing the resilience and sustainability of urban and rural infrastructure ensures that communities are better prepared for future challenges.

Goal 13: Climate Action: Understanding the impact of climatic conditions on road integrity supports adaptation and resilience strategies in the face of climate change (Sustainable Development Platform) (Wikipedia).

The providing the information on road cracks and building settlement, will guide planner and decision maker to take a sustainable solution on road cracks and soil settlement issues, which is in line with the broader goals of Agenda 2063 to promote infrastructure that supports economic activities and regional connectivity (Sustainable Development Platform).

Road infrastructure is the backbone of economic activities and social interactions, especially for landlocked countries like Rwanda. Reliable road networks enhance international trade, improve access to markets, and facilitate the movement of people and goods. The Kigali-Gatuna Road is a key transit route within the East African Community, and ensuring its stability is

essential for maintaining and boosting regional trade and cooperation (Sustainable Development Platform).

1.7. OUTLINE OF THE THESIS

This thesis begins with introductory sections, including the Declaration, approval, Dedication, and Acknowledgment, **followed by:**

Chapter 1: Introduction, which provides the Background, Problem Statement, Main Objective, Specific Objectives, research questions and the hypotheses of the research and highlighted the significance of this research. **Chapter 2:** Literature Review delves into the importance of road infrastructure, defining key concepts such as road cracks, their causes and types, and building settlement. It also discusses the impacts of these phenomena on infrastructure and current practices used to address these issues. The review highlighted existing knowledge on road deterioration, building settlement, and the engineering approaches that have been applied to mitigate these challenges.

In Chapter 3: Methodology, the thesis outlines the research methods used, starting with the description of study area. The chapter also explains the **Sampling Strategy** and details the Methods for Data Collection, focusing on the evaluation of road cracks and building settlement causes, The chapter concludes with the procedures for Data Processing and Analysis.

Chapter 4: Findings presents the results of the research. It examines the major causes of road cracks and building settlement through GIS Technology and assesses the stability of the existing infrastructure at the Gatuna OSBP. This chapter includes detailed topographical, hydrological, and geotechnical investigations, along with structural analysis and soil investigations. The findings are used to validate the hypotheses and provide insights into fragile wetland soils, poor compaction, foundation-related structural issues, and the influence of soil types and seasonal variations on road cracks. The chapter concludes with a summary of findings.

In Chapter 5: Conclusion, Proposed Solutions, and Recommendations, the thesis offers final conclusions based on the research, along with several proposed solutions to address the identified issues. These include the use of Bored Piles, Soil Nails, and Reinforced Earth (RE) techniques. The chapter also outlines remedial works and provides recommendations for future infrastructure development and maintenance at the Gatuna OSBP.

CHAPTER 2. LITERATURE REVIEW

2.1. INTRODUCTION

This section is presenting the key **definitions on road cracks and building settlement, causes and impacts of road cracks and building settlement, types of cracks**, current practice on road construction, some identified gaps for in road construction in Rwanda and best practices to reduce road cracks and building settlement

Roads are a primary mode of transport and every day; several vehicles pass through the roads. Therefore, it is important that you have good quality roads for a safe commute. Road density is closely linked to market accessibility, economic growth, natural resource exploitation (DAVID WILKIE, 2000) .The mass and the quality of the road infrastructure are primarily determining the competitiveness of the country. Its advantages of road transport include transporting passengers and carrying goods regardless of distance directly to a destination, the level to which the road infrastructure is urbanized has an force on numerous areas, such as tourism development, influx of foreign investments, regional development and cooperation ,etc which impact to on the volume of gross domestic product(GDP), (Masarova & Ivanova, 2013). However, due to the constant movement of vehicles, roads do get damaged eventually. Initially, when a road gets damaged, cracks begin to appear on its surface.

In Rwanda, a good quality infrastructure is vital to reduce transport cost at maximum, as particularly Rwanda is landlocked country. It absorb new additional traffic induced by the economic development of Rwanda and the increase of trade with neighbouring countries as the road relates the two one-stop border posts of Rusumo and Kagitumba to the main access road to Kigali the east side of Rwanda, continuing to the DRC borders in Rubavu or Goma and Rusizi or Bukavu and Kigali-Gatuna, Kigali-Kayonza, of Kigali-Huye (Karuhanga, 2019).

2.2. GENERAL DEFINITION OF ROAD CRACK, CAUSE & TYPE OF CRACK AND BUILDING SETTLEMENT

2.2.1. Road cracks.

Cracks in roads are caused by natural movement and heat changes. The **cracks** allow water to damage the **road** base which leads to bigger problems like potholes. The general causes of pavement deterioration and degradation are overloading, seepage, improper or poor road surface drainage, lack of proper road maintenance, lack of proper design, adverse climatic conditions and some other factors. (Jamal, 2017) Road pavement crack detection is one of

elements of the road inspection process. It comprises problems of crack analysis, crack classification, crack depth estimation and crack sealing. (STANIEK1, 2017). Identified basis for cracks which include Pressure from cars, but mostly due to water getting into small cracks, freezing and expanding, therefore makes the cracks bigger, and repeat. High temperature or warm on pavement, it expands the pavement crack. A lot of water into cracks and water freeze make the cracks get bigger, High traffic loads, Geometric fault, earthquake can cause huge cracks, Improper or poor road surface drainage, Lack of proper road maintenance and lack of proper design (Taher, 2019) .

2.2.2. Type of Road Cracks.

They are eleven (11) types of roads crack as discussed by (Jamal, 2017), those include;

1.Fatigue or Alligator Cracking: Fatigue cracks are interconnected cracks that are caused by the influence of frequent traffic loading. Roads that face extreme climates and heavily loaded vehicles undergo severe fatigue cracking. It is also referred to as Alligator cracking as the crack pattern resembles the skin of an alligator. Fatigue cracking if not attended causes roughness and may lead to structural failure, water seepage through the cracks and can further degrade to form potholes.

The cause of this cracks are summarized as ; Excessive loading, Weak surface, base, or sub grade, Thin surface or base, Poor drainage, Old and dried out mix, Mix was placed too dry, Fine aggregate mix with low penetration asphalt & absorptive aggregates and aggravated by low traffic volume.

2. Block Cracking in Roads: Block cracking is in the form of interconnected rectangular cracks. These types of cracks also cause roughness and water seepage through the cracks.it caused by Old and dried out mix, Mix was placed too dry, Fine aggregate mix with low penetration asphalt & absorptive aggregates and aggravated by low traffic volume.

3. Longitudinal and Transverse cracking: Cracks formed in the direction of the traffic flow are termed as longitudinal cracks and those that are formed perpendicular to the traffic flow are called transverse cracks.this is linked to Poorly constructed paving joint crack, Shrinkage of the asphalt layer , Daily temperature cycling , Cracks in an underlying layer that reflect up through the pavement and Longitudinal segregation caused by the improper operation of the paver.

4. Corrugation and Shoving: It is in the form of a sudden wave in the road surface and is perpendicular to the direction of traffic flow and is mostly located at the points where a vehicle starts or stops in the road. It causes roughness and discomfort to the driver. This type of cracks is caused by Mixtures too high in asphalt, Low air voids, high fine aggregate content, Excessive moisture or contamination in the granular base and Smooth or rounded aggregate as well as incorrect asphalt grade

5. Depression: Localized Road sections with a slight depression, it causes roughness and collects water which may lead to pothole formation.

In Rwanda, the main destructive agent of the soil is rain erosion. In some sites, active erosion mainly caused landslide hazards, which increase sediments in rivers. Other than that, erosion has also formed gully bodies through the slopes of mountainous areas in the project area. Construction activities increase the potential of occurrence of landslides and erosion in various ways, which include destabilization of rock masses by cuts in slopes, improper stockpiling of materials, destruction of vegetative cover during site clearing and uncontrolled surface run-off (slope wash) during storms may increase the erosion rate. Nevertheless, erosion is of relevance to slope stability, which is in turn relevant to the design of the project and the conduct of operations such as excavation, quarrying and borrowing.

6. Pot holes: Potholes are bowl-shaped depressions of varying sizes in the pavement surface. They generally have sharp edges. Potholes are most likely to occur on roads with thin surfaces course. They are usually caused when the severity of cracks increases. They cause roughness and rider discomfort and are a major cause of accidents especially at dark times when there visibility is very low. It caused by Continued deterioration of another type of distress, such as thawing of a frozen subgrade, cracking, raveling, or a failed patch after pieces of the original pavement surface have been dislodged, Poor surface mixtures, Weak spots in the base or subgrade and Severity of the surrounding distress and traffic action accelerate pothole.

7. Raveling: The crumbling up of the asphalt layer as a result of the disintegration between the aggregate particles and the asphalt binder. It causes roughness and skid resistance problem and may lead to pothole formation as the moisture infiltration increases because of the openings.

8. Rutting: It is identified as the slight depression along the wheel paths in a pavement. It is classified into two types.

9. Mix rutting: when the rutting does not affect the sub grade and **Sub grade Rutting:** When the rutting is prominent in the sub grade.

10. Water Bleeding: Water bleeding occurs when water seeps out of joints or cracks or through an excessively porous HMA layer.

11. Stripping: The loss of bond between aggregates and asphalt binder that typically begins at the bottom of the asphalt layer and progresses upward. When stripping begins at the surface and progresses downward it is usually called raveling.

2.2.3. Building settlement.

Settlement is the downward movement of the ground caused by a load consolidate the soil below it or causing displacement of the soil. Settlement often refers to the downward movement of the ground around an excavated space, such as that for tunnels, shafts, or basements(<https://legitcivil.com>). On the building, the extent to which the ground movement impacts upon buildings depends on a number of factors, including existing soil conditions., Methods of construction, Size and depth of the construction works, Type of structure, its condition, and its foundations. Settlement occurs within the foundation soils that surround and support the structure.

The impact of settlement is differing to the effect of heave which is the upward movement of the ground, and is different from subsidence which occurs where soil is unstable and sinks downward without any imposed load.

2.2.4. Causes of building settlement

The common cause of building settlement including;

1.Weak bearing soils: This can occur when foundations are designed based upon general guidelines rather than site-specific soil information, such as in the case of some residential construction projects.

2. Poor compaction: The levelling of a site prior to foundations being constructed often relies upon the placement of fill soils. Often these are imported from off-site locations and unless they are properly placed and compacted, may compress under loading and result in settlement of the structure.

3. Changes in moisture content : The soil's ability to support the load of a foundation settlement may be reduced by changes in moisture content within the soil. If soils are saturated, clays and silts may be softened. If soils are dried-out they have a tendency to shrink or contract.

4. Maturing trees and vegetation : a common cause of settlement is root systems expanding from maturing trees and vegetation and drawing soil moisture from beneath a structure. A general rule is that the diameter of a tree's root system is at least as large as its canopy.

5. Soil consolidation : Applied loads force water out of clay soils which compress and result in downward movement or settlement of overlying structures. Settlement caused by consolidation may take a considerable amount of time to be 'complete'.

2.2.5. Impact of road crack and building settlement.

Road distresses disturb and adversely affect the traffic flow and traffic safety leading to poor performance of the road. They also cause an increase in fuel costs, result in time delay and prove troublesome for every road user. In Jordan the cost for road construction and maintenance consumed about 292.1 M JD in the implementation program (2007-2009) and about 192.2 in (2011-2013).

Identification of the **road cracks** at an early stage is essential as preventive road maintenance and effective remedial measures can be applied before the problem becomes too severe and the pavement fails. Pavement maintenance thus leads to enormous economic benefits in preventing the pavement from failing. Also, the distresses whether of a smaller or larger scale are a nuisance to the road users and may prove hazardous if neglected for a long period as their condition worsens with time. Proper, timely and selective road maintenance thus becomes an important principle which lengthens the life of the pavement and also reduces the cost of maintenance. (Jamal, 2017).

2.3. CURRENT PRACTICE (STATUS) TO ROAD CRACKS AND BUILDING SETTLEMENT.

An automatic method of asphalt pavement crack detection, based on the application of image-processing technologies and the statistical analysis of the image intensity function was used. (STANIEK1, 2017). The method in question enables crack detection at the level of 92% of all existing cracks identified by manual visual inspection (i.e., by an expert conducting road inspections). It is assumed that this application makes it possible to identify longitudinal, transverse and alligator-type cracks. Meanwhile, due to the segmentation operation, one can perform crack surface dimensioning and establish both the type and the scope of the necessary

repairs. (STANIEK1, 2017). The properly sealing cracks and patching potholes used to Pavement Preservation and Maintenance.(<http://asphaltmagazine.com/preventing-and-repairing-potholes-and-pavement-cracks>).Galehouse said that, we have to do road maintenance when the roads are in fair-to-good condition, rather than waiting until they are in poor condition. Because the cost of addressing minor deficiencies is much less than addressing major deficiencies. In Rwanda, it was recommended to force governments to allocate funds to maintain the existing road network instead of regularly financing road rehabilitation and network expansion. Basic maintenance i.e urgent situation works like removal of landslides, routine maintenance and periodic maintenance are key to a successful road network maintenance strategy. (AIMABLE, 2013). Law sets different standards of road works as follows;

Routine maintenance: carrying out works which are required on a continuous basis and involve general cleaning of drainage channels, vegetation control and repairs to landslides on the road;

Recurrent maintenance: works which are required at intervals throughout the year and involves repair of potholes, grading of earth roads, and patching and sealing of cracks on tarmac roads;

Periodic maintenance: work which is required at intervals of several years and includes re-gravelling and stabilization of gravel roads, overlaying or re-surfacing of paved roads;

Road development: the way a road is upgraded, constructed, widened, reconstructed or adjusted by installing on its other necessary facilities (AIMABLE, 2013).

The size of national road should be 7 m of width and each road must have a large piece of land for drainage ditches, embankments, dumps and sidewalks on all integral parts of the roads (AIMABLE, 2013). An additional Ministerial Order to define the technical and service standards for roads based on the study and research conducted by experts on the entire road network of Rwanda was established by the Law roads (Aimable,2013). The major challenge today is that neither these Ministerial Orders nor the Presidential Order stated have been established or implemented. Poor planned road systems can have high maintenance and repair costs, which lead to excessive erosion, and fail to meet the needs of the users. The road planning, road location, road design, road survey, road construction and maintenance were highlighted by amiable in 2013 as best practice management system of road in Rwanda. Road survey, design, and construction are the steps in the process where road user needs are combined with geometric factors and terrain features, and the road is built on the ground. road or site survey is needed to

identify the terrain features, such as drainages, outcrops, and ground slopes, and to add some level of geometric control to a project. Elements of design include roadway geometry, design speed, drainage, stream crossing structures, slope stabilization needs, structural sections (materials type, use, and thickness), and road grades, key linkage between design and construction are the use of standard plans and drawings that show how the work should look, and specifications that describe how the work is to be done. a further key part of construction is quality control and inspection to ensure that the work is done in accordance with the plans and specifications. Some total of sampling and testing is usually precise to ensure that the materials used in construction meet specifications (AIMABLE, 2013). For materials to be used, it is necessary to Compact the road embankments, sub grade material, and surfacing materials, particularly in sensitive areas, or allow new roads to “settle” for several weeks before using the road. In wet climates a prolonged period of time is desirable, Use Road surface stabilization measures, like aggregate or pavements. Utilize durable materials that will not degrade to fine sediments under traffic (AIMABLE, 2013). In very steep terrain with high slope, use of retaining walls is required.

For building settlement, some steps were recommended to overcome the identified problem, those include , Site investigations can be undertaken to try and find out how much settlement is likely and the potential effect on buildings, measurement of actual ground and building movement, assess the existing conditions of buildings and how they may be impacted by further settlement .,Footings may be designed to spread loads over weak soils, reducing potential foundation settlement and Compensation grouting can be used to treat ground. (<https://www.designingbuildings.co.uk/wiki/Settlement>). However, from different reports provided in construction of Kigali-Gatuna roads, and building of one border station, the cracks along the road and settlement of building were observed and reported, but the problem remains as the cause of this road cracks and building settlement. This research analysed and determined the causes of this roads cracks and building settlement and to determine the stability of the existing building and the road closed to the swamp area in order to provide recommendations to guide planners, decisions makers for sustainable development.

CHAPTER.3 RESEARCH METHODOLOGY

Introduction

The methodology section of a thesis provides a structured framework detailing how research is conducted to address specific questions or objectives. It encompasses description of the study area, the research design, data collection methods, analysis techniques, and tools utilized in the study, offering transparency and ensuring the reliability of findings.

3.1. THE STUDY AREA DELINEATION

The study area map delineates the geographic region under investigation, providing a visual representation of its boundaries and key features. It serves as a crucial reference tool for understanding the spatial context of the research and the distribution of relevant variables within the study area.

3.1.1. General Description of the Juxtaposed One-Stop Border Post Project at Gatuna project Location

The Gatuna OSBP project is situated in Gatuna Village, Rwankonjo Cell, Cyumba Sector, Gicumbi District, Northern Province, Rwanda, at the Rwanda-Uganda border, approximately 80 km from Kigali. It is accessible via Rubaya-Bungwe-Musanze and Mukono-Musanze feeder roads, linking it to Musanze District.

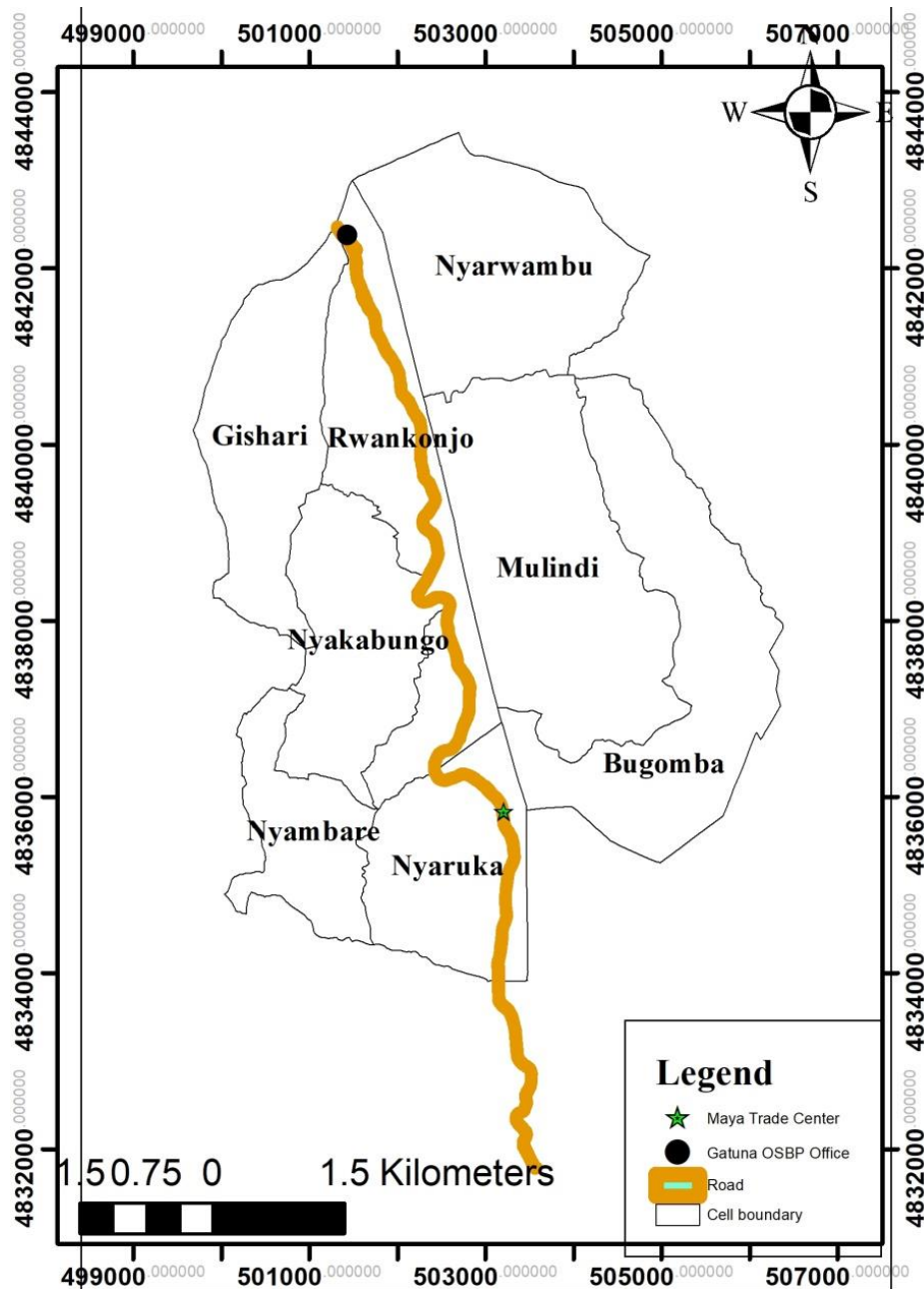


Figure 3: Study area delineation map with administrative boundaries

Source : <https://rwanda.africageoportal.com/>



Figure 4: Aerial view of Gatuna Border and its surrounding environment

Source : Google earth.

3.1.2. Topography and Landscape

Gicumbi District features a mountainous terrain characterized by high-altitude erosive relief, steep hills, and deep valleys. The topography ranges from 1800 m in valley bottoms to over 2500 m above sea level on mountains, with predominantly steep slopes. Annual rainfall averages between 1200 mm to 1500 mm, and the average temperature ranges from 15°C to 16°C. Soil quality is affected by water erosion, predominantly Kaolisol-dominated schist and quartz soils that tend to be acidic. The climate exhibits four alternating rainy and dry seasons, resulting in irregular but abundant rainfall. The landscape is shaped by extensive terracing on hillsides, utilized for settlements or cultivation (District <http://www.gicumbi.gov.rw>).

3.2. SAMPLING STRATEGY /PURPOSIVE SAMPLING

We purposefully collected 300 soil samples to investigate road cracks and an additional 8 samples to study building settlements from the nearby wetland. The road samples were specifically taken along the stretch from Maya Center to Gatuna Border Post, focusing on locations where cracks have been identified. The 30 samples were collected in one (1) Kilometer which totalling 300 samples in 10 km from Maya Center to Gatuna border post and 8 points of boreholes were identified around the building as per its size.

3.3. METHOD FOR DATA COLLECTION AND EQUIPMENT

3.3.1. Evaluation of the causes of building settlement and road cracks

For the assessment of the factors contributing to road cracks and building settlements, we employed a comprehensive approach involving detailed observations, field measurements, interviews, and soil testing. These methods were instrumental in gathering data and understanding the underlying causes of these structural issues. **Topographical and Alignment Survey**

The topographical and alignment survey was conducted using advanced tools such as a Topographical Total Station, Differential Global Positioning System (DGPS), and Auto Level. These tools measured road levels at 10-meter intervals along roads and streets, including junctions, gradient changes, road turnings, notable landmarks, and built-up areas. The data collected were used to create a detailed contour map. DGPS provided reliable data for field engineers, ensuring high precision in determining geo-spatial positions (Patil Vilas K., 2018). Additionally, the Global Navigation Satellite System (GNSS) was utilized within the network survey to provide autonomous geo-spatial positions using high-precision receivers. <https://www.princeton.edu/~alaink/Orf467F07/GNSS.pdf>.

Road Assessment

A comprehensive analysis of the road pavement was conducted, including material and structural evaluation. Tests measured layer thickness, material quality, and construction practices. The findings helped identify possible causes of pavement defects and quantify the extent of pavement distress. Building and other concrete structures were evaluated for soil bearing capacity using tools such as the Rebound Hammer and concrete scanner . (Mohammad Javed, 2012)

Embankment and Slope Analysis

Exploration test results were tabulated to show the stratification of the slope. Tests evaluated various soil parameters, properties, groundwater levels, and fluctuations, analyzing the horizontal and vertical permeability of the subsoil strata. Slope stability analysis was conducted to understand failure mechanisms, and soil characteristics were assessed to determine if issues were due to fragile soil or poor compaction. Dynamic Cone Penetration (DPC) tests were performed to assess the sequence, thickness, and bearing capacity of the slope area. Sieve

analysis determined the distribution of different grain sizes within the soil, and Atterberg limits classified fine-grained soils (liquid or plastic limits).

Various boreholes were designed to measure hydrological data, including surface and subsurface drainage, sanitation works, and groundwater levels and fluctuations, using tape and depth measurements. Additionally, interviews with elders, contractors, and engineers were conducted to gather historical information about the first observation of cracks at the site.

3.3.2. To determine the stability of the existing building and retaining wall.

To assess the stability of the existing building and retaining wall at the Gatuna site, a comprehensive structural analysis was conducted. The foundation base type was determined by evaluating the bearing capacity of the soils using the Dynamic Cone Penetration (DCP) test. This test helped ascertain whether the foundation used in the existing Gatuna building could support the building's load.

Soil testing and DCP tests were performed by taking eight soil samples along the building's length. These tests evaluated the relationship between soil pressure and soil bearing capacity. The Cone Penetration method provided essential data for comparing soil pressure under the structures with the soil's bearing capacity (Ahmed, 2016).

Additionally, the strength of the concrete used in the building was checked using a concrete scanner and rebound hammer (Reaffirmed, 2013). The results is intended to indicate whether the soil pressure exceeded the soil bearing capacity. If it did, appropriate strengthening methods were developed to ensure structural stability.



Figure 5: Soil Test and Dynamic Cone Penetration (DCP) Test,

Source : <https://www.proceq.com/company/history>

3.3.3. To provide the recommendation that will guide to prevent the roads cracks and building settlement

The methodology begins with a thorough literature review to investigate the causes, mechanisms, and strategies for mitigating road cracks and building settlements, ensuring a comprehensive understanding of current practices in the field. Data collection follows, involving extensive field surveys: a topographical survey using advanced tools to map terrain features and potential geological hazards affecting infrastructure stability; a hydrology study to assess water flows, groundwater levels, and drainage patterns impacting roads and buildings; a geotechnical assessment employing tests like Dynamic Cone Penetration (DCP) to evaluate soil strength, bearing capacity, and settlement potential; and a structural assessment using non-destructive methods such as concrete scanners and rebound hammers to evaluate existing structure conditions and load-bearing capacity.

Subsequently, the gathered data undergoes analysis to identify correlations among geological, hydrological, and structural factors contributing to road cracks and building settlements. This analysis aims to pinpoint critical risk areas and factors influencing infrastructure stability. Based on these findings, specific recommendations are developed. These recommendations include preventive measures such as enhancing drainage systems, improving soil stabilization techniques, and implementing rigorous quality control during construction. Maintenance practices are also proposed to establish protocols for regular monitoring, early detection of deterioration through structural inspections, and timely repairs. Additionally, guidelines for adaptive and resilient infrastructure design are formulated, considering local geological conditions, climate variability, and anticipated future changes.

Throughout this process, input from engineering experts, stakeholders, and local authorities is sought to validate the recommendations. Feedback is incorporated to ensure practicality, feasibility, and alignment with local regulatory frameworks and development goals. This collaborative approach ensures that the methodology not only addresses current challenges but also anticipates future needs, fostering sustainable infrastructure development and resilience against environmental and operational pressures.

3.4. METHOD FOR DATA PROCESSING AND ANALYSIS

This section details the software and equipment used to process, analyze, and interpret data generated from the field.

Software and Tools:

- AutoCAD and Covadis (Hoang_Hung Tran Nguyen, 2017): These were used for analyzing data related to road construction and structural analysis.
- ArcGIS 10.8: This was employed for spatial data analysis and mapping, particularly for assessing building settlement.
- Prokon or Prota Structure Software (Atmaca, 2018): These were utilized for analyzing the stability of building settlements.

Soil Data Analysis:

- Penetration Resistance: Soil data were analyzed based on the number of blows required to drive cones a specific length into the soil. This method helped determine soil strength and consistency.

- **Bearing Capacity Evaluation:** To ensure accuracy, random checks and manual calculations were performed to compare with computational results. The choice of research formulas or standards was based on the collected soil data and geological type. The stratigraphical column resulting from core descriptions and laboratory tests was created, with corresponding SPT values presented on a single sheet for clarity.

Digital Elevation Models (DEM) and Satellite Imagery:

- **DEM Data Acquisition and Processing:**
 - High-resolution DEM data were sourced from reliable repositories to provide detailed terrain characteristics such as elevation, slope, and aspect.
 - Preprocessing steps, including geometric correction, resampling, and mosaicking, were conducted to ensure data compatibility and accuracy.
- **Landsat 8 & 9 Satellite Imagery:**
 - Landsat imagery provided multispectral data to derive biophysical parameters, including soil moisture content.
 - Preprocessing steps, such as atmospheric correction and geometric rectification, were applied to enhance data accuracy.
 - The Soil Moisture Index (SMI) was calculated using suitable algorithms, leveraging spectral information from Landsat bands. SMI aided in characterizing soil moisture distribution and its influence on runoff generation.

Soil Type Data Acquisition and Processing:

The next step involved obtaining soil type data, a critical component in hydrological modelling for assessing infiltration rates and water retention capacities. Soil type data are sourced from reputable sources such as soil survey databases or remote sensing products. These data are downloaded and processed to create a spatial layer representing soil types within the study area. Preprocessing steps may include data cleaning, reclassification, and spatial interpolation to ensure consistency and compatibility with other datasets.

Integration and Analysis in ArcGIS:

ArcGIS served as the primary platform for integrating and analysing diverse geospatial datasets for hydrological modelling. The methodology involves importing the processed DEM, Landsat imagery, and soil type data into ArcGIS environment. Spatial analysis techniques, including

watershed delineation, hydrological modeling, and spatial interpolation, are applied to derive relevant hydrological parameters such as flow accumulation, stream networks, and runoff potential. Integration of these datasets enables the creation of a comprehensive hydrological model, which serves as a basis for understanding watershed behavior and assessing water resource management strategies.

CHAPTER 4: FINDINGS.

In this pivotal chapter, we delve into the core findings of the research, which illuminate various facets of road cracks and building settlement at Gatuna OSBP. Our investigation rigorously examined the major causes behind these infrastructural challenges, assesses the stability of existing buildings within the vicinity, validates hypotheses proposed in earlier chapters, and outlines effective strategies for managing and preventing road cracks and building settlement.

4.1 MAJOR CAUSES OF ROAD CRACKS AND BUILDING SETTLEMENT

In this, we present the findings and results of our study, focusing on utilizing Geographic Information System (GIS) technology to identify and analyze the main causes of road cracks and settlement issues in the Gatuna One-Stop Border Post (OSBP) area.

4.1.1 Assessment using GIS Technology

The assessment of road infrastructure conditions along the Kigali-Gatuna corridor is a critical aspect of ensuring sustainable transportation and infrastructure development.

One of the primary causes identified for road cracks was the geological composition of the region. GIS analysis revealed areas with a high propensity for ground movement, such as subsidence and landslides, contributing significantly to road degradation. Additionally, factors like **traffic volume, road design, and maintenance practices were integrated into our GIS models to provide a holistic understanding of the road condition dynamics.** The geologic map presented herein delineates the spatial distribution of lithologic components within the study area, providing a comprehensive overview of its geological framework. Through detailed analysis, three predominant lithological units have been identified: organic material, quartzite+shale, and shale. These lithologic components offer insights into the geological history and depositional environments of the region, shedding light on the complex interplay of processes shaping its present-day landscape. This map serves as a foundational tool for further research endeavors aimed at understanding the geological evolution and resource potential of the study area, see figure 7.

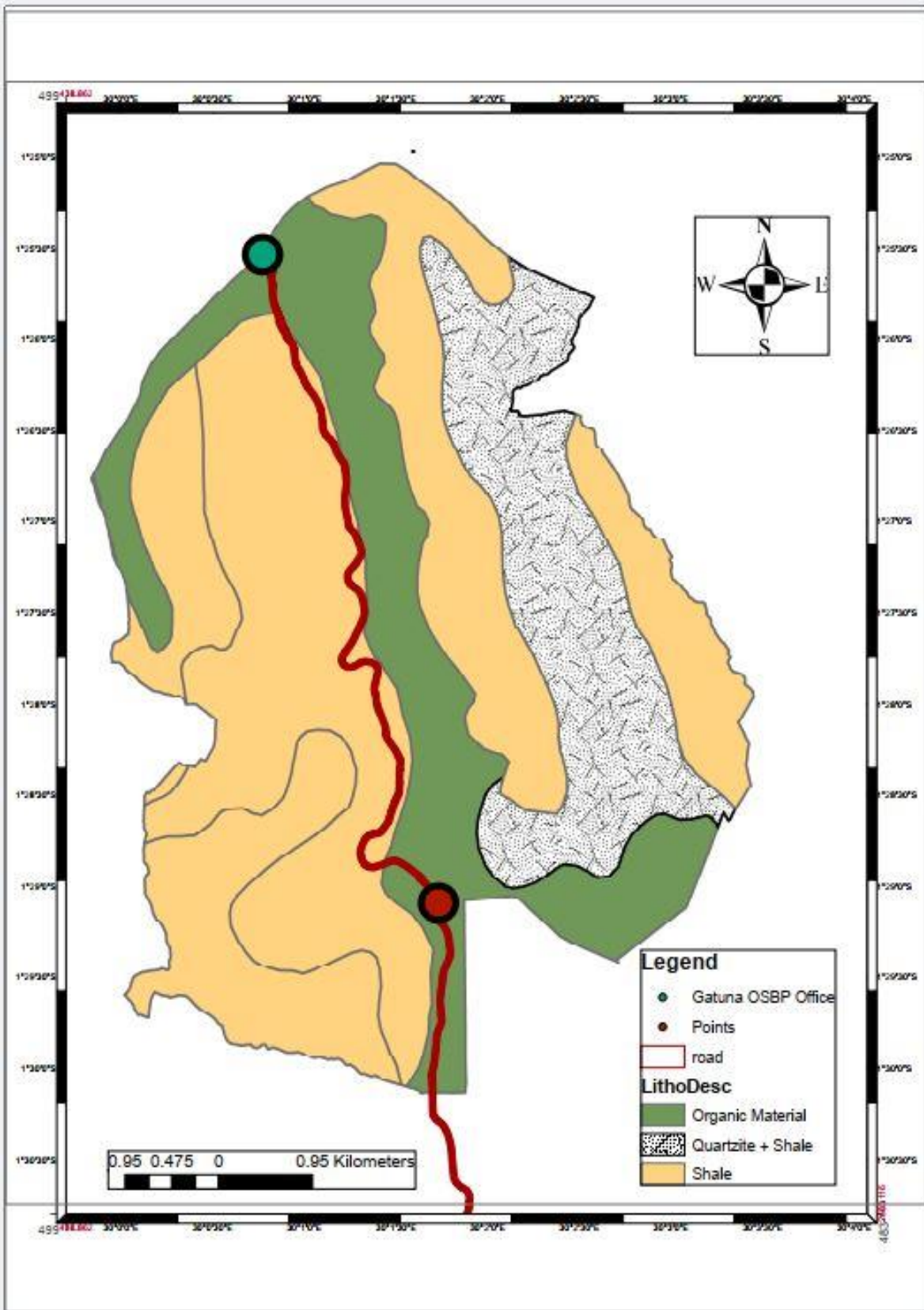


Figure 7: Geological Map of the study area

Source : <https://www.fao.org/land-water/land/httpwwwfaorgsoils-portalen>

The settlement of building Land subsidence and inadequate foundation support emerged as prominent contributors to building settlement. GIS mapping allowed us to pinpoint vulnerable zones where structural adjustments and preventive measures could be implemented to mitigate settlement risks. As indicated in the map above, both the road and building infrastructure have been constructed upon organic material and shale from a lithological perspective

Through analysis of topographical features in study area, the area spanning elevations of 1805 to 2248 feet features a serene valley adorned by the lush Gatuna wetland. An asphalt road traverses the lower reaches of the valley, providing easy access and leading to Gatuna Boarder post.

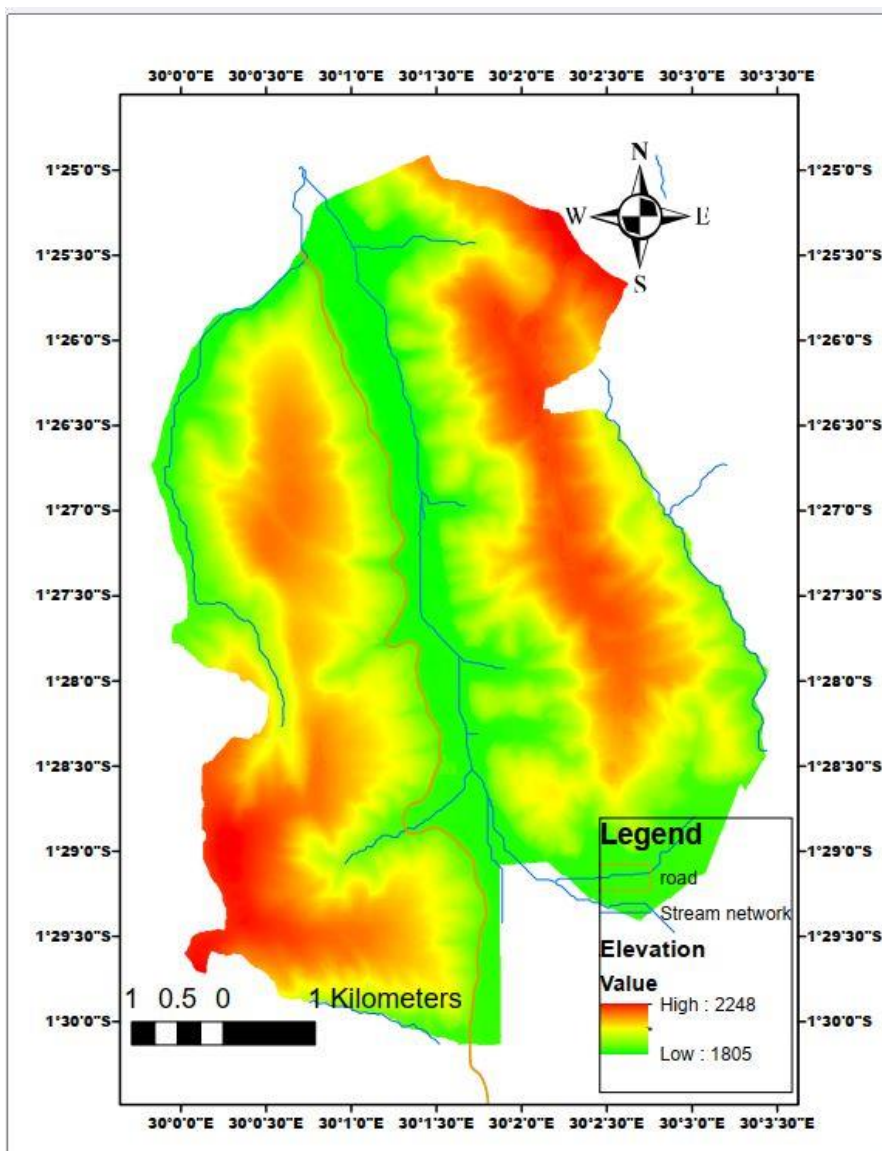


Figure 6: Topographic map in study area

Source : <https://geohub.data.undp.org/data>

4.1.1.1 Constructing a road on shale rock

Geological considerations and potential challenges according to stability issues: Shale is a sedimentary rock known for its relatively weak and fossil nature. It tends to fracture along bedding planes, making it susceptible to slope instability and landslides, especially when subjected to external forces such as road construction activities, heavy rainfall, or seismic events. This has posed significant hazards to road users and nearby infrastructure.

Erosion and Weathering: Shale is also prone to weathering and erosion, particularly when exposed to water and freeze-thaw cycles. The construction of roads can accelerate these processes by increasing surface runoff and altering natural drainage patterns. Over time, this led to the degradation of road surfaces and the undermining of road foundations, requiring costly maintenance and repair efforts.

Poor Load-Bearing Capacity: Shale typically has lower load-bearing capacity compared to harder rocks like granite or basalt. This resulted in settlement or deformation of the road surface under the weight of vehicles, leading to uneven or damaged roadways and reduced safety for motorists. In this research the major part of the road more than **80% from Maya trade center and Gatuna** OBP Office is on shale (see figure 7).

4.1.1.2 Roads and building infrastructure constructing on organic material

This construction led to Settlement and Subsidence: Organic material, such as peat found already in Gatuna wetland or organic-rich soils, is often compressible and prone to settling over time. This led to uneven settlement beneath roads and buildings, resulting in uneven surfaces, structural damage, and potential safety hazards. The organic material typically has a low bearing capacity compared to mineral soils and rocks. This means it may not provide adequate support for road and building foundations, increasing the risk of structural deformation, settlement, and instability under heavy loads. The Organic material has high water retention capacities and can become waterlogged easily. Constructing roads and buildings on such material can exacerbate drainage issues, leading to prolonged periods of saturation and increased susceptibility to erosion, especially in areas with high rainfall or poor natural drainage. The organic decomposition: Over time, organic material undergoes decomposition, which can result in changes in volume, strength, and stability.

4.1.1.3 Soil Moisture index

Soil moisture plays a significant role in both road infrastructure and building foundations. Here are some impacts of soil moisture on road cracks and building settlement:

1.Road Cracks:

Expansion and Contraction: Soil expands when it absorbs moisture and contracts when it dries out. This continuous cycle of expansion and contraction can exert pressure on the road surface, leading to cracks over time.

Weakens Road Base: Excessive moisture can weaken the base layers of the road, reducing its load-bearing capacity and increasing susceptibility to cracks.

Subsidence: Saturated soil beneath the road can cause subsidence, resulting in depressions or uneven surfaces, which can eventually lead to cracking.

2.Building Settlement:

Shrink-Swell Effect: Clayey soils are particularly susceptible to changes in moisture content. When soil becomes saturated, it expands, and when it dries out, it contracts. This continuous expansion and contraction can lead to differential settlement of buildings, causing cracks in walls and foundations.

Erosion: High moisture content can cause soil erosion around the foundation of buildings, leading to settlement and instability.

Hydrostatic Pressure: Excessive moisture in the soil can exert hydrostatic pressure on the foundation walls of buildings. Over time, this pressure can cause the walls to crack or bow inward.

3.Preventive Measures

Proper Drainage: Implementing effective drainage systems to divert excess water away from roads and buildings can help mitigate the effects of soil moisture.

Foundation Waterproofing: Waterproofing building foundations can prevent moisture penetration into the soil surrounding the foundation, reducing the risk of settlement.

Regular Maintenance: Regular inspection and maintenance of roads and buildings can help identify early signs of damage caused by soil moisture and prevent further deterioration.

In conclusion, soil moisture significantly affects the stability and longevity of road infrastructure and building foundations. Understanding these impacts and implementing appropriate preventive measures are essential for ensuring the durability and safety of roads and buildings in areas prone to soil moisture fluctuations.

Negative standard normal values indicate dry soil conditions, whereas positive values indicate wet soil conditions. The index values range from 0 to 1 where 0 indicates extreme dry conditions and 1 indicates extreme wet conditions. The fig.9 shows Negative standard normal values on the Soil Moisture Index (SMI) map, indicating dry soil conditions, that has significant implications for road construction and building projects. When soil moisture content is low, it affects the engineering properties of the soil, which in turn impacts construction activities and the long-term stability of structures. The result is :

Compaction Challenges: Dry soils are often more difficult to compact effectively compared to moist soils. Insufficient compaction can lead to reduced soil density and stability, increasing the risk of settlement and structural failure over time. Road construction projects require adequately compacted soil layers to ensure the durability and performance of pavements.

Foundation Instability: Dry soil conditions may compromise the load-bearing capacity of foundations for buildings and infrastructure. Shrinking and swelling of expansive soils due to changes in moisture content can result in foundation movement and structural damage. Proper site investigation and soil stabilization measures are essential to mitigate the risk of foundation failure.

Pavement Cracking: In road construction, dry soil conditions increase the likelihood of pavement cracking and distress due to differential settlement and moisture-induced stresses. Adequate subgrade moisture management and pavement design considerations, including proper drainage and base course materials selection, are essential to minimize pavement deterioration and maintenance costs.

In addition, as the water table drops due to an extended drought, subsoil moisture decreases, resulting in soil shrinkage. The foundation's support is removed as the soil shrinks, which may cause movement in the foundation. This movement can cause the structure to shift, resulting in doors and windows that stick because the walls no longer are square, and cracks that appear in wall sheeting such as drywall. Differential foundation movement may stress the concrete enough to cause a crack to form in a basement wall. Another problem with a gap between the soil and basement wall is that it permits rainwater to run down the wall and may result in water

getting into the basement. Soil shrinkage also will occur under concrete slabs such as basement floors or driveways if the support base is clay or other fine-particle soil that swells and shrinks with changes in moisture content, according to Hellevang. To minimize these problems, he recommends maintaining a relatively constant moisture content in the soil around the house. Water the soil evenly around the entire building during extended dry periods if the soil is pulling away from the basement wall. Apply water 1 to 2 feet away from the wall to wet the soil without getting water into the gap between the soil and house (Ken Hellevang, 701-231-7243, kenneth.hellevang@ndsu.edu).

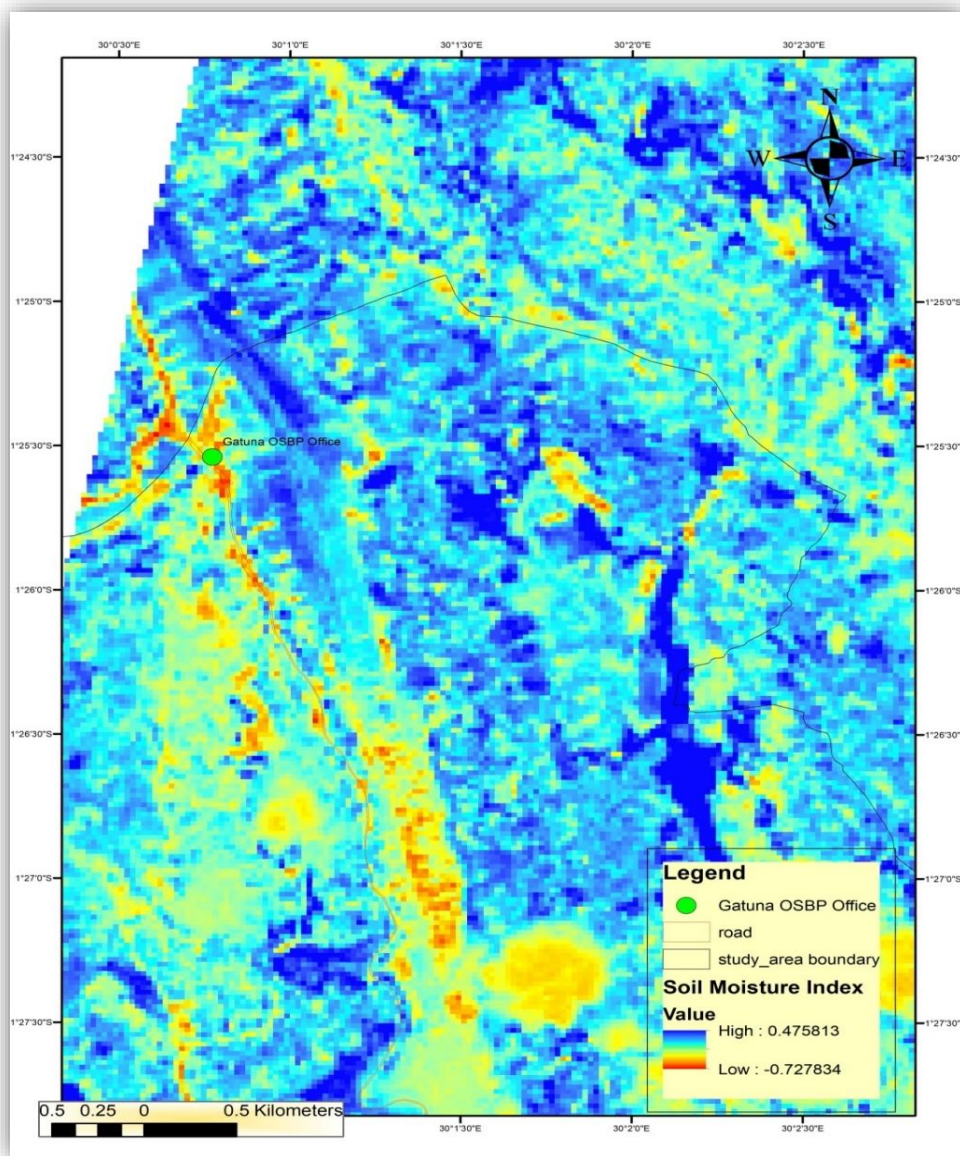


Figure 7: Map of Soil Moisture index of study area

Source : <https://earthexplorer.usgs.gov>

4.2 STABILITY OF EXISTING BUILDING AT GATUNA OSBP

Finding package necessary to ascertain stability of the existing buildings and the road and the thesis has provided details on the following research activities:

- i) **Topographical survey:** *A focus should be given to a detailed topographical survey to guide the consultant when deciding on the solutions to be proposed in the exact locations of the structures under the scope of work of this study;*
- ii) **Hydrology study:** *the consultant services consist in conducting the Hydrological Study of the Site. This will serve to determine current groundwater level, the flow direction, and the flood area for a long-term period and analysis of the surface and subsurface drainage systems;*
- iii) **Geotechnical Assessments:** *In this phase, the expertise will consist in the analysis of soil foundation of all footings of the building and load transmitted to foundations and checking of stability for both the retaining wall structures and road section close to the swamp under the scope of this study;*
- iv) **Structural analyses:** *In this phase, the expertise will consist in the analysis of all structure members of both the building and the retaining wall;*

4.2.1. Topographical Survey

The following are the components of this point

- i) Contour lines;
- ii) Position of the structures, roads, etc... in the vicinity of the crack;
- iii) The location of the crack with respect to other structures;
- iv) Location of exploration points. (See Appendice No : 1 *Figure showing the Plan of the site and geometry of the Slope*)

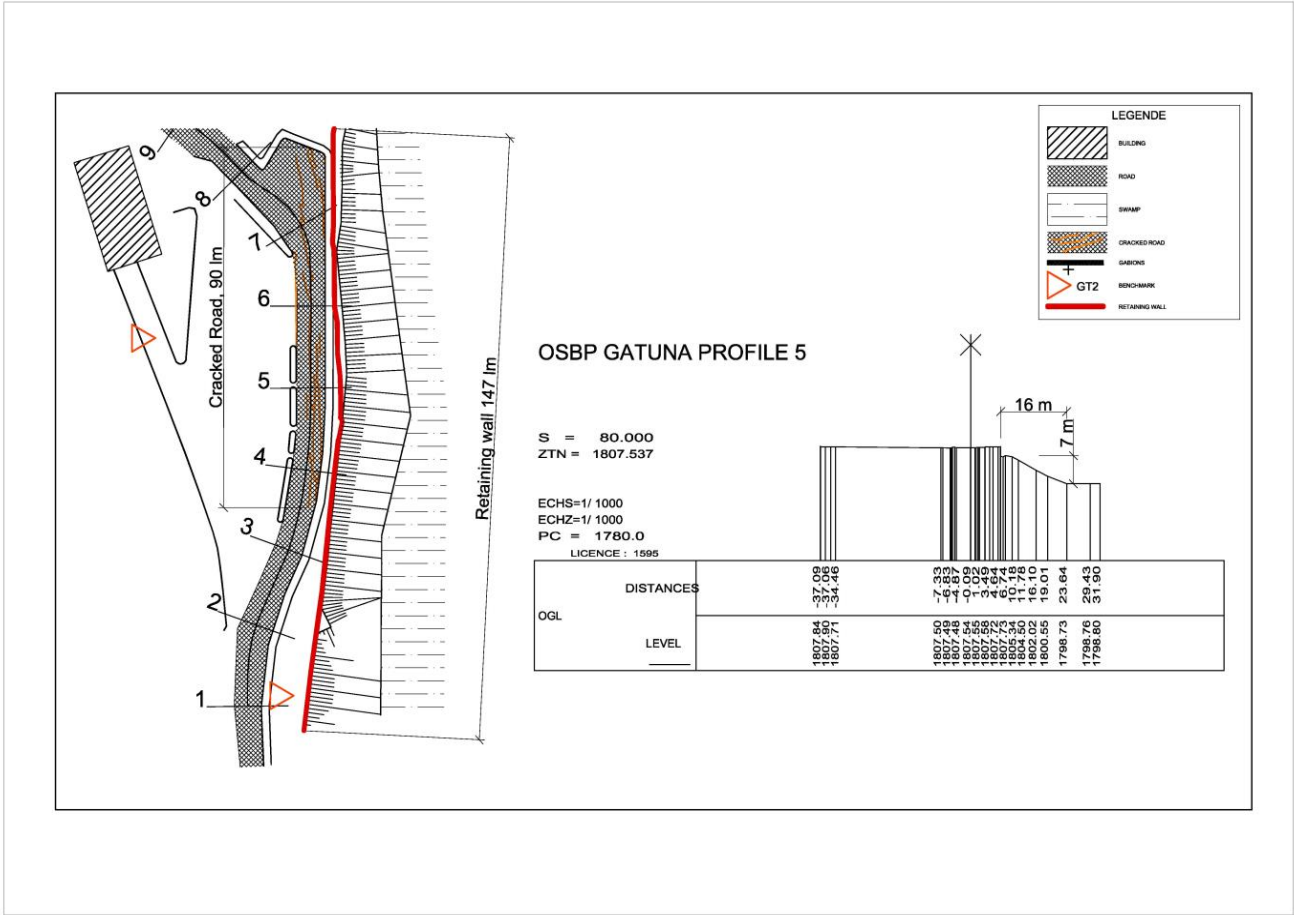


Figure 8:OSBP Gatuna Profile

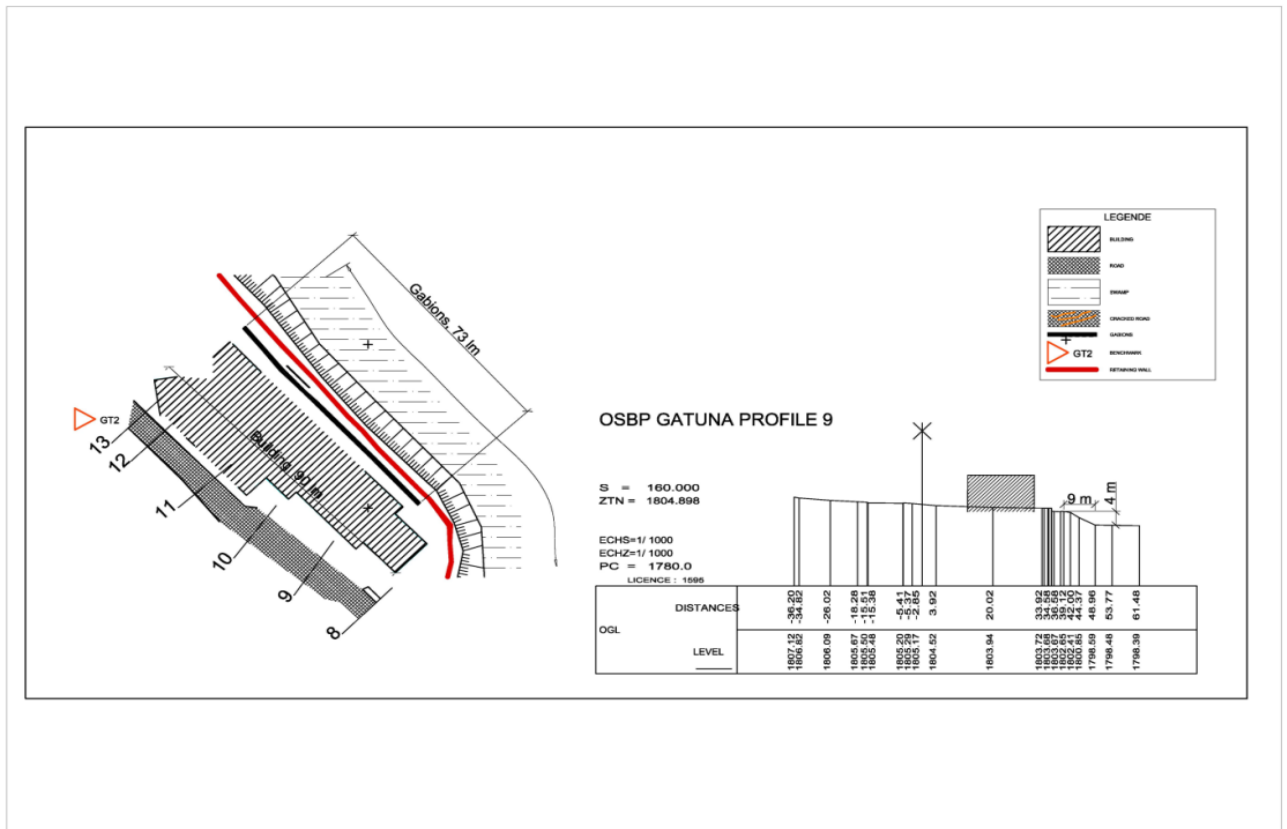


Figure 9: OSBP Gatuna Profile

So, a well-organized site layout with enough size is attached to this thesis for more details.

4.2.2. Existing Retaining Wall

During the topographical survey, we have recorded a length and thickness of a concrete retaining wall starting from profile n°1 to profile n°8 for a total length of 147 lm, an average thickness of 40 cm. and height of approximately 4.8 meters deep.

4.2.3 Hydrology

Here is the results of evaluation of the following points:

- the specific Metrology and hydrology of the site;
- the groundwater level and flow direction;
- the flood area;
- the Surface and subsurface drainage and sanitation works;

This Hydrological study specifically has focused on the following activities:

- To identify the possible aquifer and its type;

- To provide the characteristics of the aquifer and hydrodynamics parameters (e.g: thickness of saturated zone, hydraulic conductivity and transmissivity)
- To analyze the peak flow that can occur in the project area;
- To propose the drainage system for ensuring the stability of the upper geological layers supporting the building, road and other structures.

Thereafter, a hydrological study was conducted by determining the hydraulic conductivity, and transmissivity of different geological layers of the site and characterizing the aquifers

This research was about the factual data collected from the drilling works carried out on five (05) boreholes done in June 2019. With respect to the geological study, the following has been observed:

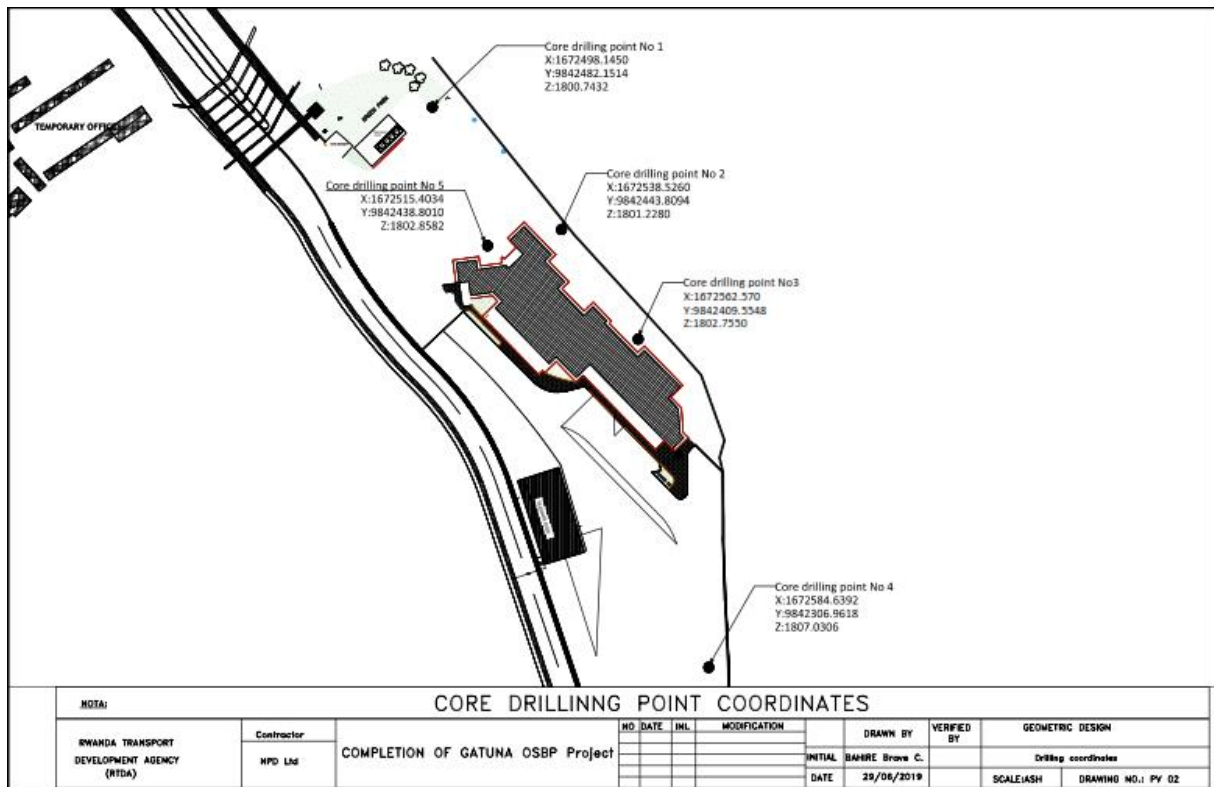


Figure 10: Core drilling points

-On borehole **BH-01**: the first 3 m depth from the ground surface is formed with sand with some gravel that define it as an aquifer. However, no any information was provided regarding the ground water with that geological formation. Below that aquifer, after 2.2m of impervious layer of clay, there is another geological formation for which its material (sand and some gravels) is favourable to the transfer of water. Therefore, there was a need of having their

hydraulic conductivity value to know the extent of that formation regarding the water storage and transfer.

-On borehole BH-02: the first 5.2m represent the geological formation of an aquifer but no information was provided regarding the groundwater. An aquifer is defined as an underground layer of water-bearing permeable rock or unconsolidated materials (gravel, sand, silt) from which groundwater can be usefully extracted. It contains water and allows water to pass through it. The same to the BH-01, there is a need of deeply characterizing that formation in order to determine to potential flow rate that should be evacuated from.

-On borehole BH-03: excluding the pavement with backfill material found on first 1.5 m, the depth of 13.5m represent the material that define the geological formation as an aquifer.

-On borehole BH-04: Below 2 m depth from the ground surface there is a geological formation of 4.5m that can be defined as an aquifer, but no information has been provided regarding the groundwater. The same information as above is needed.

-On borehole BH-05: There is a confined aquifer (sand material) below 1.5 m of silt and above 2.5m of very soft black clay. Beyond the clay layer there is a geological formation of an aquifer of around 5m depth.

A confined aquifer is an aquifer below the land surface that is saturated with water. Layers of impermeable material are both above and below the aquifer, causing it to be under pressure so that when the aquifer is penetrated by a well, the water will rise above the top of the aquifer.

A water-table or unconfined aquifer is an aquifer whose upper water surface (water table) is at atmospheric pressure, and thus is able to rise and fall. Water-table aquifers are usually closer to the Earth's surface than confined aquifers are, and as such are impacted by drought conditions sooner than confined aquifers.

The stratigraphy of geological layers of those aforementioned boreholes is summarized in Fig. 15 where the height of row represents 0.5 m of thickness of layer. (*see Appendice3 : Stratigraphic Layer of the drilled boreholes*)

The analysis showed that the area concerned with the five boreholes drilled was found to be of unconfined aquifer with variable thickness except on BH-05 which has a layer of 1.5 m of a pavement with backfill material that results into the unconfined aquifer. Thus, there is a risk of a rise of water table and this can destabilize the structure. Ground below and immediately

adjoining a dwelling that is liable to accumulate groundwater, at a level that could affect the structure or penetrate the building, requires subsoil drainage or other dewatering treatment to be provided to mitigate against the harmful effects of such water.

Capillary rise may occur in soil above the water table. Capillary rise can cause deterioration to structures formed from sandstone through dissolution of cementing minerals reducing the strength of stone and recrystallization of dissolved salts leading to expansion of the stone.

Rises in groundwater level, can cause reductions in strength of the soil that can lead to failures of slopes. In regions of significant slope instability, significant damage to buildings can occur as a result of landslides.

4.2.3.1. Site visit

On 05th February 2020, we were informed that there is a rock fill from 1 m depth to 2 m depth from the ground surface and this was confirmed through the trial pit dug between the retaining wall and the building.

4.2.3.2 Determination of hydraulic conductivity

Hydraulic conductivity, symbolically represented as K , is a property of vascular plants, soils and rocks, that describes the ease with which a fluid (usually water) can move through pore spaces or fractures. It depends on the intrinsic permeability of the material, the degree of saturation, and on the density and viscosity of the fluid. Saturated hydraulic conductivity, K_{sat} , describes water movement through saturated media. By definition, hydraulic conductivity is the ratio of velocity to hydraulic gradient indicating permeability of porous media (Oosterbaan, 1994).

The hydraulic conductivity of a soil is the fundamental physical property necessary when designing an underground drainage system. It is none other than the proportionality coefficient of Darcy's law connecting the flow of fluid to the hydraulic gradient.

$$Q = -KA \frac{dh}{dl}$$

where: **K**: Hydraulic conductivity m/s

A: Cross sectional area (m²)

dh/dl: hydraulic gradient (water level difference over distance)

The empirical formulas are often used in engineering practice for determination of hydraulic conductivity from the grain-size composition of the studied porous media. The hydraulic conductivity was determined by using the Slichter's formula;

$$K = 1 \times 10^{-2} \times \frac{g}{\nu} \times n^{3.287} \times (d_{10})^2$$

Where: K: Hydraulic conductivity;

g: acceleration of gravity

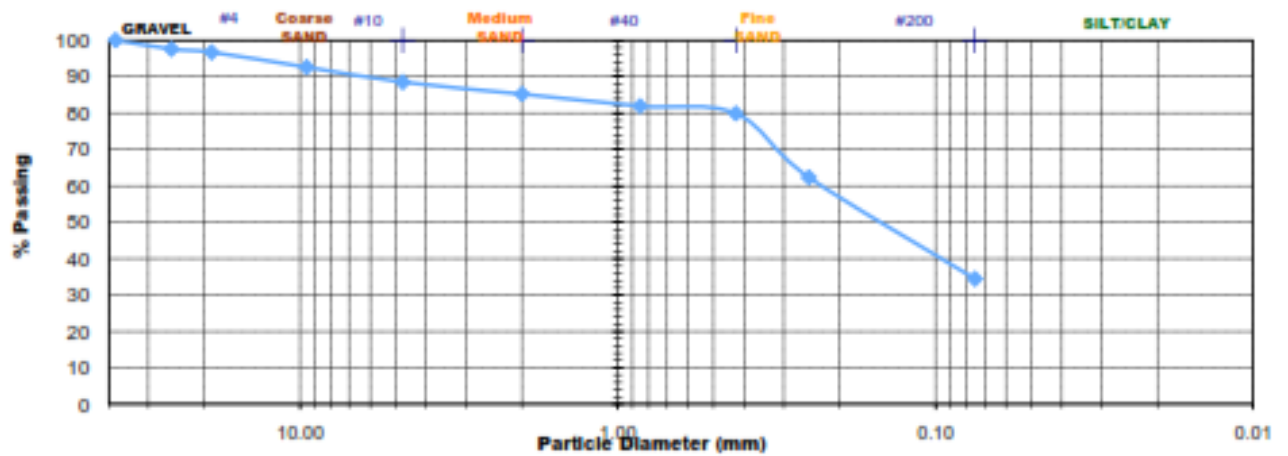
ν : kinematic viscosity

n: porosity

d_{10} : effective grain size

Slichter's formula (Slichter, 1898) uses potential theory in groundwater to quantitatively describe the steady-state flow field in response to a discharging well, and according to the USGS, it could be the first quantitative analysis of groundwater. The equation is derived after Slichter's multiple laboratory experiments of water moving through porous soils or rock under different pressures. As originally published in Slichter (1898), the formula makes use of an average grain-size, does not take into consideration the shape of the grains, and is simplified for water at 10°C. Several researchers (Vukovic and Soro, 1992; Odong, 2007; Cheng and Chen, 2007) tested this formula with different materials, and their findings indicate that it is most reliable in sand samples with grain sizes between 0.01 and 5mm with no restrictions on the uniformity coefficient of the granular media.

Sieve Number	Diameter (mm)	Mass of Sieve (g)	Soil Retained (g)	Cumulative Retained (g)	Cumulative Retained (%)	Soil Passing (%)
1-1/2"	38.10		0.0	0.0	0.0	100.0
1"	25.40		27.3	27.3	2.4	97.6
3/4"	19.00		9.9	37.2	3.3	96.7
3/8"	9.52		46.1	83.3	7.4	92.6
#4	4.75		46.5	129.8	11.5	88.5
#10	2.00		36.6	166.4	14.8	85.2
#20	0.85		37.1	203.5	18.0	82.0
#40	0.425		23.1	226.6	20.1	79.9
#60	0.250		198.2	424.8	37.7	62.3
#200	0.075		314.7	739.5	65.6	34.4
Pan			11.9	751.4	66.6	0.0



Grain Size Distribution Curve Results:

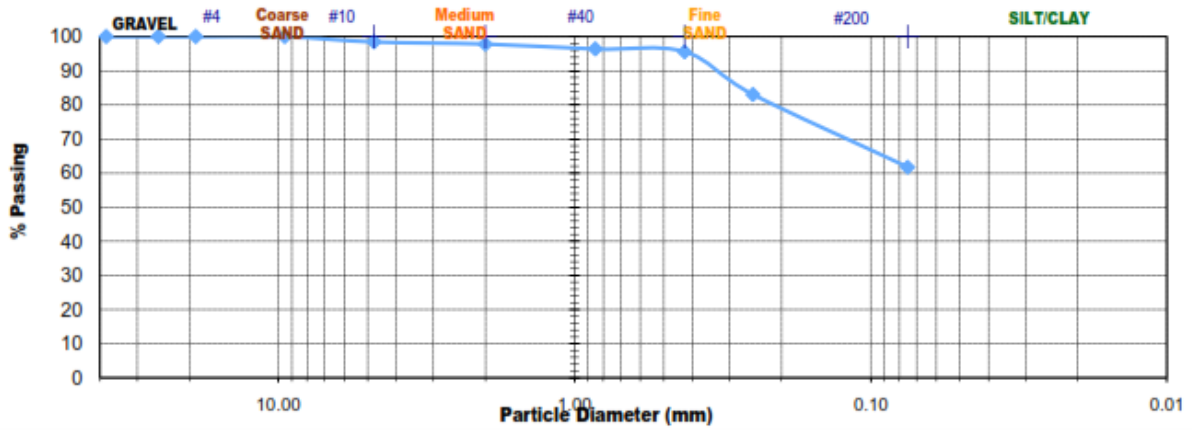
% Gravel: 11.5
 % Sand: 54.1
 % Fines: 34.4

$D_{10}^{\%}$: 0.019
 $D_{30}^{\%}$: 0.06
 $D_{60}^{\%}$: 0.23

$C_u^{\%}$: 12.1
 $C_c^{\%}$: 13.7

Figure 11: Results from sieve analysis test for BH-01

Sieve Number	Diameter (mm)	Mass of Sieve (g)	Soil Retained (g)	Cumulative Retained (g)	Cumulative Retained (%)	Soil Passing (%)
1-1/2"	38.10		0.0	0.0	0.0	100.0
1"	25.40		0.0	0.0	0.0	100.0
3/4"	19.00		0.0	0.0	0.0	100.0
3/8"	9.52		0.0	0.0	0.0	100.0
#4	4.75		11.4	11.4	1.6	98.4
#10	2.00		4.7	16.1	2.2	97.8
#20	0.85		10.5	26.6	3.7	96.3
#40	0.425		5.7	32.3	4.4	95.6
#60	0.250		91.2	123.5	17.0	83.0
#200	0.075		154.8	278.3	38.3	61.7
Pan			1.3	279.6	38.4	0.0



Grain Size Distribution Curve Results:

% Gravel: 1.6
 % Sand: 36.7
 % Fines: 61.7

D₁₀: 0.015
 D₃₀: 0.018
 D₆₀: 0.07

C_u: 4.7
 C_{ci}: 17.1

Figure 12: Results from the sieve analysis test for BH-02

Sieve Number	Diameter (mm)	Mass of Sieve (g)	Soil Retained (g)	Cumulative Retained (g)	Cumulative Retained (%)	Soil Passing (%)
1-1/2"	38.10		0.0	0.0	0.0	100.0
1"	25.40		0.0	0.0	0.0	100.0
3/4"	19.00		0.0	0.0	0.0	100.0
3/8"	9.52		1.9	1.9	0.2	99.8
#4	4.75		7.5	9.4	0.8	99.2
#10	2.00		5.3	14.7	1.3	98.7
#20	0.85		6.4	21.1	1.8	98.2
#40	0.425		9.2	30.3	2.6	97.4
#60	0.250		139.3	109.6	14.7	85.3
#200	0.075		176.2	347.8	30.2	69.8
Pan				351.0	30.5	0.0

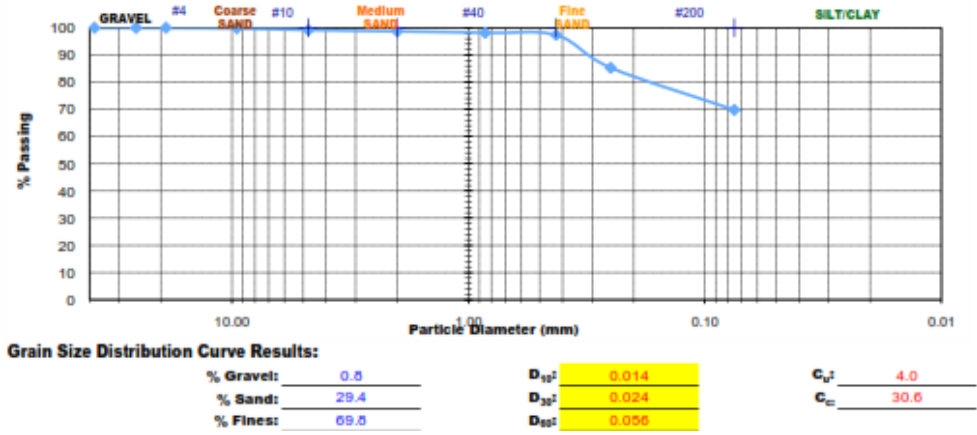
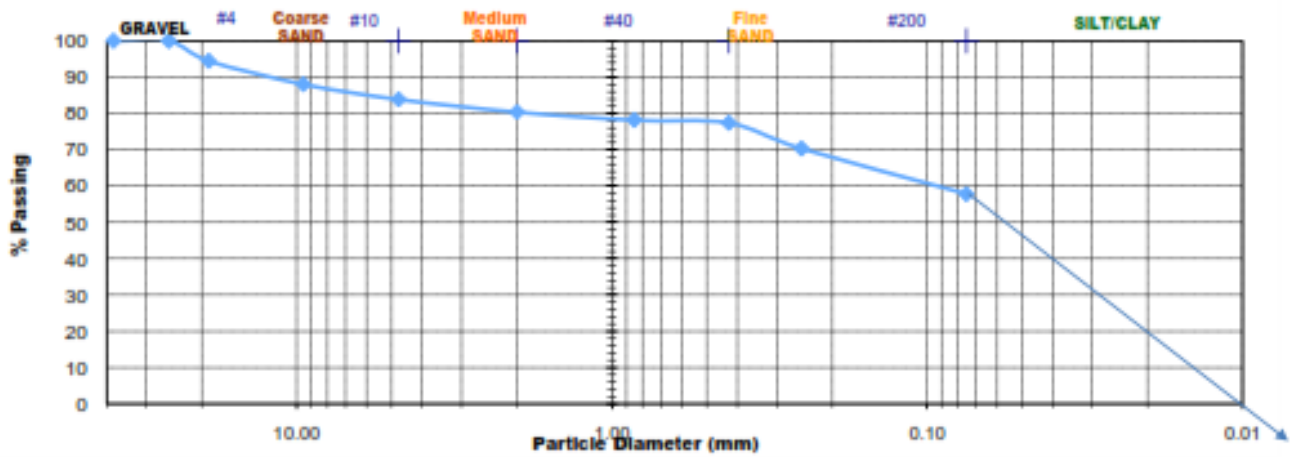


Figure 13: Results from the sieve analysis test for BH-03

Sieve Number	Diameter (mm)	Mass of Sieve (g)	Soil Retained (g)	Cumulative Retained (g)	Cumulative Retained (%)	Soil Passing (%)
1-1/2"	38.10		0.0	0.0	0.0	100.0
1"	25.40		0.0	0.0	0.0	100.0
3/4"	19.00		72.3	72.3	5.5	94.5
3/8"	9.52		63.6	156.1	11.9	88.1
#4	4.75		54.5	210.6	16.1	83.9
#10	2.00		45.7	256.3	19.6	80.4
#20	0.85		29.3	285.6	21.9	78.1
#40	0.425		9.9	295.5	22.6	77.4
#60	0.250		92.7	388.2	29.7	70.3
#200	0.075		163.2	551.4	42.2	57.8
Pan			3.9	555.3	42.5	0.0



Grain Size Distribution Curve Results:

% Gravel: 16.1
 % Sand: 26.1
 % Fines: 57.8

D₁₀: 0.015
 D₃₀: 0.029
 D₆₀: 0.095

C_u: 6.3
 C_c: 20.4

Figure 14: Results from the sieve analysis for BH-04

Sieve Number	Diameter (mm)	Mass of Sieve (g)	Soil Retained (g)	Cumulative Retained (g)	Cumulative Retained (%)	Soil Passing (%)
1-1/2"	38.10		0.0	0.0	0.0	100.0
1"	25.40		0.0	0.0	0.0	100.0
3/4"	19.00		44.5	44.5	3.6	96.4
3/8"	9.52		59.4	103.9	8.4	91.6
#4	4.75		60.4	164.3	13.3	86.7
#10	2.00		59.6	223.9	18.1	81.9
#20	0.85		49.7	273.6	22.1	77.9
#40	0.425		13.6	287.2	23.2	76.8
#60	0.250		146.0	433.2	35.0	65.0
#200	0.075		167.6	600.8	50.2	49.8
Pan			2.0	623.6	50.4	0.0

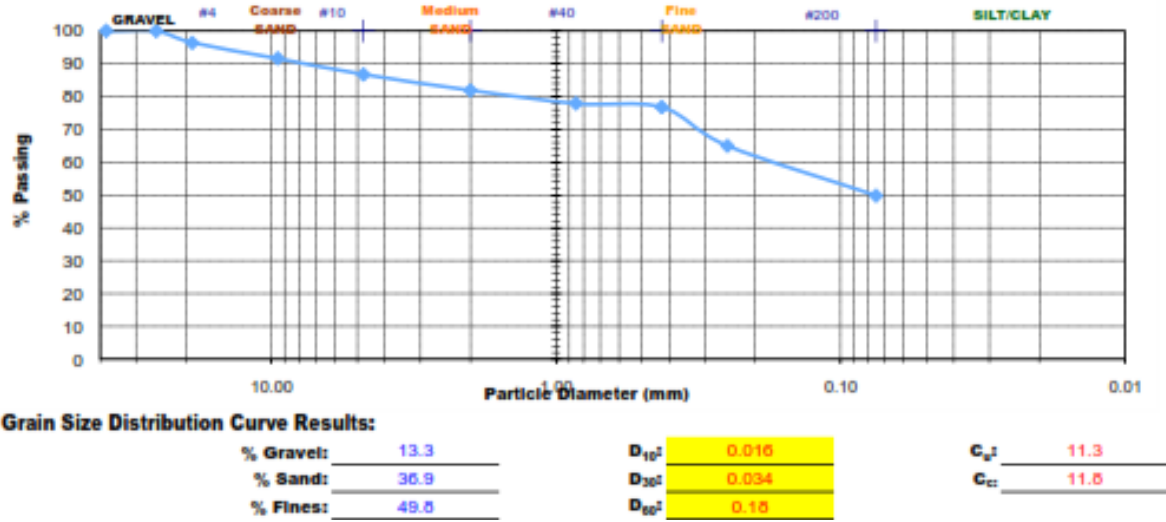


Figure 15: Results from the sieve analysis for BH-05

Therefore, considering the sieve analysis results from the geotechnical study done by RMTC Ltd, the formula of Slichter (1898) was used for determination of the hydraulic conductivity of different geologic layers.

The hydraulic conductivity was determined in reference to the types of soil that were found on site project (refer to figure 1). For this project, the hydraulic conductivity was calculated based on the results from sieve analysis performed by RMTC Ltd. The findings on hydraulic conductivity are depicted in Table 2.

Table 1: Hydraulic conductivity and transmissivity for the five boreholes

Depth(m)	K (cm/s)	K (m/sec)	T (m ² /s)
BH1			
0-3m	0.25	0.0025	0.0075
3-5m	0.00001	1E-07	0.0000002
5-6m	0.25	0.0025	0.0025
6-10,5	1	0.01	0.045
10,5-15	0.001	0.00001	0.000045
BH2			
0-2,5M	0.001	0.00001	0.000025
2,5-4,5	0.0055	0.000055	0.00011
4,5-5,5	0.01	0.0001	0.0001
5,5-9	0.00001	1E-07	0.00000035
9-10,5	0.5	0.005	0.0075
10,5-15	0.01	0.0001	0.00045
BH3			
0-1,5m	0.000001	1E-08	0.000000015
1,5-4,5m	0.001	0.00001	0.00003
4,5-15	0.001	0.00001	0.000105
BH4			
0-2m	0.00001	1E-07	0.0000002
2-6m	0.001	0.00001	0.00004
6-7,5	0.00001	1E-07	0.00000015
7,5-15m	0.000001	1E-08	0.000000075
BH5			
0-1,5m	0.00001	1E-07	0.00000015
1,5-3m	0.001	0.00001	0.000015
3-5,5m	0.00001	1E-07	0.00000025
5,5-7,5	0.001	0.00001	0.00002
7,5-9,5m	0.01	0.0001	0.0002

After determination of hydraulic conductivity, the pervious layers were confirmed by referring to the typical values of hydraulic conductivity (Table 3).

Table 2: Typical values of K for pervious and impervious layers

K (cm/s)	10^2	10^1	$10^0=1$	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
Relative Permeability	Pervious				Semi-Pervious				Impervious				
<u>Aquifer</u>	Good				Poor				None				

From the analysis of table 2 and by referring to the table 3, the permeable layers that form the aquifer were identified as follows:

BH-1: the unconfined aquifer is of 3 m below the ground surface;

BH-2: the unconfined aquifer is of 5.7 m below the ground surface;

BH-3: due to the pavement with backfill material, the aquifer starts from 1.5 m from the ground surface;

BH-4: the nearest aquifer is from 2 m up to 6 m below the ground surface. BH-5: the aquifer is at 1.5 m from the surface.

The value of hydraulic conductivity (k) varies widely for different soils. Some typical values for saturated soils are given in Table 4. The hydraulic conductivity of unsaturated soils is lower and increases rapidly with the degree of saturation. Therefore, the flow rate of groundwater was determined by :

Table 3: Typical values of Hydraulic conductivity of saturated soils

Soil type	Hydraulic conductivity (cm/sec)
Clean gravel	100-1.0
Coarse sand	1.0-0.01
Fine sand	0.01-0.001
Silty clay	0.001-0.00001
Clay	<0.000001

By referring to these typical values, it was found that the determined values of hydraulic conductivity were in the range of saturated soil.

With the aim of providing the sustainable solution for the existing structure and infrastructure, it is very necessary to provide a drainage system that will help to remove the water or avoid water in the area of foundation. Therefore, the transmissivity, the rate at which groundwater flows horizontally through an aquifer, was determined for further calculations of the discharge that can be collected along the concerned section.

4.2.3.3. Transmissivity

Transmissivity is the rate at which water passes through a unit width of the aquifer under a unit hydraulic gradient.

$$T = K \times D$$

where T is the transmissivity (m²/s), K is the average horizontal conductivity (m/s) and D is the aquifer thickness (m).

Table 5: Calculated transmissivity for different layers

Depth(m)	K (m/sec)	T (m²/s)
BH1		
0-3m	0.0025	0.0075
3-5m	1E-07	0.0000002
5-6m	0.0025	0.0025
6-10,5	0.01	0.045
10,5-15	0.00001	0.000045
BH2		
0-2,5M	0.00001	0.000025
2,5-4,5	0.000055	0.00011
4,5-5,5	0.0001	0.0001
5,5-9	1E-07	0.00000035
9-10,5	0.005	0.0075
10,5-15	0.0001	0.00045
BH3		
0-1,5m	1E-08	0.000000015
1,5-4,5m	0.00001	0.00003
4,5-15	0.00001	0.000105
BH4		
0-2m	1E-07	0.0000002
2-6m	0.00001	0.00004
6-7,5	1E-07	0.00000015
7,5-15m	1E-08	0.000000075
BH5		
0-1,5m	1E-07	0.00000015
1,5-3m	0.00001	0.000015
3-5,5m	1E-07	0.00000025
5,5-7,5	0.00001	0.00002
7,5-9,5m	0.0001	0.0002

From the table 5, it was found that on Borehole 1 and 2 there is an unconfined aquifer that are susceptible to rise water level up to the ground surface. On borehole 3 and 5, there is a semi-permeable layer above the porous layer (identified aquifer). This can also contribute to the rise of groundwater.

Toll et al. (2012) reported that some soils are liable to collapse (reduce in volume) when inundated by water. Collapse is a phenomenon that occurs when a soil exists in a loose state, with large inter-particle voids, supported either by suctions in the pore-water, by cemented bonds between particles or by clay “bridges” between particles (that are themselves supported by suctions). If the soil is wetted, the suctions reduce and/or the cementing is dissolved or removed and the loose state can no longer be supported. When the support between the particles is lost, the soil “collapses” causing a sudden reduction in volume resulting in surface settlements.

Chapman (1999), reported that soils that are susceptible to collapse are anthropogenic fill materials (collapsible fills are likely to contain clayey fines, rather than clean granular fills), loess (a fine-grained wind-blown deposit), residual soils or weakly cemented soils.

Peterson and Wade (1997) describe collapse settlements of buildings in Alberta, Canada constructed on mine waste. The settlements resulted from unanticipated groundwater rise.

Referring to the above literature review and the situation of the project area with the aim of providing the sustainable solution for the existing structure and infrastructure, it is very necessary to provide a drainage system that will help to avoid groundwater in the area of foundation.

Therefore, there was a need of analyzing the rainfall-runoff relationship within the catchment area in order to assess the effect the flooding in the swampy area to the site of the concerned building.

4.2.3.4. Rainfall-runoff relationship analysis

The process of design flow estimation for a watershed from rainfall data generally involves the following main steps: determination of the catchment basins, determination of the time of concentration, design rainfall intensity, Areal reduction factor, and Estimation of runoff.

CATCHMENT AREA ANALYSIS

The establishment of the characteristics of catchment area and conditions of flow was based on a deep recognition made on development plan and on the geographical information system Global mapper v. 17. The topography of the sub-catchments was extracted from the Digital Elevation Model (DEM) with the resolution of 10m, with reference to the conducted topographical survey (especially the levelling).

This cartographic document was used to demarcate the catchment area that drain water towards the outlet in order to analyze the relief and to estimate the physiographic parameters of catchment area (such as surface, length of the talweg, the slope, etc.). The topography was described in terms of slope, soil aspect, and elevation. These three basic characteristics affect the movement and storage of water in the catchment. The catchment boundary was delineated from the Digital Elevation Model (DEM) within the ArcGIS environment. The figure below shows the delineated area for the project.

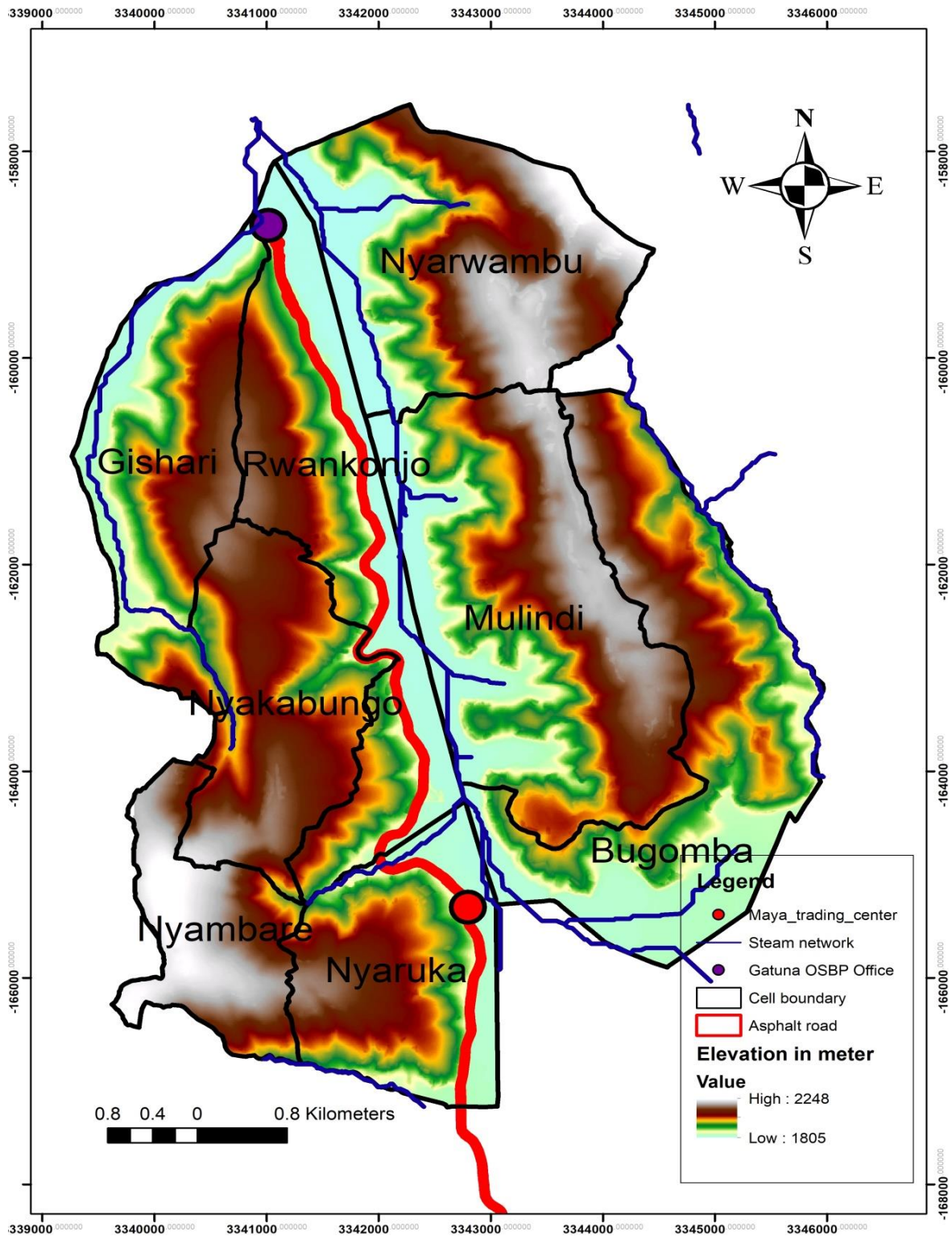


Figure 16: Delineated project area

Source : <https://geohub.data.undp.org/data>

Table 4: Geometric characteristics of the delineated catchment

Catchment	Area (km ²)	Perimeter (m)	Longest flow path (m)	Elevation (m)		Slope
				Max	Min	
Catchment	18.465	19708	5866.28	1950	1810	0.024

These geometric characteristics were used for the estimation of the peak runoff.

A. Time of concentration

Time of concentration (T_c) is the time required for an entire watershed to contribute to runoff at the point of interest for hydraulic design. This time was calculated as the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation.

For the case of this study, the time of concentration was calculated by PASSINI Formula.

PASSINI Formula:

$$T_c = 0.14 \times (A \times L)^{1/3} \times S^{-0.5}$$

Where T_c: Time of concentration (min)

A: watershed drainage area (ha)

L: length of flow (m)

S: Slope of flow (m/m)

B. Peak runoff

$$Q = KCIA$$

Where Q: peak discharge for the return period in m³/s

C: Runoff coefficient

I: Rainfall intensity in mm/hr depending on the time of concentration (t_c) and return period (T)

A: Catchment area in km²

K: conversion factor (0.28)

a) **Areal Reduction Factor**

Generally, for any specified average recurrence interval and duration, the average depth over an area is less than the point rainfall depth. The ratio of the areal average rainfall with a specified duration and average recurrence interval to the point rainfall with the same duration and average recurrence interval is termed as the areal reduction factor.

Areal reduction factors are applied to only design point rainfall intensities. to account for the fact that it is not likely that rainfall will occur at the same intensity over the entire area of a storm. The areal reduction is expressed a factor smaller than 1. For larger catchments, the following areal reduction factor equation is used:

$$ARF = 1 - 0.044 * A^{0.275}$$

Where: A= area of catchment (km²).

C. **Choice of return period**

The choice of the return period should consider the consequences of an overflowing for a more important frequency. In hydrology, design return periods vary typically from 2 years to 100 years, and in countries (or places) where the Probable Maximum Precipitation has not been defined, up to 1000 years. The choice of return period depends on various factors, including the size of the drainage area, the risk of failure, the importance of the structure, and the desired degree of conservatism (Ponce, 2015). In the absence of prior determination of return periods, the choice of return period is based on the type of structure. Hence, the hydraulic structures are designed accordingly.

D. **. Calculation of intensity**

Rainfall intensity (I): Rainfall intensity is the intensity of rainfall in mm per hour for duration equal to the time of concentration. Intensity is the rate of rainfall per hour for the total amount of rainfall over an interval of time such that intensity multiplied by duration equals amount of rain.

Design rainfall intensity represents the average rainfall intensity of duration equal to the time of concentration for the catchment. The total storm rainfall depth at a point, for a given rainfall duration and average recurrence interval, is a function of the local climate. Rainfall depths can be further processed and converted into rainfall intensities (intensity = depth/duration), which

are then presented in the form of “Intensity-Duration-Frequency” (IDF) curves. Such curves are particularly useful in storm water drainage design because many computational procedures require rainfall input in the form of average rainfall intensity.

The rainfall intensity was derived from the Montana Formula:

The Montana-type IDF-curve for precipitation used:

$$i(T, tc) = \frac{a(T)}{(tc + \theta)^\eta}$$

Where tc is the time of concentration and T is the return period of the event. The nominator $a(T)$ is independent of the aggregation times so that the family of curves are parallel. The expression in the nominator and is determined by the inverse function of $a(T)$ (Koutsoyiannis et al., 1998). The denominator $(tc + \theta)^\eta$ of the Montana-type of IDF-curve provides the shape of the IDF-curves; the dynamical parameters θ and η describe the dynamics of the extreme rainfall process in function of the duration and are climate-related. The cumulative distribution function of G is given by:

$$G(x; \mu, \sigma) = \exp\left(-\exp\left(-\frac{x - \mu}{\sigma}\right)\right)$$

Where x is the function variable. μ and σ are the parameters of the Gumbel distribution function. They are called the location and scale-parameter, respectively. To be more precise, the location-parameter μ specifies the center of the distribution; and the scale-parameter σ determines the size of deviations about the location parameter. The inverse function of the Gumbel distribution $G(x; \mu, \sigma)$ is given by the following formula:

$$a(T) = \mu - \sigma \ln\left[-\ln\left(1 - \frac{1}{T}\right)\right]$$

Table 5: Constant values for calculation of rainfall intensity

Station	μ	$\sigma(m)$	$\theta(h)$	η
Rwerere	40.41	5.7	0.24	0.95

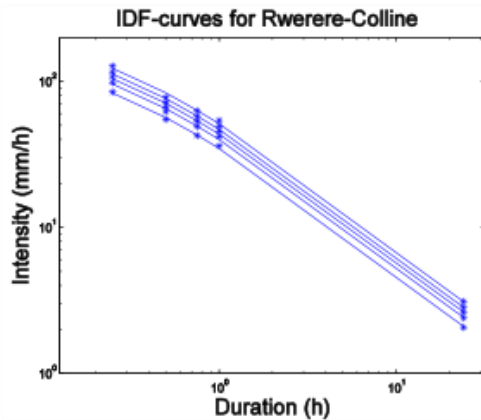


Figure 17: IDF Curves representation for RWERERE Station

Source: Demarée and Van de Vyver, 2013

E. . Runoff coefficients

Watershed runoff coefficients depend upon the land use, soil type and slope of the watershed. The runoff coefficient (rational coefficient) is difficult to determine precisely and can be interpreted in different ways. Engineering judgment is necessary in selecting the appropriate procedure. Selection of coefficients should be based on ultimate catchment development and weighted average (see formula below) need be computed where more than one land use is encountered in a catchment. The runoff coefficients are needed to calculate storm water runoff rate using the Rational Method.

$$C_W = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \dots + C_nA_n}{A}$$

Where; $A_1, A_2, A_3, \dots, A_n$ = areas of relatively uniform land use or surface character each comprising the total area (A);

$C_1, C_2, C_3, \dots, C_n$ = Corresponding runoff coefficients.

Table 6: Typical values of rational runoff coefficients for urban areas (RTDA. 2014)

Land use	Condition	Range of <i>C</i> values
Lawns	sandy soil, flat<2%	0.05-0.10
	sandy soil, steep>7%	0.15-0.20
	Heavy soil, flat<2%	0.13-0.17
	Heavy soil, steep>7%	0.25-0.35
Residential	single family areas	0.30-0.50
	Apartment dwelling areas	0.50-0.70
Industrial	Light areas	0.50-0.80
	Heavy areas	0.60-0.90
Business	Downtown areas	0.70-0.95
	Neighborhood areas	0.50-0.70
Streets	Asphaltic	0.70-0.95
	Concrete	0.80-0.95
	Bricks	0.70-0.85
Roofs		0.75-0.95

Table 7: Typical values of rational runoff coefficients for rural areas (RTDA. 2014)

Factor	Component	Mean Annual Precipitation (mm)		
		<300	300- 600	>600
<i>C_s</i>	3.5% flat	0.01	0.02	0.03
	3.5%-11% soft to moderate	0.04	0.06	0.08
	11%-35% steep	0.09	0.12	0.16
	>35% very steep	0.18	0.22	0.26
<i>C_k</i>	very permeable	0.02	0.03	0.04
	permeable	0.04	0.06	0.08
	semi-permeable	0.08	0.12	0.16
	impermeable	0.15	0.21	0.26
<i>C_v</i>	Dense forest or very loose deposits	0.02	0.03	0.04
	Cultivated land or thin forest	0.04	0.07	0.11
	Grassland	0.13	0.17	0.21
	bare rock	0.24	0.26	0.28

Note:

1. For contour cultivated lands: $C = 0.80 \times (C_s + C_k + C_v)$
2. 100% dense wood : flat<3.5% $C = 0.60 \times (C_s + C_k + C_v)$ steep>11% $C = 0.80 \times (C_s + C_k + C_v)$
3. For lakes, swamps and dams $C = 1.00 \times (C_s + C_k + C_v)$

Table 8: Calculated Peak discharge

Catchment	Area (km ²)	Time of concentration		Intensity (mm/hr)	Discharge (m ³ /s)			
		(min)	(hr)		10 yrs	20 yrs	25yrs	50 yrs
Catchment	18.465	63.819	1.064	41.434	53.56	57.70	59.02	63.07

For this research, a discharge of 50 years return period was considered as design flow rate.

From the topography, the surface area of the swampy area close to the built site was found to be 1.48 km² (length of 5.8 km with a width of 253 m). These dimensions were measured on the topographic map after delineating the catchment. Hence, the flood level above the ground surface was calculated for a time equal to the time of concentration are depicted into table 3:

Table 9: Determination of flood level

	10 years	20 years	25 years	50 years
Water volume (m³/s)	205079.7	220948.3	225982.1	241488.7
Flood level (m)	0.166	0.179	0.183	0.195

From the table 3, it was found that maximum flood level is 20 cm above the ground surface and this represents the maximum submerged level of the toe of the slope next to the retaining wall.

4.2.3.5 Design of drainage system

The drainage of groundwater may also be necessary for the following reasons:

- to increase the stability of the ground
- to avoid surface flooding
- to alleviate subsoil water pressures likely to cause dampness to below-ground accommodation
- to assist in preventing damage to foundations of buildings
- to prevent frost heave of subsoil that could cause fractures to structures such as concrete slabs.

The selection of an appropriate drainage layout will depend on the nature of the subsoil and the topography of the ground.

According to Toll *et al.* (2012), lowering of the groundwater table can cause the soil to consolidate, which induces settlement. With softer, more compressible soils, settlements can become large. Due to its high compressibility, any changes in stress resulting from groundwater level changes are likely to result in large surface settlement or heave. Dewatering is effective in improving stability by reducing pore pressures around excavation. Sometimes the dewatering

system may fail which bring the water level close to the foundation and reduce the capacity of the soil, which contribute to the settlements and displacements of the adjacent structures (Zumrawi and Hassan, 2016).

To avoid the rise of groundwater up to the foundation and soil liquefaction, it is necessary to construct a drainage system that will limit that rise and at the same time evacuate the water from the swamp.

The formula of Hooghoudt (1940), was used to estimate the discharge that can be evacuated on 1 m depth along the length of the retaining wall.

$$q = \frac{8K_2Dh+4K_1h^2}{L^2} \quad (\text{Hooghoudt, 1940})$$

where: q_t =drain discharge per unit surface area, K = hydraulic conductivity of the soil layer (m/s), d = equivalent depth of the soil layer below the drain level (m), L = drain spacing (m), h = height of water table (m), D = depth from the impervious layer up to the drain level (m).

The depth and spacing of drains are two closely linked parameters. As drain depth is increased, spacing between drains may also be increased. In homogeneous land, deep drainage has a certain number of advantages over superficial drainage since:

- the water table is lowered further, giving better soil aeration;
- drains may be more widely spaced, which results in a reduction in drain number and drainage cost;
- drains are in this way protected against root invasion which tends to clog them.

On the other hand, deep drains tend to cause a too rapid fall of the water table in the dry season.

For practical purposes, drain depth varies between 0.70 m and 1.50 m. A depth of 0.60-0.80 m is considered small; a depth of over 1.20 m is considered as deep drainage.

A large number of formulae have been developed for calculating drain spacing for the permanent regime. In irrigated zones and in regions of high intensity and short duration rainfall, drain water input is not constant and it is therefore necessary to use transitory regime formulae (Odong, 2007).

In the case of unsteady/transient flow, the flow is not constant, rather it changes with time as water is stored in or released from the soil. The change in storage is reflected either in a rise or

a fall of the water table. The Dupuit-Forchheimer approach is used to derive a differential equation of unsteady flow in the waterlogged soil (Odong, 2007). Calculating the spacing between drains is easier for the permanent regime than for the transitory regime; moreover, use of the permanent regime formulae is often justifiable even in the transitory regime, especially when precise knowledge about flood conditions and hydrological constants is not available.

4.2.3.6. Design of drainage system

The drain discharge was estimated by considering the case where the unconfined aquifer is saturated, and the drainage system has to lower the water table at least to the level of 1 m below the foundation of the retaining wall. Therefore, a fall of 1 m of water table was considered. That depth represents 1 m of layer below the foundation which will be free from the groundwater. Hence, one field drain was designed along the retaining wall.

$$q = \frac{8K_2Dh+4K_1h^2}{L^2} \quad (\text{Hooghoudt, 1940})$$

Considering the site conditions of this project, the drainage system was proposed to have two lateral drains of 0.10 m diameter that discharge into the collector proposed in downhill after the existing retaining wall. Therefore, the proposed main drain (collector) along the downhill side of the built-up area is designed for conveying the discharge from the groundwater of **40.4 l/s** and the runoff from the flooding of the marshland.

For the design of the collector (main field drain) the following parameters were considered for determination of the peak runoff; length of the collector (L) in m, width of the strip between the collector and the existing stream draining the marshland (l) in m, flood level above the ground surface close to the sloping side close to the retaining wall (h) in m, and the calculated time of concentration for the concerned catchment (T_c) in minutes. Hence, the volume (expressed in m³) of stagnated water was calculated as follows:

$$V = L \times l \times h$$

Or the discharge Q (m³/s) is the rate of volume.

$$Q = \frac{V}{T_c}$$

Table 10: Runoff to be drained

L (m)	160
l (m)	125
h (m)	0.2
V (m ³)	4000
Tc (min)	63.82
Q (m³/s)	1.042

The total discharge to be drainage was $1.0446 \text{ m}^3/\text{s} + 0.0404 \text{ m}^3/\text{s} = \mathbf{1.0824 \text{ m}^3/\text{s}}$.

4.2.3.7. Hydraulic design of the collector

Adequate storm water drainage system or adequate waterway is a system or a waterway, which has the hydraulic characteristics to accommodate the maximum expected flow (design flow) of storm water for a given watershed or a portion/s thereof.

The adequate system;

- 1) should be designed to account for both off-site and on-site storm water, including storm water coming into a given tract of land from upstream;
- 2) should discharge the flow into natural drainage line or other appropriate outlets; and
- 3) should carry water to a point where it should flow downstream into a stream channel or water way.

Design flood standards are influenced by many factors including:

- safety;
- The level of hydraulic performance required;
- Environmental impact;
- Construction and operation costs;
- Maintenance requirements;
- Serviceability; and,
- Legal and statutory requirements.

The standards are expressed by average recurrence intervals adopted. These measures determine the magnitude of a design rainfall or runoff event with which the system can cope.

Assigning an average recurrence interval takes into account the level of safety (standard) required. The required level of safety in turn depends upon potential losses to occur following the probable failure (both structural and serviceability) of an element of a drainage system or a combination thereof. Generally, the longer the average recurrence interval, the higher the magnitude of flow will be and the safer the level of protection.

The hydraulic capacity be verified by using the Manning's formula.

$$Q = K * A_w * R_h^{2/3} * S^{0.5}$$

Where Q: Discharge (m³/s),

K: roughness Coefficient

A_w: Wetted area, expressed in (m²),

P_w: Wetted perimeter (m)

R_h: hydraulic radius = A_w/P_w, expressed in (m),

S: Slope of the natural flow, expressed in (m/m).

The collector was designed for accommodating the runoff and the drained groundwater of **1.0824 m³/s**.

Table 11: Hydraulic characteristics of the designed channel

Hydraulic parameters	Values
Discharge Q(m ³ /s)	1.08
Strickler coefficient Ks	37.04
Slope S (m/m)	0.0300
Widht B (m)	1.00
Water depth (m)	0.44
Velocity (m/s)	2.44
Cross section (m ²)	0.44
Wetted Perimeter (m)	1.89
Freeboard (m)	0.150
Total height (m)	0.593

By considering the need of limiting the groundwater level under the foundation it is proposed to fill the trench with filter material (especially gravels) that will play also a role of supporting the uphill soil. Therefore, the channel will be of the following dimensions: height of 1.2 m and 1.2 m wide. To avoid the escape the fines a geotextile is required between soil and filter material.

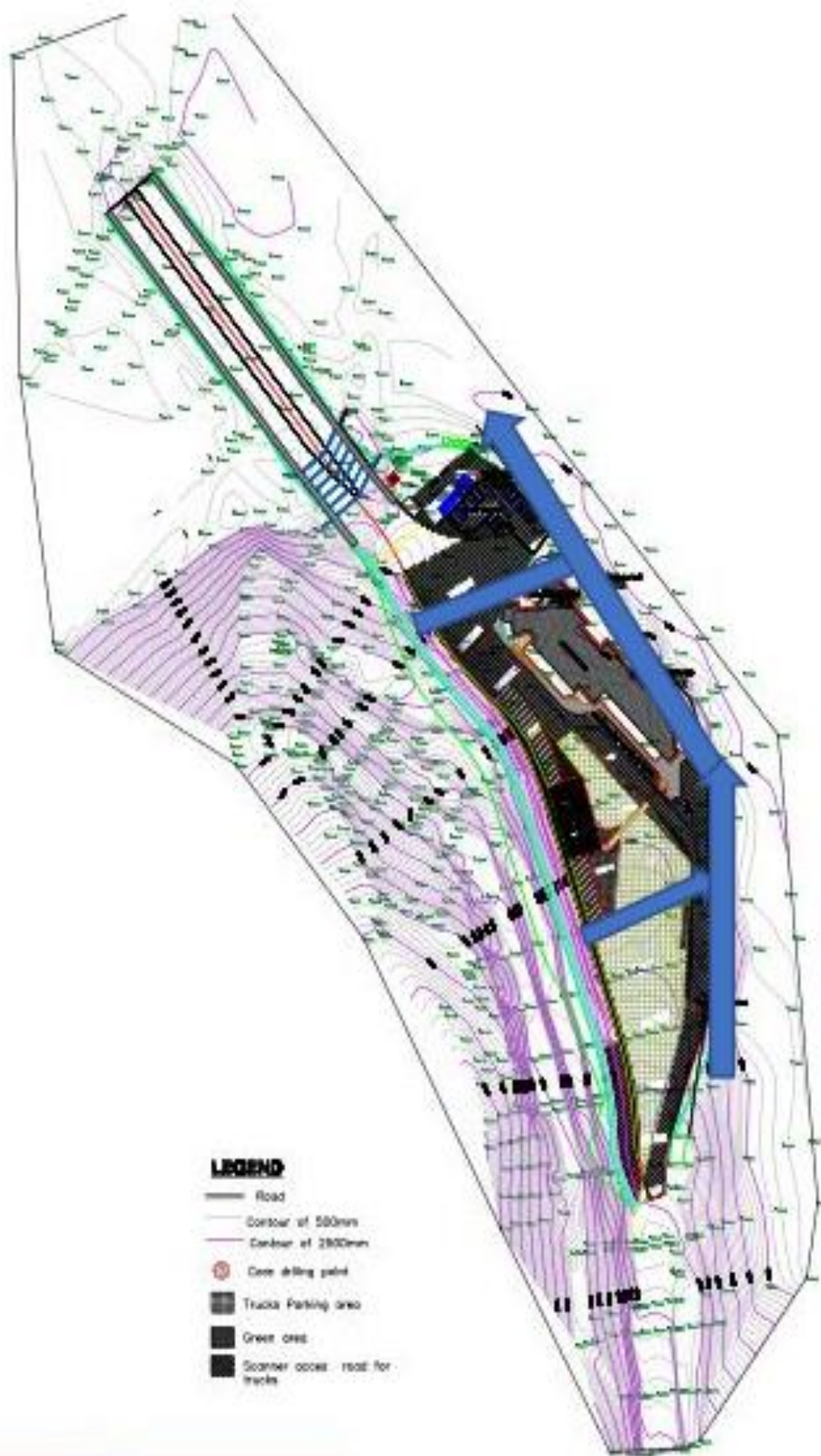


Figure 18: Layout of the proposed drainage system

The arrows in blue represent the proposed drainage system.

The proposed drain pipe is shown in figure 3, and it will be installed at 5.5 m for the lateral drains and at 1.2 m for the collector. The designed longitudinal slope is of 3%.

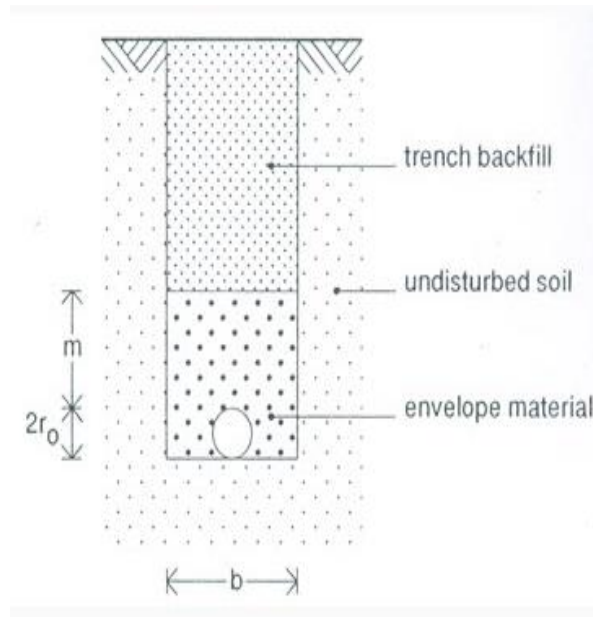


Figure 19: Sketch of the proposed pipe drain

Filters for drains are permeable materials placed around the drains for the purpose of preventing fine-grained materials in the surrounding soils from being carried into the drain by ground water. Envelopes for drains are permeable materials placed around the drains for the purposes of improving flow conditions in the area immediately surrounding the drain, and for improving bedding conditions.

4.2.3.8 Quantity estimation

The estimation of quantity concerned the following items:

Volume to be excavated (earthwork), the linear of the drain pipes, geotextile, filter material (gravel) and backfill (earthwork).

Table 12 : Quantity of work for the implementation of the drainage system

Item	Unit	Qty
Trench excavation for lateral pipe drain I (85 m*5.5m*1m)	m3	467.5
Trench excavation for lateral pipe drain I (100 m*5.5m*1m)	m3	550
Trench excavation for the collector (160 m*1.2 m*1.2 m)	m3	230.4
Pipe drain (10 cm diameter)	MI	100
Pipe drain (10 cm diameter)	MI	85
Pipe drain (30 cm diameter)	MI	160
Geotextile	m ²	452.4
Filter media (gravels)	m3	452.4
Backfill	m3	1174.116

4.2.4. Geotechnical

The objective of this section is to analyze existing geotechnical data carried on Gatuna one stop border post and foundation settlement analysis of both main building and the one of reinforced concrete retaining wall.

By providing adequate sustainable and cost-effective solutions in case the settlement and distortion exceed the allowable values as per **Euroced7 table 3.4.1**.

Analyzed data

Two geotechnical reports have been provided as the basic existing geotechnical data

1. Geotechnical Report

As per the table.13.the soil of swampy area varies, but the predominant type is clay and peat, for example the soil sample A,B,F,F-6 and F-7 characteristic of organic soil . This type of soil may undergo very high settlement when loaded, even small loaded structure should be constructed on this soil with adequate substructure, which should mitigate excessive settlement.

Table 13: Summary of results issued for swampy area

sample no.	Soil description	depth m	Natural density		Plasticity (Atterberg Limits)			Grading			Soil Classification		Cohesion kN/m2	angle of internal friction degree
			kg/dm3	water %	PL %	LL %	PI %	<5,0mm %	<2,0 mm %	<0,8mm %	ASTM	AASHTO		
B	peat	0-4,5	0,79	268			NP	98	94	86	OL			
A	peat with silt	4,5-5,5	0,85	228			NP	92	72	53	OL	A-4		
C	soft clay	5,5-6,5	1,42	56	25	41	16	99	99	94	CL	A-7-6	0,4	41
D	medium consistant clay	6,5-7,5	1,57	35	20	41	21	100	100	81	CL	A-7-6	0,5	32
E	sandy silt	7,5-8,5	1,53	50			NP	99	96	64	ML	A-4		
F	peat	8,5-9,5	0,87	537			NP	91	67	35	OL			
SITE no. 1 RESULTS OF TEST PITS F6 AND F7														
F-6	brown soft clay	4,2-6,2	0,98	36	26	42	16		100	96	CL	A-7-6		
F-6	brown medium consistant clay	6,2-8,0	1,09	19	24	48	24		100	97	CL	A-7-6		
F-7	soft clay	5,5-8,0	0,9	37	27	49	22		100	95	CL	A-7-6		

Table 14: Summary of laboratory test results site n°2

SITE no. 2 Material samples from 5 trial pits															
pit no.	Soil description	depth m	Compaction (Proctor)		CBR at 95%MDD %	Plasticity (Atterberg Limits)			Grading			Soil Classification		Cohesion kN/m2	angle of internal friction degree
			MDD kN/m3	OMC %		PL %	LL %	PI %	<5,0mm %	<2,0 mm %	<0,8mm %	ASTM	AASHTO		
1	altered rock	4,2	1,69	21,6	6	20	34	14	99	97	85	ML	A-6	3,2	38
2	red clay silt	7,0				29	44	15	100	100	76	CL	A-7-6	3,0	30
3	brown silt	7,0				NP	38	NP	100	100	81	ML	A-4	2,9	42
4	brown clay	8,0	1,65	22,7	8	33	48	15	95	84	49	CL	A-7-6	2,5	45
5	brown silt with laterite	9,1	1,89	15,9	12	NP	36	NP	87	82	71	ML	A-4	3,2	37

As mentioned in the report, this soil formation has been encountered on the upper side of the project location of the former border offices, their physical properties: liquid limits varies between 0 to 48% with average internal friction angle of 38° and cohesion of 5Kpa and plasticity index varies from 0 to 15% and mechanical properties reveals that, the medium to good bearing capacity is expected, the reason why our analysis has taken also this formation into consideration.

On the other hand for retaining wall analysis, we have only considered the recent geotechnical report in our analysis because aforementioned geotechnical investigation test station were nearby the location of retaining wall.

2. Geotechnical report

Table 15: Standard penetration test results conducted around existing retaining wall

BH No.	Depth [m]	El. [m]	N _{field}	N ₆₀
BH-01	0.0	1,800.743	5	5
BH-01	1.5	1,799.243	4	4
BH-01	3.0	1,797.743	0	0
BH-01	4.5	1,796.243	2	3
BH-01	6.0	1,794.743	0	0
BH-01	7.5	1,793.243	16	22
BH-01	9.0	1,791.743	41	50
BH-01	12.0	1,788.743	R	50
BH-02	0.0	1,801.228	15	16
BH-02	4.5	1,796.728	2	3
BH-02	6.0	1,795.228	0	0
BH-02	7.5	1,793.728	0	0
BH-02	9.0	1,792.228	5	7
BH-02	10.5	1,790.728	13	18
BH-02	12.0	1,789.228	11	16
BH-03	0.0	1,802.755	8	9
BH-03	1.5	1,801.255	5	5
BH-03	3.0	1,799.755	5	6
BH-03	4.5	1,798.255	2	3
BH-03	6.0	1,796.755	6	8
BH-03	7.5	1,795.255	4	5
BH-03	9.0	1,793.755	12	17
BH-03	10.5	1,792.255	16	23
BH-03	12.0	1,790.755	20	28
BH-04	0.0	1,807.031	3	3
BH-04	1.5	1,805.531	6	6
BH-04	6.0	1,801.031	6	8
BH-04	7.5	1,799.531	0	0
BH-04	9.0	1,798.031	2	3
BH-04	10.5	1,796.531	2	3
BH-04	12.0	1,795.031	8	11
BH-05	0.0	1,802.858	13	14
BH-05	1.5	1,801.358	3	3
BH-05	3.0	1,799.858	0	0
BH-05	4.5	1,798.358	3	4
BH-05	6.0	1,796.858	17	23
BH-05	7.5	1,795.358	R	50

From table15, we can see that from SPT test with zero values can be explained either by the presence of a cavity or a very soft organic soil. Swamp loose soil is unlikely to have cavities insofar we have to assume a layer with a bearing capacity near to zero.

Table 16: Summary of Mechanical properties of soil

Sample No.	Borehole No.	Depth (m)	Shear test (CD) c'/ϕ'		Consolidation test	
			Angle of internal friction (ϕ')	Cohesion (kPa)	Compressibility (m_v)	Consolidation coefficient (c_v)
SL 101	1	4.5-5.0	22.12	15.36	0.024	1.3230
SL 102	1	6.6-9.5	5.03	12.29	30.000	0.3714
SL 103	1	6.9-7.9	12.27	25.42	46.856	0.2442
SL 104	2	7.5-9.0	12.87	6.23	0.122	0.959
SL 105	2	10.5-12	21.72	17.18	0.119	0.341
SL 106	3	7.9-9.0	24.74	25.90	0.040	1.097
SL 107	3	9.5-10.5	15.16	3.78	7.056	0.736
SL 108	3	12.4-13	9.85	5.48	3.46	0.286
SL 109	4	7.5-9.0	26.08	15.16	0.010	1.596
SL 112	4	6.4-7.5	10.96	3.27	1.276	0.353
SL 110	5	3.4-4.5	0.2	13.79	0.059	0.648
SL 111	5	4.5-6.0	1.61	18.90	0.035	0.77

From table16 we can see that internal friction is varying between 0.2 to 26° which is very different from what we have obtained in the previous report, the coefficient of compressibility of 46 and 30 are not also reliable for settlement analysis; during our analysis if we take into consideration this parameter the soil will obviously collapse.

Based on Geotechnical data and soil pressure provided by structural engineer, settlement analysis has been carried out by using PLAXIS 3D, whereby two situations have been considered:

- By considering only static load
- By considering also load induced by vibrations from traffic

It is to be noted that two scenarios were also analyzed to see the expected settlements **Standard documents used as reference**

-**BS400 Part 2: Specifications for load**

-**Eurocode 7: Geotechnical design**

- **Braja M.Das (2006). Principle of geotechnical engineering 7th edition.**

Presentation of results

A. Subsoil of the Building Structure

Design parameters

Concrete foundation design parameters are as follow:

Thickness: 0,4m

Unit weight: 25KN/m³

Elastic modulus E: 30*10⁶

Poissson ratio: 0.2

Based on the geotechnical data: Soil profile description, physical properties of soil and mechanical properties of soil the following parameters as design parameters of the possible worse scenario

Table 17: Soil profile description

Layer	Unsaturated density(KN/m ³)	Saturated density (KN/m ³)	Cohesion(Kpa)	Internal Friction angle(ϕ)	Elastic modulus (Kpa)	Poissson ratio
1 (0-3m)	17	19	14	26	100000	0.2
Layer 2	18	20	18	33	105000	0.2
Layer 3	17.5	19.5	12	29	95000	0.24
Layer 4	18	19	3	15	80000	0.2
Layer 5	18	20	12	29	10000	0.25

With water table level kept at 3m below the foundation level for main building and at 1m for retail wall

Soil design parameters for favorable scenario: for this case we have considered that the settlement within soil layer of two times the width of foundation, the maximum layer thickness obtained is 3.6m

Coefficient of volume compressibility of $M_v=0.2\text{m}^2/\text{MN}$, the coefficient of secondary consolidation used is 0.007.

Total expected settlement after 30 years was calculated based on the relationship below:

$$S_t = S_e + S_c + S_s \quad (1)$$

S_t = Total settlement

S_e = Elastic settlement = $0.1S_c$

S_c = Primary consolidation settlement

S_s = Secondary consolidation settlement

$$S_c = m_v * q * H \quad (2)$$

a. Results

Settlement analysis has been done by using PLAXIS 3D to calculate total settlement

And results are summarized in the following tables

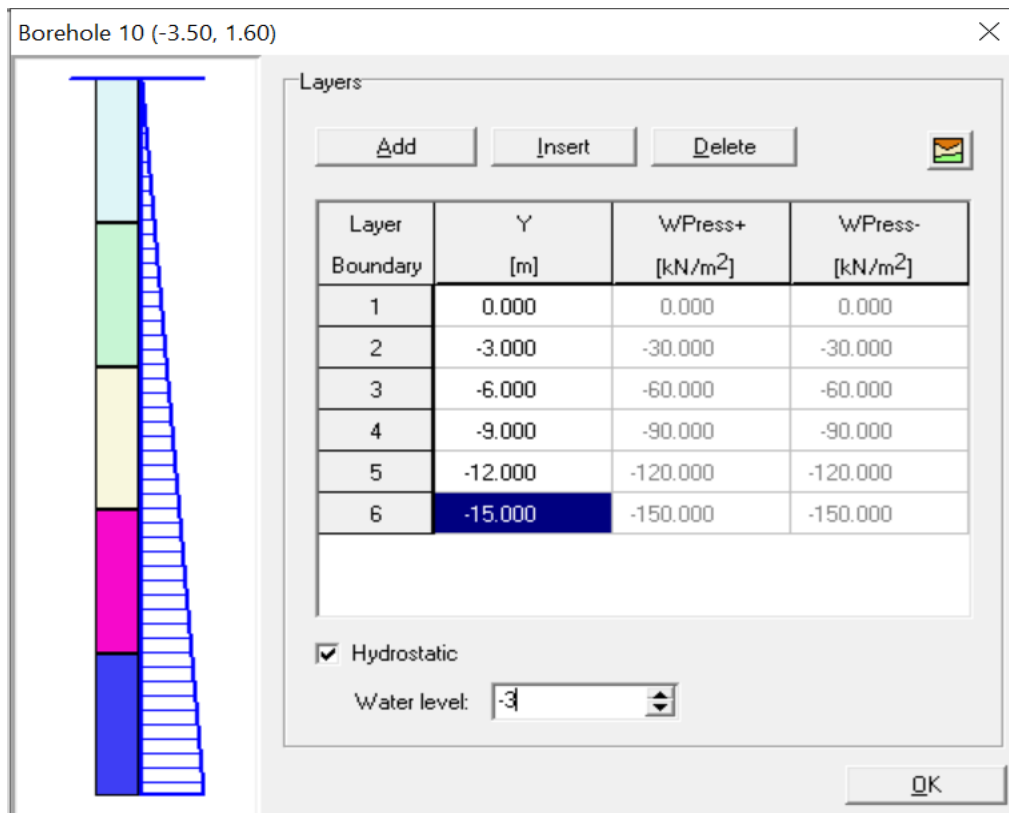


Figure 20: Settlement analysis

As per the figure above layers under foundation are considered for the model

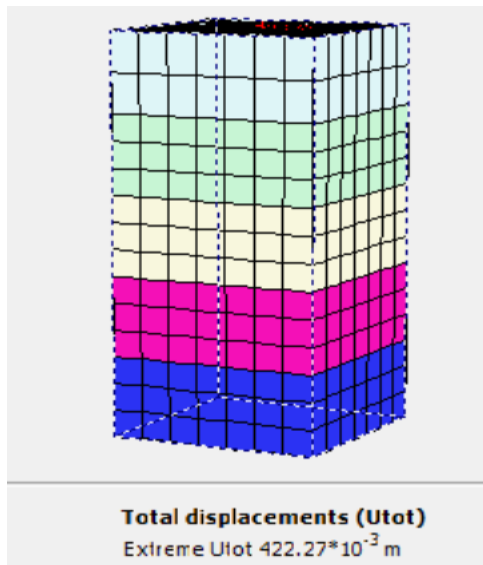
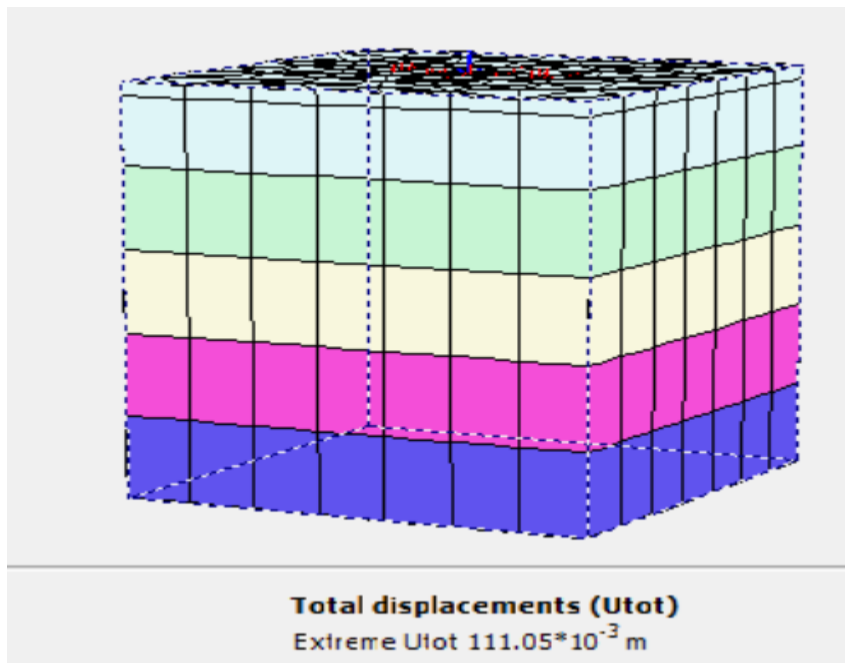


Figure 21: Total displacement

Typical higher values were obtained for some pad foundation under combination of static and dynamic loads.



For the retaining wall the critical section of 4m high was analyzed and the total settlement calculated is 111mm, which is greater than 75mm but less than 135mm.

Table 18: Results analysis by considering only static load

S/No	Foundation	Location	Foundation			Total load on foundation	Foundation area	Soil pressure (KN/m ²)	Soil bearing capacity (KN/m ²)	Settlement (mm)
			L	W	T					
1	B1	F-2	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
2	B1	E-2	1.8	1.8	0.4	288.306	3.24	88.983333 33	150	27.1
3	B1	ED-2	1.8	1.8	0.4	177.448 5	3.24	54.768055 56	150	14.8
4	B1	F-3	1.8	1.8	0.4	395.586	3.24	122.09444 44	150	38.9
5	B1	E-3	1.8	1.8	0.4	438.516	3.24	135.34444 44	150	43.6
6	B1	ED-3	1.8	1.8	0.4	289.273 5	3.24	89.281944 44	150	27.2
7	B1	D-3	1.8	1.8	0.4	268.146	3.24	82.761111 11	150	24.8
8	B3	CD-3	1.8	3.5 7	0.4	434.784 6	6.426	67.660224 09	150	23.9
9	B1	F-4	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
10	B4	E-4	1.8	1.8	0.4	406.611	3.24	125.49722 22	150	40.1
11	B4	ED-4	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
12	B4	D-4	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
13	B4	CD-4	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4

14	B4	CB-4	1.8	1.8	0.4	351.182 25	3.24	108.38958 33	150	34.0
15	B5	A-4	1.2	1.2	0.4	172.901 25	1.44	120.07031 25	150	28.7
16	B1	4,5-A	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
17	B1	E-5	1.8	1.8	0.4	269.113 5	3.24	83.059722 22	150	24.9
18	B1	ED-5	1.8	1.8	0.4	353.331	3.24	109.05277 78	150	34.2
19	B1	D-5	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
20	B1	CD-5	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
21	B1	CB-5	1.8	1.8	0.4	390.996	3.24	120.67777 78	150	38.4
22	B1	AB-5	1.8	1.8	0.4	355.603 5	3.24	109.75416 67	150	34.5
23	B5	A-5	1.2	1.2	0.4	149.985	1.44	104.15625	150	24.5
24	B1	5,6-E	1.8	1.8	0.4	248.953 5	3.24	76.8375	150	22.7
25	B1	ED-5,6	1.8	1.8	0.4	242.473 5	3.24	74.8375	150	22.0
26	B1	ED-6	1.8	1.8	0.4	246.793 5	3.24	76.170833 33	150	22.5
27	B1	D-6	1.8	1.8	0.4	378.036	3.24	116.67777 78	150	37.0
28	B1	CD-6	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
29	B1	CB-6	1.8	1.8	0.4	390.996	3.24	120.67777 78	150	38.4
30	B1	AB-6	1.8	1.8	0.4	355.603 5	3.24	109.75416 67	150	34.5

31	B1	ED-7	1.8	1.8	0.4	248.953 5	3.24	76.8375	150	22.7
32	B1	D-7	1.8	1.8	0.4	252.531	3.24	77.941666 67	150	23.1
33	B1	CD-7	1.8	1.8	0.4	388.851	3.24	120.01574 07	150	38.1
34	B1	CB-7	1.8	1.8	0.4	390.996	3.24	120.67777 78	150	38.4
35	B1	AB-7	1.8	1.8	0.4	345.523 5	3.24	106.64305 56	150	33.4
36	B1	7,8- EF	1.8	1.8	0.4	263.601	3.24	81.358333 33	150	24.3
37	B1	ED-7,8	1.8	1.8	0.4	316.221	3.24	97.599074 07	150	30.1
38	B1	D-8	1.8	1.8	0.4	313.011	3.24	96.608333 33	150	29.8
39	B1	CD-8	1.8	1.8	0.4	345.039 75	3.24	106.49375	150	33.3
40	B1	CB-8	1.8	1.8	0.4	386.451	3.24	119.275	150	37.9
41	B1	B-8	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
42	B1	E,F-8,9	1.8	1.8	0.4	263.601	3.24	81.358333 33	150	24.3
43	B1	C-9	1.8	1.8	0.4	377.068 5	3.24	116.37916 67	150	36.8
44	B1	B-9	1.8	1.8	0.4	313.011	3.24	96.608333 33	150	29.8
45	B1	ED-9,10	1.8	1.8	0.4	353.331	3.24	109.05277 78	150	34.2
46	B1	CD-9,10	1.8	1.8	0.4	353.331	3.24	109.05277 78	150	34.2

47	B1	CB-9,10	1.8	1.8	0.4	288.306	3.24	88.983333 33	150	27.1
48	B1	B-9,10	1.8	1.8	0.4	177.748 0125	3.24	54.860497 69	150	14.9
49	B1	E-10	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
50	B1	D-10	1.8	1.8	0.4	353.331	3.24	109.05277 78	150	34.2
51	B1	C-10	1.8	1.8	0.4	397.228 5	3.24	122.60138 89	150	39.1
52	B1	CB-10	1.8	1.8	0.4	248.953 5	3.24	76.8375	150	22.7
53	B4	F-11	1.8	1.8	0.4	283.761	3.24	87.580555 56	150	26.6
54	B4	E-11	1.8	1.8	0.4	418.356	3.24	129.12222 22	150	41.4
55	B4	D-11	1.8	1.8	0.4	406.611	3.24	125.49722 22	150	40.1
56	B4	C-11	1.8	1.8	0.4	397.228 5	3.24	122.60138 89	150	39.1
57	B4	CB-11	1.8	1.8	0.4	294.786	3.24	90.983333 33	150	27.8
58	B1	ED-11,1 2	1.8	1.8	0.4	406.611	3.24	125.49722 22	150	40.1
59	B1	F-12	1.8	1.8	0.4	309.433 5	3.24	95.504166 67	150	29.4
60	B1	E-12	1.8	1.8	0.4	373.491	3.24	115.275	150	36.5
61	B1	C-12	1.8	1.8	0.4	397.228 5	3.24	122.60138 89	150	39.1
62	B1	CB-12	1.8	1.8	0.4	269.113 5	3.24	83.059722 22	150	24.9

63	B2	F-13	1.8	3.5 7	0.4	372.193 875	6.426	57.919993	150	19.7
64	B2	E-13	1.8	1.8	0.4	298.105 9125	3.24	92.007997 69	150	28.1
65	B1	ED- 13	1.8	1.8	0.4	339.021	3.24	104.63611 11	150	32.6
66	B2	CD- 13	1.8	3.5 7	0.4	365.879 5125	6.426	56.937365 78	150	19.2
67	B2	C-13	1.8	3.5 7	0.4	406.199 5125	6.426	63.211875 58	150	22.0
68	B2	CB- 13	1.8	3.5 7	0.4	363.928 275	6.426	56.633718 49	150	19.1
69	B6	E- 13,1 4	1.8	1.8	0.4	181.195 65	3.24	55.924583 33	150	15.2
70	B6	ED- 13.1 4	1.8	1.8	0.4	214.737 075	3.24	66.276875	150	18.9
71	B6	CD- 13,1 4	1.8	1.8	0.4	197.966 3625	3.24	61.100729 17	150	17.1
72	B6	C- 13,1 4	1.8	1.8	0.4	197.966 3625	3.24	61.100729 17	150	17.1
73	B1	F-14	1.8	1.8	0.4	196.239 825	3.24	60.567847 22	150	16.9
74	B1	E-14	1.8	1.8	0.4	204.098 625	3.24	62.993402 78	150	17.8
75	B1	ED- 14	1.8	1.8	0.4	210.606 5625	3.24	65.002025 46	150	18.5
76	B1	CD- 14	1.8	1.8	0.4	254.232 5088	3.24	78.466823 69	150	23.3

77	B1	C-14	1.8	1.8	0.4	369.218 4	3.24	113.95629 63	150	36.0
78	B1	ED- 14,1 5	1.8	1.8	0.4	262.775 25	3.24	81.103472 22	150	24.2
79	B1	ED- 15,1 6	1.8	1.8	0.4	167.599 125	3.24	51.728125	150	13.7
80	B1	CD- 15	1.8	1.8	0.4	295.511 625	3.24	91.207291 67	150	27.8
81	B1	C-15	1.8	1.8	0.4	326.111 625	3.24	100.65173 61	150	31.2
82	B1	CD- 16	1.8	1.8	0.4	234.791 625	3.24	72.466550 93	150	21.1
83	B1	C-16	1.8	1.8	0.4	265.031 625	3.24	81.799884 26	150	24.5
84	B1	ED- 2,3	1.8	1.8	0.4	243.441	3.24	75.136111 11	150	22.1
85	B1	D- 2,3	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
86	B1	CD- 2,3	1.8	1.8	0.4	177.448 5	3.24	54.768055 56	150	14.8
87	B1	AB- 1	1.8	1.8	0.4	288.306	3.24	88.983333 33	150	27.1
88	B1	CB- 1,2	1.8	1.8	0.4	268.146	3.24	82.761111 11	150	24.8
89	B1	CB- 2,3	1.8	1.8	0.4	314.946	3.24	97.205555 56	150	30.0
90	B1	AB- 1,2	1.8	1.8	0.4	271.182 2625	3.24	83.698229 17	150	25.2
91	B1	AB- 2	1.8	1.8	0.4	274.765 725	3.24	84.804236 11	150	25.6

92	B1	AB- 2,3	1.8	1.8	0.4	314.610 525	3.24	97.102013 89	150	30.0
93	B1	CB- 2,3	1.8	1.8	0.4	300.245 445	3.24	92.668347 22	150	28.4
94	B1	A- 2,3	1.8	1.8	0.4	254.818 125	3.24	78.647569 44	150	23.4
95	B1	AB- 2,3	1.8	1.8	0.4	307.289 4188	3.24	94.842413 19	150	29.1
96	B1	B- 3,4	1.8	1.8	0.4	268.593 0518	3.24	82.899090 05	150	24.9
97	B1	CB- 3,4	1.8	1.8	0.4	227.585 3625	3.24	70.242395 83	150	20.4
98	B1	CB- 3,4	1.8	1.8	0.4	252.533 6438	3.24	77.942482 64	150	23.1
99	B5	5,6- A	1.8	1.8	0.4	242.473 5	3.24	74.8375 74.8375	150	22.0
100	B5	6,7- A	1.8	1.8	0.4	242.473 5	3.24	74.8375 74.8375	150	22.0
101	B5	A-7	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
102	B5	7,8- A	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
103	B5	8-A	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
104	B1	4,5- AB	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
105	B1	5,6- AB	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
106	B1	6,7- AB	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
107	B1	7,8- AB	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9

108	B5	AB-8	1.8	1.8	0.4	223.281	3.24	68.913888 89	150	19.9
109	B1	8,9-ED	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
110	B1	8,9-D	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
111	B1	8,9-CD	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
112	B1	E-9	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
113	B1	CB-14	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
114	B1	ED-2,3	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
115	B1	2,3-D	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1
116	B1	2,3-CD	1.8	1.8	0.4	261.666	3.24	80.761111 11	150	24.1

Table 19: Results of Analysis by Considering the Combination of Permanent and Dynamic/traffic Loads

S/No	Founda tion	Locat ion	Foun datio n		Total load on founda tion	Founda tion area	Soil press ure (KN/ m2)	Soil bearing capacity (KN/m2)		
			L	W	T					Settle ment(mm)
1	B1	F-2	1.8	1.8	0.4	223.28	3.24	88.9138888	150	40.5
2	B1	E-2	1.8	1.8	0.4	288.30	3.24	108.983333	150	51.3
3	B1	ED-2	1.8	1.8	0.4	177.44	3.24	74.7680555	150	33.0
4	B1	F-3	1.8	1.8	0.4	395.58	3.24	142.094444	150	69.1
5	B1	E-3	1.8	1.8	0.4	438.51	3.24	155.344444	150	76.2
6	B1	ED-3	1.8	1.8	0.4	289.27	3.24	109.281944	150	51.5
7	B1	D-3	1.8	1.8	0.4	268.14	3.24	102.761111	150	48.0
8	B3	CD-3	1.8	3.57	0.4	434.78	6.426	87.6602240	150	49.1
9	B1	F-4	1.8	1.8	0.4	223.28	3.24	88.9138888	150	40.5
10	B4	E-4	1.8	1.8	0.4	406.61	3.24	145.497222	150	70.9
11	B4	ED-4	1.8	1.8	0.4	418.35	3.24	149.122222	150	72.8

12	B4	D-4	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
13	B4	CD-4	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
14	B4	CB-4	1.8	1.8	0.4	351.18 225	3.24	128.389583 3	150	61.7
15	B5	A-4	1.2	1.2	0.3	172.90 125	1.44	140.070312 5	150	51.2
16	B1	4,5-A	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
17	B1	E-5	1.8	1.8	0.4	269.11 35	3.24	103.059722 2	150	48.1
18	B1	ED-5	1.8	1.8	0.4	353.33 1	3.24	129.052777 8	150	62.1
19	B1	D-5	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
20	B1	CD-5	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
21	B1	CB-5	1.8	1.8	0.4	390.99 6	3.24	140.677777 8	150	68.3
22	B1	AB-5	1.8	1.8	0.4	355.60 35	3.24	129.754166 7	150	62.4
23	B5	A-5	1.2	1.2	0.3	149.98 5	1.44	124.15625 150	150	44.8
24	B1	5,6-E	1.8	1.8	0.4	248.95 35	3.24	96.8375 150	150	44.8
25	B1	ED-5,6	1.8	1.8	0.4	242.47 35	3.24	94.8375 150	150	43.7
26	B1	ED-6	1.8	1.8	0.4	246.79 35	3.24	96.1708333 3	150	44.4
27	B1	D-6	1.8	1.8	0.4	378.03 6	3.24	136.677777 8	150	66.2

28	B1	CD-6	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
29	B1	CB-6	1.8	1.8	0.4	390.99 6	3.24	140.677777 8	150	68.3
30	B1	AB-6	1.8	1.8	0.4	355.60 35	3.24	129.754166 7	150	62.4
31	B1	ED-7	1.8	1.8	0.4	248.95 35	3.24	96.8375	150	44.8
32	B1	D-7	1.8	1.8	0.4	252.53 1	3.24	97.9416666 7	150	45.4
33	B1	CD-7	1.8	1.8	0.4	388.85 1	3.24	140.015740 7	150	67.9
34	B1	CB-7	1.8	1.8	0.4	390.99 6	3.24	140.677777 8	150	68.3
35	B1	AB-7	1.8	1.8	0.4	345.52 35	3.24	126.643055 6	150	60.8
36	B1	7,8- EF	1.8	1.8	0.4	263.60 1	3.24	101.358333 3	150	47.2
37	B1	ED- 7,8	1.8	1.8	0.4	316.22 1	3.24	117.599074 1	150	55.9
38	B1	D-8	1.8	1.8	0.4	313.01 1	3.24	116.608333 3	150	55.4
39	B1	CD-8	1.8	1.8	0.4	345.03 975	3.24	126.49375	150	60.7
40	B1	CB-8	1.8	1.8	0.4	386.45 1	3.24	139.275	150	67.5
41	B1	B-8	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
42	B1	E,F- 8,9	1.8	1.8	0.4	263.60 1	3.24	101.358333 3	150	47.2
43	B1	C-9	1.8	1.8	0.4	377.06 85	3.24	136.379166 7	150	66.0



44	B1	B-9	1.8	1.8	0.4	313.01 1	3.24	116.608333 3	150	55.4
45	B1	ED- 9,10	1.8	1.8	0.4	353.33 1	3.24	129.052777 8	150	62.1
46	B1	CD- 9,10	1.8	1.8	0.4	353.33 1	3.24	129.052777 8	150	62.1
47	B1	CB- 9,10	1.8	1.8	0.4	288.30 6	3.24	108.983333 3	150	51.3
48	B1	B- 9,10	1.8	1.8	0.4	177.74 80125	3.24	74.8604976 9	150	33.0
49	B1	E-10	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
50	B1	D-10	1.8	1.8	0.4	353.33 1	3.24	129.052777 8	150	62.1
51	B1	C-10	1.8	1.8	0.4	397.22 85	3.24	142.601388 9	150	69.3
52	B1	CB- 10	1.8	1.8	0.4	248.95 35	3.24	96.8375 150	150	44.8
53	B4	F-11	1.8	1.8	0.4	283.76 1	3.24	107.580555 6	150	50.6
54	B4	E-11	1.8	1.8	0.4	418.35 6	3.24	149.122222 2	150	72.8
55	B4	D-11	1.8	1.8	0.4	406.61 1	3.24	145.497222 2	150	70.9
56	B4	C-11	1.8	1.8	0.4	397.22 85	3.24	142.601388 9	150	69.3
57	B4	CB- 11	1.8	1.8	0.4	294.78 6	3.24	110.983333 3	150	52.4
58	B1	ED- 11,12	1.8	1.8	0.4	406.61 1	3.24	145.497222 2	150	70.9
59	B1	F-12	1.8	1.8	0.4	309.43 35	3.24	115.504166 7	150	54.8

60	B1	E-12	1.8	1.8	0.4	373.49 1	3.24	135.275	150	65.4
61	B1	C-12	1.8	1.8	0.4	397.22 85	3.24	142.601388 9	150	69.3
62	B1	CB- 12	1.8	1.8	0.4	269.11 35	3.24	103.059722 2	150	48.1
63	B2	F-13	1.8	3.57	0.4	372.19 3875	6.426	77.919993	150	42.7
64	B2	E-13	1.8	1.8	0.4	298.10 59125	3.24	112.007997 7	150	52.9
65	B1	ED- 13	1.8	1.8	0.4	339.02 1	3.24	124.636111 1	150	59.7
66	B2	CD- 13	1.8	3.57	0.4	365.87 95125	6.426	76.9373657 8	150	42.0
67	B2	C-13	1.8	3.57	0.4	406.19 95125	6.426	83.2118755 8	150	46.2
68	B2	CB- 13	1.8	3.57	0.4	363.92 8275	6.426	76.6337184 9	150	41.8
69	B6	E- 13,14	1.8	1.8	0.4	181.19 565	3.24	75.9245833 3	150	33.6
70	B6	ED- 13,14	1.8	1.8	0.4	214.73 7075	3.24	86.276875 150	39.1	
71	B6	CD- 13,14	1.8	1.8	0.4	197.96 63625	3.24	81.1007291 7	150	36.4
72	B6	C- 13,14	1.8	1.8	0.4	197.96 63625	3.24	81.1007291 7	150	36.4
73	B1	F-14	1.8	1.8	0.4	196.23 9825	3.24	80.5678472 2	150	36.1
74	B1	E-14	1.8	1.8	0.4	204.09 8625	3.24	82.9934027 8	150	37.4
75	B1	ED- 14	1.8	1.8	0.4	210.60 65625	3.24	85.0020254 6	150	38.4

76	B1	CD-14	1.8	1.8	0.4	254.23 25088	3.24	98.4668236 9	150	45.7
77	B1	C-14	1.8	1.8	0.4	369.21 84	3.24	133.956296 3	150	64.7
78	B1	ED-14,15	1.8	1.8	0.4	262.77 525	3.24	101.103472 2	150	47.1
79	B1	ED-15,16	1.8	1.8	0.4	167.59 9125	3.24	71.728125	150	31.3
80	B1	CD-15	1.8	1.8	0.4	295.51 1625	3.24	111.207291 7	150	52.5
81	B1	C-15	1.8	1.8	0.4	326.11 1625	3.24	120.651736 1	150	57.6
82	B1	CD-16	1.8	1.8	0.4	234.79 1625	3.24	92.4665509 3	150	42.4
83	B1	C-16	1.8	1.8	0.4	265.03 1625	3.24	101.799884 3	150	47.5
84	B1	ED-2,3	1.8	1.8	0.4	243.44 1	3.24	95.1361111 1	150	43.9
85	B1	D-2,3	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
86	B1	CD-2,3	1.8	1.8	0.4	177.44 85	3.24	74.7680555 6	150	33.0
87	B1	AB-1	1.8	1.8	0.4	288.30 6	3.24	108.983333 3	150	51.3
88	B1	CB-1,2	1.8	1.8	0.4	268.14 6	3.24	102.761111 1	150	48.0
89	B1	CB-2,3	1.8	1.8	0.4	314.94 6	3.24	117.205555 6	150	55.7
90	B1	AB-1,2	1.8	1.8	0.4	271.18 22625	3.24	103.698229 2	150	48.5
91	B1	AB-2	1.8	1.8	0.4	274.76 5725	3.24	104.804236 1	150	49.1

92	B1	AB- 2,3	1.8	1.8	0.4	314.61 0525	3.24	117.102013 9	150	55.7
93	B1	CB- 2,3	1.8	1.8	0.4	300.24 5445	3.24	112.668347 2	150	53.3
94	B1	A-2,3	1.8	1.8	0.4	254.81 8125	3.24	98.6475694 4	150	45.8
95	B1	AB- 2,3	1.8	1.8	0.4	307.28 94188	3.24	114.842413 2	150	54.4
96	B1	B-3,4	1.8	1.8	0.4	268.59 30518	3.24	102.89909 150	150	48.0
97	B1	CB- 3,4	1.8	1.8	0.4	227.58 53625	3.24	90.2423958 3	150	41.3
98	B1	CB- 3,4	1.8	1.8	0.4	252.53 36438	3.24	97.9424826 4	150	45.4
99	B5	5,6-A	1.8	1.8	0.4	242.47 35	3.24	94.8375 150	150	43.7
100	B5	6,7-A	1.8	1.8	0.4	242.47 35	3.24	94.8375 150	150	43.7
101	B5	A-7	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
102	B5	7,8-A	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
103	B5	8-A	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
104	B1	4,5- AB	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
105	B1	5,6- AB	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
106	B1	6,7- AB	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5
107	B1	7,8- AB	1.8	1.8	0.4	223.28 1	3.24	88.9138888 9	150	40.5

108	B5	AB-8	1.8	1.8	0.4	223.28	1	3.24	88.9138888	9	150	40.5
109	B1	8,9- ED	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
110	B1	8,9-D	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
111	B1	8,9- CD	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
112	B1	E-9	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
113	B1	CB- 14	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
114	B1	ED- 2,3	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
115	B1	2,3-D	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9
116	B1	2,3- CD	1.8	1.8	0.4	261.66	6	3.24	100.761111	1	150	46.9

 Settlement within permissible range
 Settlement out of permissible range

None of the settlements exceeding 80mm;

20% of footings will undergo the total settlement greater than 65mm;

From the results we can deduce that all applied soil pressure are reduced to maximum stress of 100Kpa all settlement will be less than 50cm within a period 10years.

Table 20: Results of Angular distortion

Footings	Distance center to center	Difference in settlement	Angular distortion	Max Allowable distortion	Max distortion to affect serviceability limit	Areas that affected by serviceability
f2-f3	6000	28.6	0.004766667	0.006	0.0033	
f3-f4	6000	28.6	0.004766666	0.006	0.0033	
f11-f12	6000	4.2	0.0007	0.006	0.0033	
f12-f13	3885	12.1	0.003114543	0.006	0.0033	
f13-f14	4230	0	0	0.006	0.0033	
f2-E2	3000	10.8	0.0036	0.006	0.0033	
E2-E3	6000	24.9	0.00415	0.006	0.0033	
E3-E4	6000	5.3	0.000883333	0.006	0.0033	
E4-E5	6000	22.8	0.0038	0.006	0.0033	
E5-E5,6	3000	3.3	0.0011	0.006	0.0033	
E7,8-E8,9	6000	0	0	0.006	0.0033	
E9-E10	6000	25.9	0.004316666	0.006	0.0033	
E10-E11	6000	0	0	0.006	0.0033	
F11-E11	3000	22.2	0.0074	0.006	0.0033	
F13-E12,13	3000	10.2	0.0034	0.006	0.0033	
E14--F14	3000	1.3	0.000433333	0.006	0.0033	
E2-ED2	3000	18.3	0.0061	0.006	0.0033	
ED2-ED2,3	3000	10.9	0.003633333	0.006	0.0033	

ED2,3- ED3	3000	7.6	0.00253333 3	0.006	0.0033	
ED3-ED4	6000	21.3	0.00355	0.006	0.0033	
ED4-E4	3000	1.9	0.00063333 3	0.006	0.0033	
ED5,6- E5,6	3000	1.1	0.00036666 7	0.006	0.0033	
ED5,6- ED6	3000	0.7	0.00023333 3	0.006	0.0033	
ED6-ED7	6000	0.4	6.66667E- 05	0.006	0.0033	
ED7- ED7,8	3000	11.1	0.0037	0.006	0.0033	
ED7,8- E7,8	4000	8.7	0.002175	0.006	0.0033	
ED7,8- ED8,9	6000	9	0.0015	0.006	0.0033	
ED8,9- E8,9	4000	0.3	0.000075	0.006	0.0033	
ED14-E14	3000	1	0.00033333 3	0.006	0.0033	
ED14- ED14,15	3250	8.7	0.00267692 3	0.006	0.0033	
D3-ED3	3000	3.5	0.00116666 7	0.006	0.0033	
D3-D4	6000	24.8	0.00413333 3	0.006	0.0033	
D4-ED4	3000	0	0	0.006	0.0033	
D10-D11	6000	8.8	0.00146666 7	0.006	0.0033	
D11-E11	6000	1.9	0.00031666 7	0.006	0.0033	

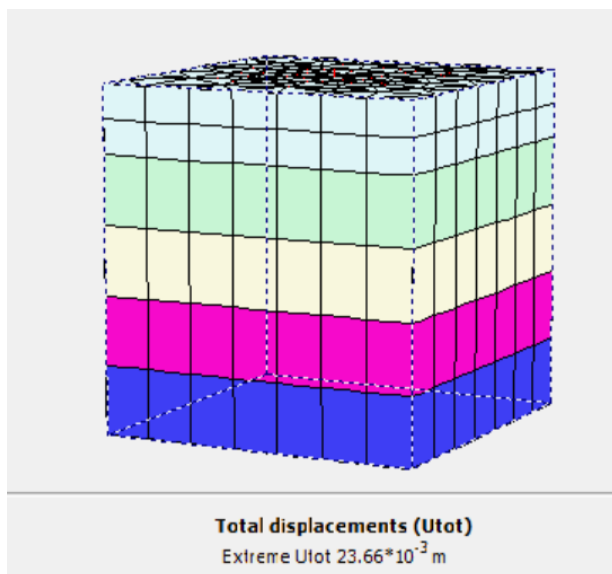
CD3-D3	3000	1.1	0.00036666 7	0.006	0.0033	
CD3-CD4	6000	23.7	0.00395	0.006	0.0033	
CD4-D4	3000	0	0	0.006	0.0033	
CD4-CD5	6000	0	0	0.006	0.0033	
CD5-CD6	6000	0	0	0.006	0.0033	
CD6-CD7	6000	4.9	0.00081666 7	0.006	0.0033	
CD7-CD8	6000	7.2	0.0012	0.006	0.0033	
CD8-D8	3000	5.3	0.00176666 7	0.006	0.0033	
CD8- CD8,9	3000	13.8	0.0046	0.006	0.0033	
CD8,9- D8,9	3000	0	0	0.006	0.0033	
C9-C10	6000	3.3	0.00055	0.006	0.0033	
CD9,10- ED9,10	6000	0	0	0.006	0.0033	
C10-C11	6000	0	0	0.006	0.0033	
C11-C12	6000	0	0	0.006	0.0033	
C12-C13	3885	23.1	0.00594594 6	0.006	0.0033	
C13-CD13	4500	4.2	0.00093333 3	0.006	0.0033	
C13- C13,14	2115	9.8	0.00463357	0.006	0.0033	
C14-ED14	4500	26.3	0.00584444 4	0.006	0.0033	
C14-C15	6000	19.2	0.0032	0.006	0.0033	
C15-CD15	4500	5.1	0.00113333 3	0.006	0.0033	
C15-C16	6000	10.1	0.00168333 3	0.006	0.0033	

C16-CD16	4500	5.1	0.00113333 3	0.006	0.0033	
CB4-CD4	4500	11.1	0.00246666 7	0.006	0.0033	
CB4-CB5	6000	11.1	0.00185	0.006	0.0033	
CB5-CB6	6000	6.6	0.0011	0.006	0.0033	
CB6-CD6	4500	4.5	0.001	0.006	0.0033	
CB6-CB7	6000	0	0	0.006	0.0033	
CB7-CB8	6000	0.8	0.00013333 3	0.006	0.0033	
CB8-CD8	4500	6.8	0.00151111 1	0.006	0.0033	
CB9,10- CD9,10	4465	10.8	0.00241881 3	0.006	0.0033	
CB11-C11	2965	16.9	0.00569983 1	0.006	0.0033	
B8-CB8	4500	5.3	0.00117777 8	0.006	0.0033	
B8-B9	6000	17.4	0.0029	0.006	0.0033	
B9-B9,10	3000	22.4	0.00746666 7	0.006	0.0033	
B9,10- CB9,10	3035	18.3	0.00602965 4	0.006	0.0033	
AB4-CB4	7500	10.5	0.0014	0.006	0.0033	
AB4- 4,5BA	3000	10.7	0.00356666 7	0.006	0.0033	
4,5BA- BA5	3000	21.9	0.0073	0.006	0.0033	
BA5- 5,6BA	3000	21.9	0.0073	0.006	0.0033	
5,6BA- AB6	3000	21.9	0.0073	0.006	0.0033	

AB6-6,7BA	3000	21.9	0.0073	0.006	0.0033	
6,7BA-AB7	3000	20.3	0.006766667	0.006	0.0033	
AB7-7,8BA	3000	20.3	0.006766667	0.006	0.0033	
7,8BA-AB8	3000	0	0	0.006	0.0033	
AB8-B8	3000	32.3	0.010766667	0.006	0.0033	
C13,14-C14	2115	28.3	0.01338061	0.006	0.0033	

- Spans that can be possibly affected at serviceability limit(As per eurocode 7)
- Spans where angular distortion exceed 1/500(as per euroced 7)

Settlement computed at the end of primary consolidation after completing strip foundation



Given that the soil may undergo creep due to rearrangement of soil particles, this phenomena is often represented by secondary consolidation settlement which calculated by the relationship below:

$S_s = C_s * H * \log(t_2/t_1)$, by estimating the consolidation rate will reach its maximum after 30 years and the primary consolidation end after 6years.

$$S_s = 0.007 * 3.6 * 0.69 = 0.017 \text{m.}$$

The total settlement is $S_t = 23.66 + 17 \text{mm} = 40.66 \text{mm} < 75 \text{mm}$ ok.

B. Retaining Wall

The retaining wall the critical section of 4m high was analyzed and the total settlement calculated is 111mm, which is greater than 75mm but less than 135mm.

C. Road Structure

The technical audit addresses the technical aspects of the project by auditing the design and construction issues to identify the possible causes of failure in the pavement structure. The audit was carried out through site visits, for general assessment. Office data review, detailed condition survey, and surface and subsurface drainage assessment.

Office data review includes a review of the available data of the project to evaluate the pavement structural design for upgraded section with distresses. Pavement condition survey is carried out in accordance with industry standard, to quantify the pavement distresses and to identify the possible causes of this distress.

The surface and subsurface drainage marked the real beginning of the observations made during the site visit. It is fairly certain that the contractor would still have to solve the problem of drainage at the entrance of the road section with distresses.

In general, the pavement along all road sections is new, with almost all surfaces of the road in excellent condition except one section along the retaining wall in reinforced concrete with structure distresses.

Analysis of the pavement thickness and general condition of the road in the section with severe distresses showed that:

- ◆ In the pavement design analysis, it should be noted that the distresses observed on one section of the road are relatively early in the service life of the pavement. As a result, these distresses might not be resulting from an insufficient pavement structural

section. Insufficient pavement structural section would reduce the pavement service life, rather than show very early structural failure observed on one section of the road;

- ◆ Materials used in the construction process are not the possible cause of pavement failure. Yet, in current best practices, four (4) meters height compaction is a considerable tallness of embankment to build without the use of reinforced earth system;

- ◆ In view of the developed damages recorded in the access road, the possible causes are mainly resulting from the embankment failure due too the movement of the retaining wall in reinforced concrete close to the swamp area. As a result of a cracked pavement, water flow not meeting the site conditions is causing accelerated failure of the pavement structure.

4.2.5. Structural Analysis

The objective of this structural study is to analyse the structural stability of the existing building and reinforced concrete retaining wall, and propose the strengthening method in case the structures are not safe.

4.2.5.1 Structural analysis of the building

- Visual analysis of the existing building structure
- Checking of structural safety of building structure
- Calculation of all the loads transmitted to each column
- Calculation of all the loads transmitted to each foundation,
- Calculation of soil pressure under each foundation,
- Make the comparison between the soil pressure under each foundation and soil bearing capacity,
- Calculation of settlement of each foundation
- If the soil pressure is bigger than the soil bearing capacity for some foundations, the strengthening method of those foundations will be developed,
- Bills of quantities of eventual strengthening works

ANALYSIS OF THE EXISTING BUILDING

The presence of cracks solely in the exterior masonry walls of the Gatuna Building raises concerns. Brick masonry cracking often stems from various ground movements like clay shrinkage, vibration, settlement, and others. It's probable that the cracking in the Gatuna walls results from a blend of ground movement and foundation settlement. Addressing these issues promptly is crucial to ensure the structural integrity and safety of the building.



Figure 22: Cracking in the Gatuna walls

LOADS FROM SLAB

i.Characteristic permanent Loads on the slab:

- Self-load: $0.05 \times 1 \times 1 \times 24 = 1.2 \text{ KN/m}^2$
- Secondary beam: $0.15 \times 0.2 \times 1 \times 24 = 0.72 \text{ KN/m}^2$
- Hourdis : $6 \times 0.10 = 0.60 \text{ KN/m}^2$
- Finishes = 1.5 KN/m^2

Total: 4.02 KN/m^2

ii.Characteristic live load (for general offices):

Characteristic live load: **2.5 KN/m^2** .

iii.Characteristic permanent load from main beam

Main beam: $0.3 \times 0.6 \times 1 \times 24 = 4.32 \text{ KN/m}$

iv.Characteristic load from columns

Column: $0.3 \times 0.3 \times 7.2 \times 24 = 15.552\text{KN}$

v.Walls characteristic loads

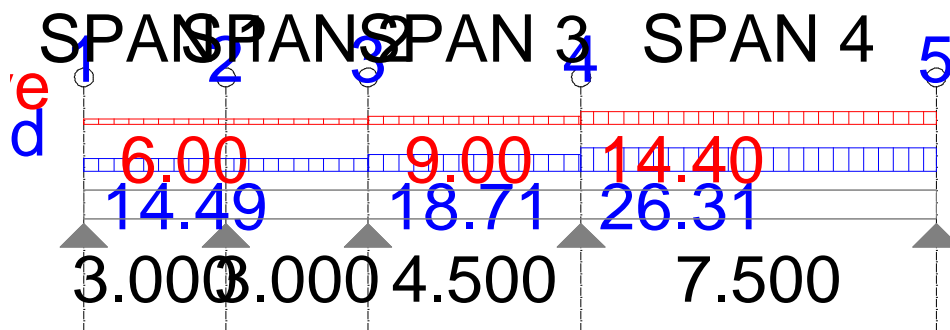
- Masonry wall: $0.2 \times 2.80 \times 1 \times 18 = 10.08\text{KN/m}$
- Plaster on the wall: $0.03 \times 2.80 \times 1 \times 20 \times 2 = 3.36 \text{ KN/m}$

Total wall: 13.44 KN/m

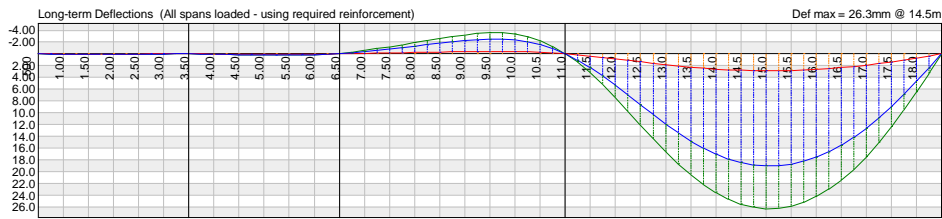
Characteristic load from roof: 2KN/m^2 .

B. BEAMS

1.Beam on the axis 6

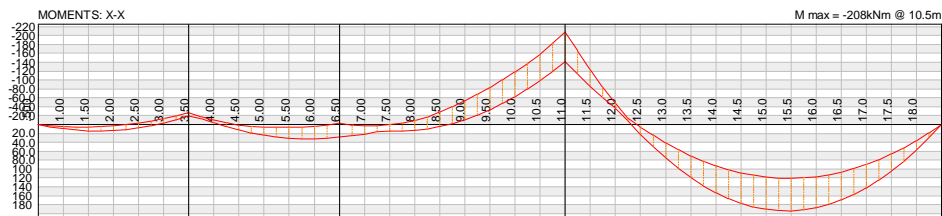


Beam deflection



Maximum long-term deflection: 2.63cm. Admissible deflection: $L/250 = 750/250 = 3\text{cm}$ OK.

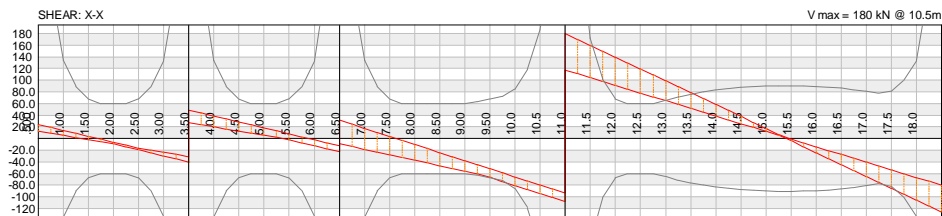
Bending moments



$M_{\text{max}}^- = 208\text{KNm}$

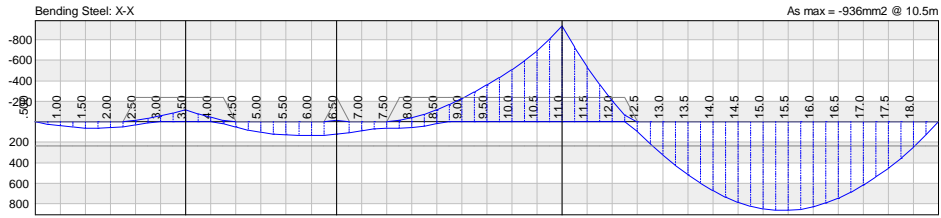
$M_{\text{max}}^+ = 194\text{KNm}$

Shear forces



$V_{\text{max}} = 180\text{KN}$

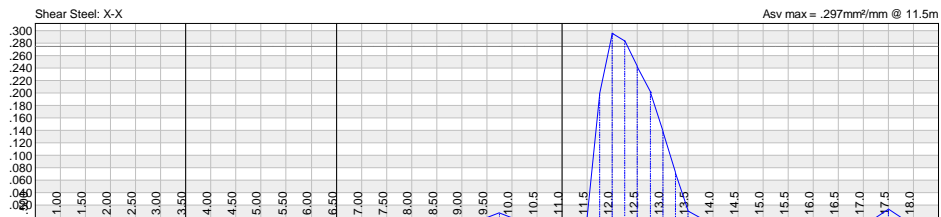
Required main steel reinforcement



As top: 936mm².

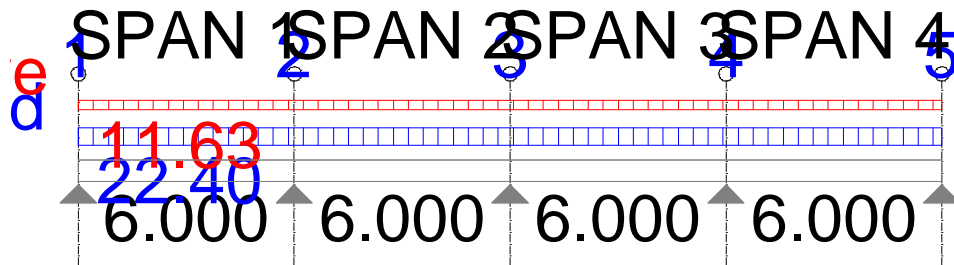
As bottom: 865 mm².

Required stirrups

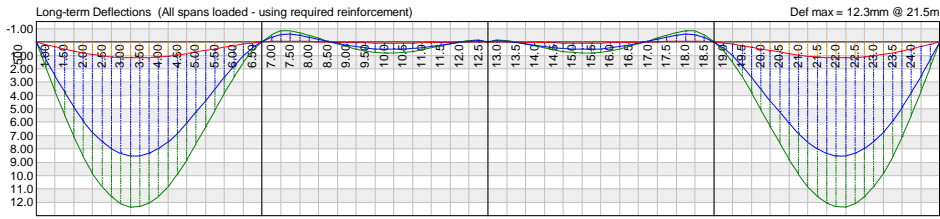


Required: 0.297mm²/mm.

2.Beam between axis c-b

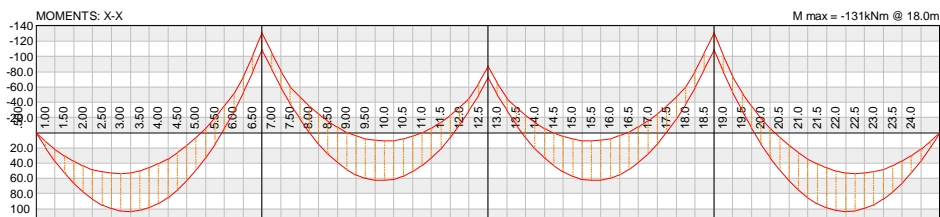


Beam deflection



Maximum long-term deflection: 1.23cm. Admissible deflection: $L/250 = 600/250 = 2.4\text{cm}$ OK.

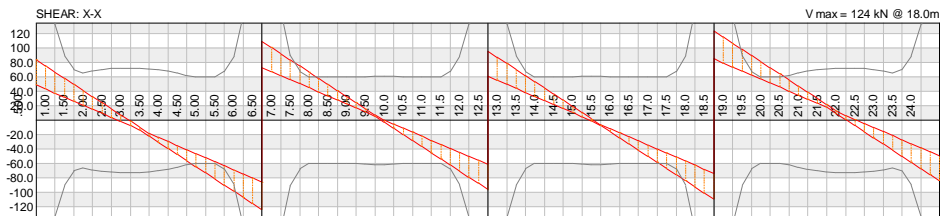
Bending moments



$M_{\text{max}}^- = 131\text{KNm}$

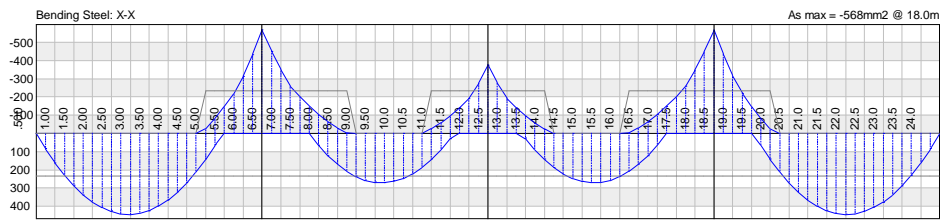
$M_{\text{max}}^+ = 104\text{KNm}$

Shear forces



$V_{\text{max}} = 124\text{KN}$

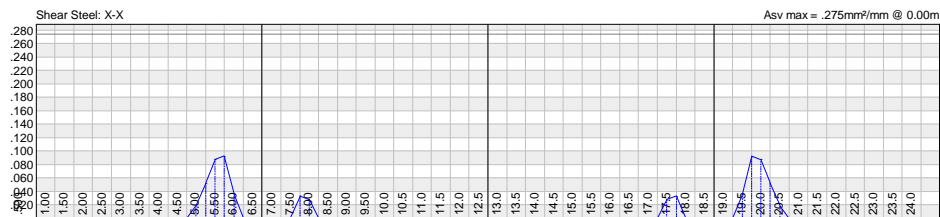
Required main steel reinforcement



As top: 568mm². < 3Φ20 = 942mm² = Existing steel

As bottom: 445mm² < 4Φ16 = 804mm² = Existing steel

Required stirrups



Required: 0.275mm²/mm < existing 2Φ8@150mm = 0.673mm²/mm

D.DESIGN OF COLUMNS

Loads transmitted on each column are calculated by considering the loads in column tributary area.

The most loaded column is CB-7 supporting a total characteristic load equal to 544.54KN

Total live characteristic load: 90KN

Total permanent characteristic load: 544.54-90 = 454.54KN

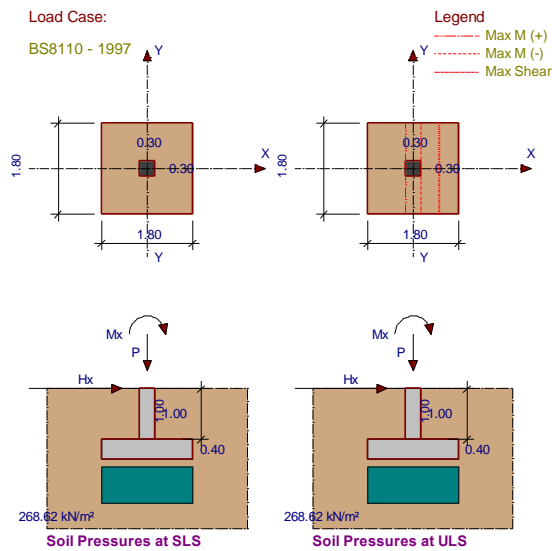
COLUMNS DESIGN LOAD: 1.4 x 454.54 + 90 x 1.6 = 780.356 KN

Required steel at the ground floor part of the column

Design results for load case :				
		Top	Middle	Bottom
N	(kN)	780.4	780.4	780.4
M _{xadd}	(kNm)	13.3	0.0	13.3
M _{yadd}	(kNm)	13.3	0.0	13.3
M _x	(kNm)	13.3	0.0	13.3
M _y	(kNm)	13.3	11.7	13.3
M _f x (kNm)		21.2	11.7	21.2
Design axis		X-X	Y-Y	X-X
A _{sc}	(mm ²)	360	360	360
A _{sc} /A _c (%)		0.40	0.40	0.40
A _{sc} perpendicular		360	360	360
Critical load case		Case 1		

Required steel: $360\text{mm}^2 < 6\Phi 16 = 1206\text{mm}^2$. Existing steel

D. DESIGN OF FOUNDATIONS USING DESIGN LOADS



Required steel reinforcement

Output for Load Case	
Soil pressure (ULS)(kN/m ²)	268.62
Soil pressure (SLS)(kN/m ²)	268.62
SF overturning (SLS)	>100
SF overturning (ULS)	>100
Safety Factor slip (ULS)	>100
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	67.78
Reinforcement X (mm ² /m)	466
Design moment Y (kNm/m)	67.78
Reinforcement Y (mm ² /m)	466
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm ² /m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm ² /m)	0
Linear Shear X (MPa)	0.273
vc (MPa)	0.347
Linear Shear Y (MPa)	0.273
vc (MPa)	0.347
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.162
vc (MPa)	0.347
Cost	613.47

Required steel: 466mm²/m at the bottom < Existing $\Phi 16@200\text{mm}^2$.

CODES: BS8110-1997

Software: PROKON

A. Checking of the stability of existing retaining wall under static loads

Maximum soil pressure: $94.6\text{kPa} < 100\text{kPa}$, OK

Stability against sliding: $2.19 > 1.5$ OK

Stability against: $3.79 > 1.5$ OK

Required steel reinforcements:

Under static loads the soil pressure is lesser than the soil bearing capacity

SETTLEMENT OF THE RETAINING WALL

S/No Loads Retaining wall settlement (mm)

1 Under static loads 111

2 Under static and traffic loads Collapses

4.2.5. Additional Soil Investigation

Additional field and laboratory testing and the site subsoil strata evaluation deemed necessary at the section close to the swamp may be carried out.

Need for additional investigation may arise based on the chosen a solution among the proposed ones hereunder for more details. To this end,we have set up a list of additional investigation works that need to be performed to acquire the missing information about the soil.

The present consultancy services report aims at providing the solutions suitable for the stability of the structures close to the swamp area.

4.3 MANAGEMENT AND PREVENTION OF ROAD CRACKS AND BUILDING SETTLEMENT

The following is the guide for Management and Prevention of Road Cracks and Building Settlement

Based on Geology, Lithology, Soil Type, and Engineering Techniques:

■ **Geological and Lithological Assessment:**

Conduct a thorough geological and lithological assessment of the site to understand the underlying formations and soil types.

Identify any potential geological hazards such as fault lines, expansive soils, or areas prone to landslides.

■ **Soil Analysis:**

Perform soil testing to determine the physical and chemical properties of the soil, including its composition, moisture content, and bearing capacity.

Assess the presence of expansive soils, organic materials, or other factors that may contribute to road cracks and building settlement.

■ **Engineering Techniques:**

Subsurface Drainage: Implement subsurface drainage systems to mitigate the effects of water infiltration and reduce the risk of soil saturation, which can lead to settlement and road pavement deterioration.

Soil Stabilization: Utilize soil stabilization techniques such as chemical additives, mechanical compaction, or geosynthetic reinforcement to improve the strength and stability of the soil.

Foundation Design: Design foundations that are appropriate for the site-specific soil conditions, considering factors such as load-bearing capacity, settlement potential, and lateral stability. Options may include shallow foundations, deep foundations, or innovative foundation systems tailored to the geological context.

Pavement Design: Develop pavement designs that account for the anticipated traffic loads, soil characteristics, and environmental conditions. Consider incorporating flexible pavement structures, proper base course materials, and effective drainage systems to minimize the risk of road cracks and pavement failure.

■ **Preventive Maintenance:**

Implement a proactive maintenance program to identify and address potential issues before they escalate. This may include regular inspections, crack sealing, pothole repair, and pavement rehabilitation techniques.

Conduct routine monitoring of buildings and infrastructure to detect signs of settlement, cracking, or structural distress early on, allowing for timely intervention and remediation.

▣ Collaborative Approach:

Foster collaboration between geologists, geotechnical engineers, civil engineers, and construction professionals throughout the project lifecycle. Integrated planning and decision-making can help optimize design solutions and mitigate geological risks effectively.

Encourage ongoing communication and knowledge sharing among stakeholders to ensure a holistic understanding of the geological and engineering challenges associated with road and building construction.

▣ Continual Improvement:

Emphasize the importance of continual improvement and adaptation in response to changing geological conditions, technological advancements, and lessons learned from previous projects.

Invest in research and development efforts to explore innovative materials, construction techniques, and monitoring technologies that can enhance the resilience and longevity of road infrastructure and buildings in geologically complex environments.

By following these guidelines and integrating geological considerations, soil analysis, engineering techniques, and proactive maintenance practices, stakeholders can effectively manage and prevent road cracks and building settlement associated with geological and lithological factors. This holistic approach promotes sustainable development and resilience in infrastructure projects, ultimately enhancing safety, durability, and performance over the long term.

4.4 HYPOTHESES VALIDATION

4.4.1 Fragile Soils on Wetlands

Generally, the major part of the existing Gatuna Border Post structures are located in the depression near the Mulindi-Gatuna Wetland.

The dominant soils are a result of alteration of the granite and the gneiss. Disruption of drainage due to tectonic movements of the Pleistocene caused the formation of alluvial valleys. The soils consist of alluvium and colluvium in the basin as a result of erosion. They have generally colluvial and alluvial in the valleys around rivers.

However, the soil features vary from an ecological aspect to another, what makes a diversity of soils depending on whether one is at the summit of the mountain, at the lower level of the slope or in the shallows. Swamps and shallows are characterized by clayey soils, rich in silts and covered in some places by the alluviums and colluviums. The presence of humidity is very high.

The main destructive agent of the soil is rain erosion. In some sites, active erosion mainly caused landslide hazards, which increase sediments in rivers. Other than that, erosion has also formed gully bodies through the slopes of mountainous areas in the project area.

Construction activities increase the potential of occurrence of landslides and erosion in various ways, which include destabilization of rock masses by cuts in slopes, improper stockpiling of materials, destruction of vegetative cover during site clearing and uncontrolled surface run-off (slope wash) during storms may increase the erosion rate.

Nevertheless, erosion is of relevance to slope stability, which is in turn relevant to the design of the project and the conduct of operations such as excavation, quarrying and borrowing.

So fragile soils on wetlands sides surrounding the building are associated with building settlement and the road and buildings are on sale and organic material (peat soil) it is mentioned in figure 4 .

4.4.2 Poor Compaction and Sub Base Quality

The technical audit addresses the technical aspects of the project by auditing the design and construction issues to identify the possible causes of failure in the pavement structure. The audit was carried out through site visits, for general assessment. Office data review, detailed condition survey, and surface and subsurface drainage assessment.

Office data review includes a review of the available data of the project to evaluate the pavement structural design for upgraded section with distresses. Pavement condition survey is carried out in accordance with industry standard, to quantify the pavement distresses and to identify the possible causes of this distress.

The surface and subsurface drainage marked the real beginning of the observations made during the site visit. It is fairly certain that the contractor would still have to solve the problem of drainage at the entrance of the road section with distresses.

In general, the pavement along all road sections is new, with almost all surfaces of the road in excellent condition except one section along the retaining wall in reinforced concrete with structure distresses.

Analysis of the pavement thickness and general condition of the road in the section with severe distresses showed that:

- In the pavement design analysis, it should be noted that the distresses observed on one section of the road are relatively early in the service life of the pavement. As a result, these distresses might not be resulting from an insufficient pavement structural section. Insufficient pavement structural section would reduce the pavement service life, rather than show very early structural failure observed on one section of the road;
- Materials used in the construction process are not the possible cause of pavement failure. Yet, in current best practices, four (4) meters height compaction is a considerable tallness of embankment to build without the use of reinforced earth system;
- In view of the developed damages recorded in the access road, the possible causes are mainly resulting from the embankment failure due to the movement of the retaining wall in reinforced concrete close to the swamp area. As a result of a cracked pavement, water flow not meeting the site conditions is causing accelerated failure of the pavement structure

Present data and analysis refuting Hypothesis 2 saying that: Poor compaction and quality of the sub-base used during roads construction are major causes of road cracks.

4.4.3 Structural Building Frame and Foundations

The Gatuna Building has cracks only in the outside masonry walls.

The most common causes of brick masonry cracking are: Ground movement caused by clay shrinkage, land slip, vibration, subsidence, settlement, heave, sway, and so on.

The Gatuna wall cracking is probably caused by a combination of ground movement and foundation settlement. Present data and analysis refuting Hypothesis 3 saying that the structural building frame contributes to building settlement due to the type of foundations used.

4.4.4 Road Cracks and Soil Types/Dry Season

The Gatuna Building has cracks only in the outside masonry walls.

The most common causes of brick masonry cracking are: Ground movement caused by clay shrinkage, land slip, vibration, subsidence, settlement, heave, sway, and so on.

The Gatuna wall cracking is probably caused by a combination of ground movement and foundation settlement. Organic material soil during dry seasons can shrink and lose moisture content, leading to increased settlement, cracking, and reduced bearing capacity for roads and building foundations. Compaction becomes challenging, posing risks of uneven surfaces and structural failures. Furthermore, heightened fire susceptibility and environmental concerns arise, necessitating specialized engineering solutions and adherence to stringent regulations for sustainable construction practices.

Present data and analysis are supporting Hypothesis 4 saying that Road cracks are associated with soil types and the dry season (summer). .

4.6 SUMMARY OF FINDINGS

The compound of Gatuna One Stop Border Post has faced a problem of settlement where the cracks appeared on surface and water seepage was detected. A hydrological study was conducted with an aim of obtaining the technical inputs for further engineering solutions to contribute to the stability of the building and road on Gatuna One Stop Border Post. Thereafter a hydrological study was conducted by determining the hydraulic conductivity of different geological layers of the site and characterizing the aquifers and discharge calculations, and the rainfall-runoff relationship within the concerned catchment was performed in order to determine the flooding level near the retaining The findings of this study, centered around the Gatuna One-Stop Border Post area, were achieved through the utilization of Geographic Information System (GIS) technology to identify and analyze the major causes of road cracks and building settlement. The assessment using GIS technology aimed to understand the geological composition, traffic volume, road design, and maintenance practices contributing to road deterioration and building settlement. Through satellite imagery, topographic maps, and geospatial data, the study created comprehensive maps highlighting areas of concern, particularly focusing on the geological framework, including organic material, quartzite+shale, and shale components.

The study identified several significant factors contributing to road cracks and building settlement. For roads constructed on shale rock, issues such as slope instability, weathering, and poor load-bearing capacity were prominent, leading to hazards for road users and infrastructure. Conversely, building and road infrastructure constructed on organic material soil, particularly peat-rich soil, experienced settlement and subsidence due to its compressible nature, low bearing capacity, and susceptibility to waterlogging. Additionally, decomposition processes in organic material soil exacerbated settlement and structural deterioration over time, posing ongoing maintenance challenges.

Structural analysis of existing buildings at Gatuna One-Stop Border Post revealed cracks predominantly in outside masonry walls, likely caused by ground movement and foundation settlement. Foundation design and load-bearing capacity were critical considerations for stability assessments, with some foundations exhibiting safety concerns under dynamic loads.

Management and prevention strategies proposed based on geological, lithological, and engineering considerations emphasized thorough site assessments, soil analysis, implementation of subsurface drainage systems, soil stabilization techniques, and proactive maintenance practices. Collaboration among stakeholders and continual improvement efforts were highlighted to enhance the resilience and longevity of infrastructure in geologically complex environments.

Hypotheses validation indicated that fragile soils on wetlands, poor compaction and sub-base quality, structural building frame, and dry seasons are indeed associated with road cracks and building settlement, confirming the importance of geological and engineering factors in infrastructure stability.

The study underscores the critical role of GIS technology in identifying and analyzing the causes of road cracks and building settlement, providing valuable insights for informed decision-making and sustainable infrastructure development. By integrating geological assessments, soil analysis, and engineering techniques, stakeholders can effectively manage and prevent road cracks and building settlement, ensuring the durability and safety of infrastructure in geologically diverse environments.

CHAPTER.V. CONCLUSION, PROPOSED SOLUTIONS ALTERNATIVES AND RECOMMENDATION

This chapter show the significance of road construction and maintenance, particularly in the context of Gatuna One Stop Border Post, the chapter concludes by outlining the pressing challenges faced and the inception of a new project aimed at addressing these issues.

5.1 CONCLUSION

The construction on soil containing shale and organic material poses significant challenges due to its propensity for cracks and settlement, which can compromise the structural integrity of buildings. These issues arise from the unique properties of shale and organic material, including their susceptibility to expansion, contraction, and decomposition over time. As a result, thorough assessment and appropriate mitigation strategies are imperative to ensure the safety and longevity of structures built on such soil types.

So, the Maya Trade Center and Gatuna OBP Office, along with the roads in the vicinity, are built on organic material, which serves as the underlying cause of building settlements and road cracks observed in this area due to the behavior of Shale and Organic material.

- **Topographical Survey:** The topography and landscape of Gatuna OSBP is a mountainous area with a succession of high-altitude erosive relief steep hills, separated by deep and narrow valley located in the depression near the Mulindi-Gatuna Wetland, these steeper gradients are more susceptible to road cracks and building settlement. Higher slopes can accelerate water runoff, leading to erosion of the subgrade and base layers, weakening structural foundations and increasing the likelihood of cracking and settlement issues.
- **Hydrology Study:** The hydrological analysis found that the project site is on the unconfined aquifer for which water table can rise up to the surface; The hydrological study focused on issues of groundwater that can rise up to the foundation of the structures (building, retaining wall and water tank) even up to ground surface due to the seepage and runoff from the flooding of the marshland. The hydrologic analysis found that the project site is on the unconfined aquifer for which water table can rise up to foundation. Therefore, a drainage system was designed for lowering/limiting water table below 1 m of the foundation. Poor drainage can cause water accumulation and soil saturation, which weakens the soil bearing capacity and promotes settlement, particularly in low-lying areas.

- **Geotechnical Assessment:** Subsequent to the computation of the settlements, all the pad foundation of the newly constructed building of the border post will undergo settlement $\leq 80\text{mm}$ within a period of 10 years. On the other hand, it must be considered that the results above are resulting from the borehole data that have been drilled some meters outside the building foundation area. It is therefore advisable, before taking expensive remedial action, to drill 3 additional boreholes within the buildings. The retaining wall is safe under permanent loads, but its settlement is greater than allowable value under the combination of permanent and dynamic/traffic loads.
- **Structural Assessment:** The structural assessment shows that, under permanent and full live loads, all the foundations of the building at the border are safe but under permanent, full live load and dynamic (traffic vibrations) only one foundation on the axis E-3 is not safe.

Insufficient reinforcement in Pavement and Building foundations: An evaluation of structural materials might show that reinforcement in pavements and foundations is insufficient to withstand the load demands and soil conditions. For roads, this could mean that pavements are more prone to cracking, especially under high traffic loads, while buildings may experience foundational weaknesses that lead to settlement issues.

5.2 PROPOSED SOLUTIONS ALTERNATIVES.

The proposed alternative solution is on first of all at hydrology study focused on designing a drainage system to lower the water table below 1 meter of the existing retaining wall and manage runoff from the marshland effectively. Following a geotechnical assessment, three stabilization solutions were proposed, with reinforced earth walls recommended for immediate embankment stabilization due to their cost-effectiveness and flexibility. Structural assessment outcomes highlighted the need for interventions on foundation E-3, suggesting either load limitations or retrofitting options to address safety concerns.

Hydrology Study:

A drainage system was designed for lowering water table below 1 m of the foundation of the existing retaining wall and conveying the runoff contribution from the marshland.

Geotechnical Assessment:

After the Geotechnical analysis, it was worked out 3 different solutions that are suitable to the stabilization of the slope and the structures close to the swamp such as road and buildings as follow: Bored piles, Soil nail; Reinforced Earth RE.

The geotechnical assessment recommends stabilizing the embankment with immediate effect using a reinforced earth (RE) wall. The reinforced earth retaining walls are very cost effective than the rest of the proposed solutions above as well as conventional concrete retaining walls too. Furthermore, these systems are more flexible than the conventional earth retaining walls such as reinforced concrete-cantilever or gravity walls.

Following the outcomes of the structural assessment under permanent, full live load and dynamic (traffic vibrations) only one foundation on the axis E-3 is not safe. The assessment recommends therefore:

- 1: We recommend limiting the live load to 1KN/m² in the offices around the columns on the axis E-3;
- 2: Retrofitting the footing of the columns on the axis E-3.

As a result of the conservative approach, 20% of footings will have settlement ≥ 65 mm, we are proposing 2 solutions: 3: Retrofitting the foundations and 4: Monitoring of the building with high precision topographical survey equipment.

Geotechnical Investigation: Conduct a comprehensive geotechnical investigation to assess the characteristics of the soil, including its composition, moisture content, and load-bearing capacity. This data will inform the design and construction process, enabling engineers to implement suitable solutions.

Soil Stabilization: Employ soil stabilization techniques such as chemical stabilization or soil reinforcement to enhance the strength and stability of the soil. This may involve the addition of stabilizing agents or the installation of geosynthetic materials to mitigate the effects of expansion and settlement.

Foundation Design: Design custom foundations tailored to the specific challenges posed by shale and organic material soil. Options may include deep foundations such as piles or caissons to bypass unstable soil layers and reach more stable strata capable of supporting the structure's load. **Structural Monitoring:** Implement a comprehensive structural monitoring program to detect early signs of movement or distress within the building. This may involve the installation

of sensors and regular inspections to assess the ongoing performance of the structure and identify any issues promptly. **Alternative Building Materials:** Explore alternative building materials and construction techniques that are better suited to the challenges posed by shale and organic material soil. For example, lightweight construction methods or prefabricated modular components may reduce the load on the foundation and minimize the risk of settlement.

Following the review of existing documents, the observation made at the site facilities and surrounding in Gatuna and based on experience of Redstone personnel, we have come to the conclusion that the main source of the damages are to be found in the nearness of the main building and the new road to the type of soil in the swamp.

The swamp soil (RRI report) shows, for the investigated depth of max. 12 m, a sequence of peat, up to 5 m thick, followed by clay of low to good consistency. Although none of the study reviewed by Redstone, mention the existence of ground water, we have observed at the site, that the irrigation canal for the tea plantation bear water. Hence, we assume that the swamp soil is in a saturated condition. Such soil has a low bearing capacity, a high degree of plastic deformability and a low permeability. Once subjected to any additional static load, machine and traffic vibration this soil reacts with a volume change.

The consequence of the volume change is a sudden movement of the slope soil that destabilize the soil and any structure nearby the slope.

The road Kigali Gatuna is built for a considerable length 28 Km out of the total 78 Km along swampy areas. At 3 points the road is directly crossing extended swamps where, since the first construction of the road took place, the repeated damages have occurred. The 3 areas of swamp crossing were finally successfully repaired during the recent rehabilitation works.

Here one can mention the recent major failure of the road at the swamp side at Km 77 in that interrupted the traffic during several hours? The road centerline was then realigned of several meter and rebuilt toward the uphill slope.

The swamp soil at Gatuna OSBP has proven to be unstable and the source of damages has occurred at the main border building and at the newly built road.

Since the start of the contractual delay Redstone Ltd has proceeded with the review of existing documents, Redstone personnel has also visited the site on several occasion.

In order for the Client to make an informed decision about the different solution, as proposed by Redstone at Gatuna OSBP, the cost, risk, working wise, and equipment are presented here under.

5.3. RECOMMENDATIONS

The recommendations for infrastructure improvements, maintenance strategies, and land-use planning are derived from the GIS analysis, offering valuable insights for policymakers, urban planners, and infrastructure developers in the Kigali-Gatuna region.

Based on the challenges associated with construction on shale and organic material soil, it is recommended to adopt a multidisciplinary approach that integrates geotechnical engineering, structural design, and construction management. Collaboration between experts in these fields is essential to develop effective solutions tailored to the site-specific conditions and mitigate the risks of cracks and settlement. Additionally, ongoing monitoring and maintenance should be prioritized to ensure the long-term stability and safety of the build

- After this hydrological study, the following recommendations were provided: The filter material should be selected based on the grain size distribution of the excavated trenches.
- The collector should be constructed at the toe of the sloping side.
- Considering the site conditions, the installation of piezometer should be done for a good monitoring of piezo metric level.
- **We recommend also to proceed as follows:** Limiting the live load to 1KN/m² in the offices around the columns on the axis E-3;
- Stabilizing the embankment with immediate effect using the reinforced earth (RE);
- Immediate start of the regular monitoring of the building and carry out retrofitting before the settlements reach 50mm.

Recommendations for ground improvement were provided, including the identification of computer-aided software used and descriptions of design alternatives. Standard penetration tests were conducted to understand the design and construction works of the roads. Multiple boreholes were designed to measure hydrological data, such as surface and subsurface drainage, sanitation works, and groundwater level fluctuations, using tape and depth measurements. Additionally, interviews with elders, contractors, and engineers were conducted to gather historical information on the initial observation of cracks at the site.

Bibliography

- Ahmed, M. A. (2016., June 01). EVALUATION OF CONE PENETRATION TEST (CPT) CLASSIFICATION METHODS FOR SOME LOCAL SOILS. *researchgate*.
- AIMABLE, R. (2013). *DEVELOPING ALTERNATIVE CONSTRUCTION AND MAINTENANCE STANDARDS FOR EFFICIENT AND JUSTIFIED INVESTMENTS IN RURAL UNPAVED ROADS IN RWANDA*. University of Rwanda. Kigali: University of Rwanda.
- Atmaca, N. A. (18-19 October, 2018). THE SHEAR WALL EFFECT ON CONCRETE STRUCTURES. *3rd International Energy & Engineering Congress Proceedings Book*. Gaziantep University, Turkey.
- BONISHULI, C. (2014, July). EVALUATION OF DAMAGE CAUSED TO ROAD STRUCTURAL PAVEMENTS BY UTILITY SERVICE PROVIDERS in Kigali.
- DAVID WILKIE, E. S. (2000). Roads, Development, and Conservation in the Congo basin. *Conservation biology, Volume 14, No. 6, December 2000*.
- District, G. (n.d.). *Gicumbi Local Urban Development Plan*. Gicumbi District on <http://www.gicumbi.gov.rw/fileadmin/templates/document>.
- geotechnique, R. (2018). *Financial Proposal, Methodology Approach & Work Schedule*. KIGALI.
- Group, A. D. (2013). *Rwanda Transport Sector Review and Action Plan, RWANDA*.
- Hoang_Hung Tran NGuyen, H. W. (2017). *Proceeding of 4TH International congress de Geotechniques _Ouvrages Structures*. HO Chi Mnh city, Vietnam: Springer.
- Jamal, H. (2017). *What are Road Cracks? How they are Formed .html*.
- JICA. (2019). *Data collection survey on development of urban roads in urban transport system ,kigali city*.
- Karuhanga, J. (2019, January 03). *Investments in infrastructure in 2018 reflect resolve, direction Rwanda chose*. Retrieved April 2020, from www.newtimes.co.rw/news.
- LTD, C. C. (2017). *Geotechnical factual report Gatuna One Border Post Rwanda Report No.: 303-00716/01/R0 dated August 2019*. KIGALI: CAVICON.

- Masarova, J., & Ivanova, E. (2013, August). IMPORTANCE OF ROAD INFRASTRUCTURE IN THE ECONOMIC DEVELOPMENT AND COMPETITIVENESS. *Research gate*.
- MINIFRA. (2017). - *MININFRA Contract for Consulting Services for preparation of the designs and bidding documents of the construction works for the establishment of the Juxtaposed border post at Gatuna*. KIGALI: MI NIFRA .
- MINIFRA. (2018). - *Environmental and Social Impact Assessment for the construction and operation of One Stop Border Post at Gatuna, Rwanda*. KIGALI: MINIFRA.
- Mohammad Javed, B. A. (2012, september). *FIELD TESTING OF REINFORCED CONCRETE BRIDGES FOR SHEAR AND FLEXURAL STRENGTHENING IN PESHAWAR, PAKISTAN*. Retrieved April 2020, from researchgate.net.
- Mohammadi, M. A. (2015, May). The new empirical formula based on dynamic probing test results in fine cohesive soils. *International journal of Civil Engineering, Geotechnique, Vol. 13(No.2)*.
- Newtimes. (2017). How One Stop Border posts have helped reduce truck transit time.
- Patil Vilas K1, K. M. (2018). DGPS BASED DIGITAL TOPOGRAPHIC SURVEY, A Case Study of Tembhapuri Reservoir, Aurangabad District, Maharashtra, India. *6th International Conference on Recent Trends in Engineering & Technology (ICRTET - 2018)*. IJREAM.
- Reaffirmed. (2013, May). “*Non Destructive Testing of Concrete-Methods of Test(Rebound hammer)*”. Retrieved April 04, 2020, from [http://www.ircen.gov.in/LAB: IS-13311 \(Part 2\):1992](http://www.ircen.gov.in/LAB: IS-13311 (Part 2):1992)
- RTDA. (2018). *Architecture drawing*. rtda.
- RTDA. (208). *Geotechnical report* . KIGALI: RTDA.
- STANIEK1, M. (2017). DETECTION OF CRACKS IN ASPHALT PAVEMENT DURING ROAD INSPECTION PROCESSES. *Scientific Journal of Silesian University of Technology. Series Transport., 96(ISSN: 0209-3324,DOI: <https://doi.org/10.20858>), <http://sjsutst.polsl.pl>*.
- Taher, b. (2019). *Reason why road crack? BSc Civil and Environmental Engineering, Shahjalal University of Science and Technology*.

<https://www.princeton.edu/~alaink/Orf467F07/GNSS.pdf>.

https://www.eqc.govt.nz/sites/public_files/images/What%20is%20a%20cone%20penetration%20test.pdf

<https://www.proceq.com/company/history>

<http://www.ircen.gov.in/LAB/res/pdf/test-31.pdf>).

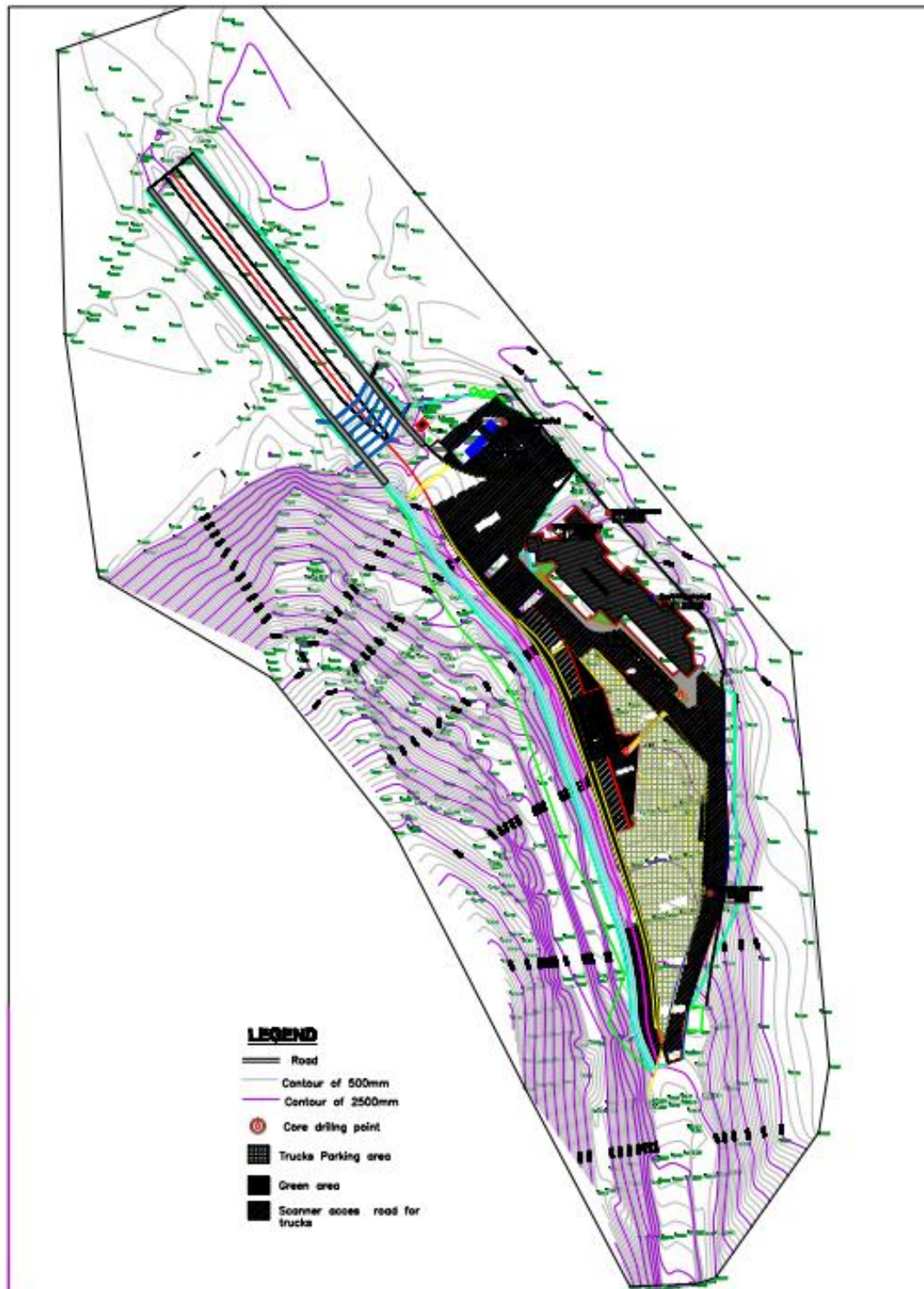
https://www.youtube.com/playlist?list=PLiK7qkGjS_tJVuMZclQdINgLSOOGN6Jmk

<https://www.yumpu.com/en/document/read/34118614/users-manual-dynamic-probing-geostru-software>

<https://www.newtimes.co.rw/news/investments-infrastructure-2018-reflect-resolve-direction-rwanda-chose>

APPENDICES

Appendices 1: Figure showing the Plan of the site and geometry of the Slope



, Figure 8 : Plan of the site and geometry of the Slope

Appendices 2 : Figure showing core drilling Point

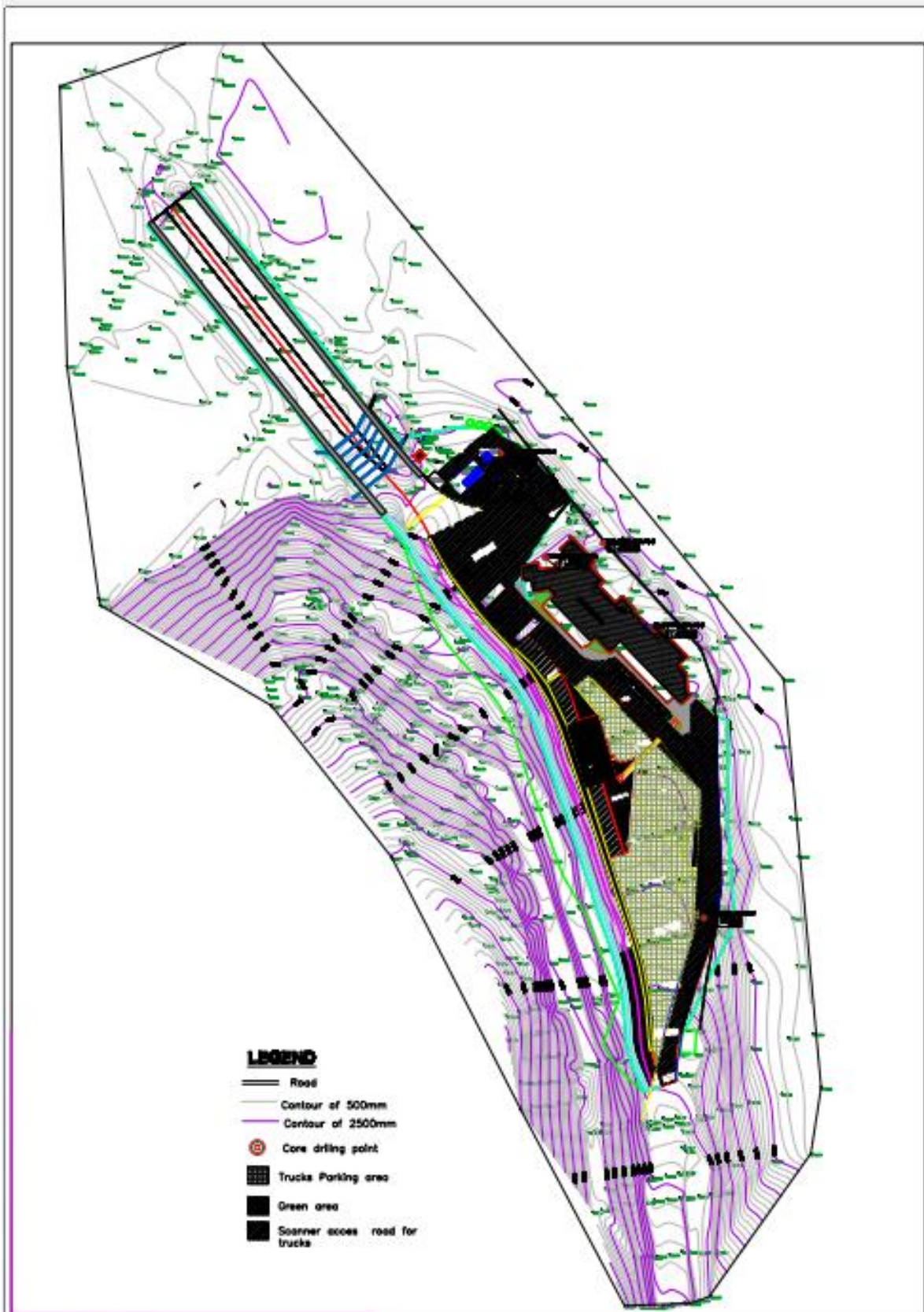


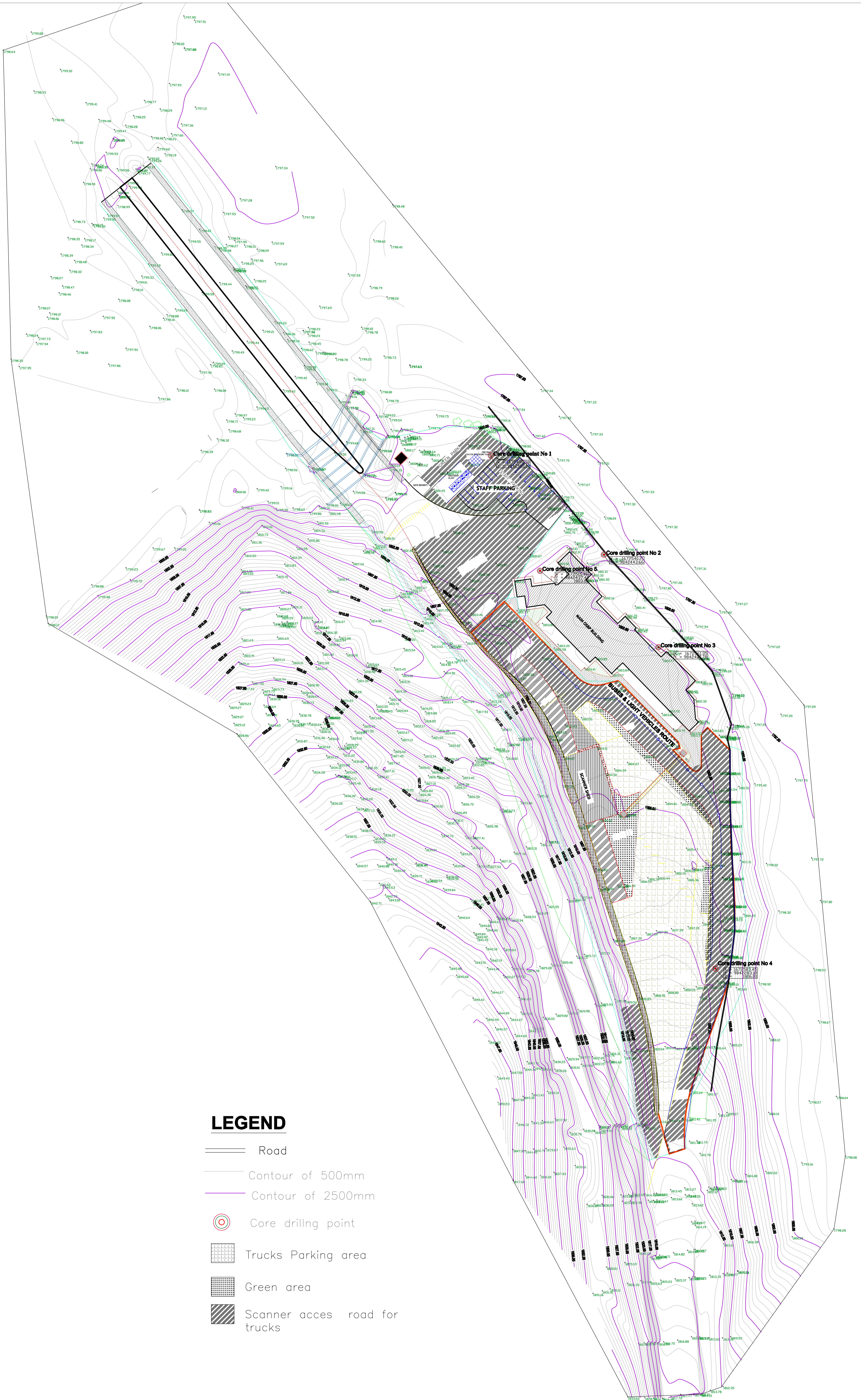
Figure 9 : showing core drilling Point

Appendices 3: table showing Stratigraphic Layer of the drilled boreholes

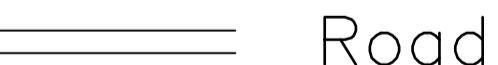



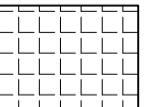


Elevation (m)					
1807					BH-04 : 1807.03
1807					Silt
1806					Silt
1806					Firm clay
1805					Gravel of limestone
1805					Gravel of limestone
1804					Gravel of limestone
1804					Gravel of limestone
1803					Gravel of limestone
1803		BH-05 : 1802.86		BH-03 : 1802.76	Gravel of limestone
1802		Silt		Pavement with backfill material	Gravel of limestone
1802		Silt		Pavement with backfill material	Gravel of limestone
1801		Silt	BH-02 : 1801.23	Pavement with backfill material	Gravel of limestone
1801	BH-01 : 1800.74	Sand	Sand silt	Sand silt	Firm clay
1800	Sand with some gravel	Sand	Sand silt	Sand silt	Firm clay
1800	Sand with some gravel	Sand	Sand silt	Sand silt	Firm clay
1799	Sand with some gravel	Very soft black clay	Sand silt	Clay with gravel of limestone	Very soft black peat with clay
1799	Sand with some gravel	Very soft black clay	Sand silt	Clay with gravel of limestone	Very soft black peat with clay
1798	Sand with some gravel	Very soft black clay	Gravel of limestone	Clay with gravel of limestone	Very soft black peat with clay
1798	Sand with some gravel	Very soft black clay	Gravel of limestone	Sand with some clay	Very soft black clay with peat
1797	Black clay	Very soft black clay	Gravel of limestone	Sand with some clay	Very soft black clay with peat
1797	Black clay	Gravel of limestone	Gravel of limestone	Sand with some clay	Very soft black clay with peat
1796	Black clay	Gravel of limestone	Gravel of limestone	Sand with some clay	Very soft black clay
1796	Black clay	Gravel of limestone	Gravel of limestone	Sand with some silt/clay and gravel	Very soft black clay
1795	Black clay	Gravel of limestone	Gravel of limestone	Sand with some silt/clay and gravel	Very soft black clay
1795	Sand with some gravel	Gravel of limestone	Sand clay with peat	Sand with some silt/clay and gravel	Very soft Sandy clay
1794	Sand with some gravel	Gravel of limestone	Clay with peat	Sand with some silt/clay and gravel	Very soft Sandy clay
1794	Clay	Gravel of limestone	Clay with peat	Sand with some silt/clay and gravel	Very soft Sandy clay
1793	Gravel of limestone	Gravel of limestone	Clay with peat	Sand with some silt/clay and gravel	Gravel of limestone with some clay
1793	Gravel of limestone	Gravel of limestone	Clay with peat	Sand with some gravel	Gravel of limestone with some clay
1792	Gravel of limestone	Gravel of limestone	Clay with peat	Sand with some gravel	Gravel of limestone with some clay
1792	Gravel of limestone		Clay with peat	Sand with some gravel	
1791	Gravel of limestone		Sand	Sand with some gravel	
1791	Gravel of limestone		Sand	Sand with some gravel	
1790	Gravel of limestone		Sand	Sand with some gravel	
1790	Gravel of limestone		Clay with occasional limestone	Gravel of limestone with sand silt	
1789	Gravel of limestone		Clay with occasional limestone	Gravel of limestone with sand silt	

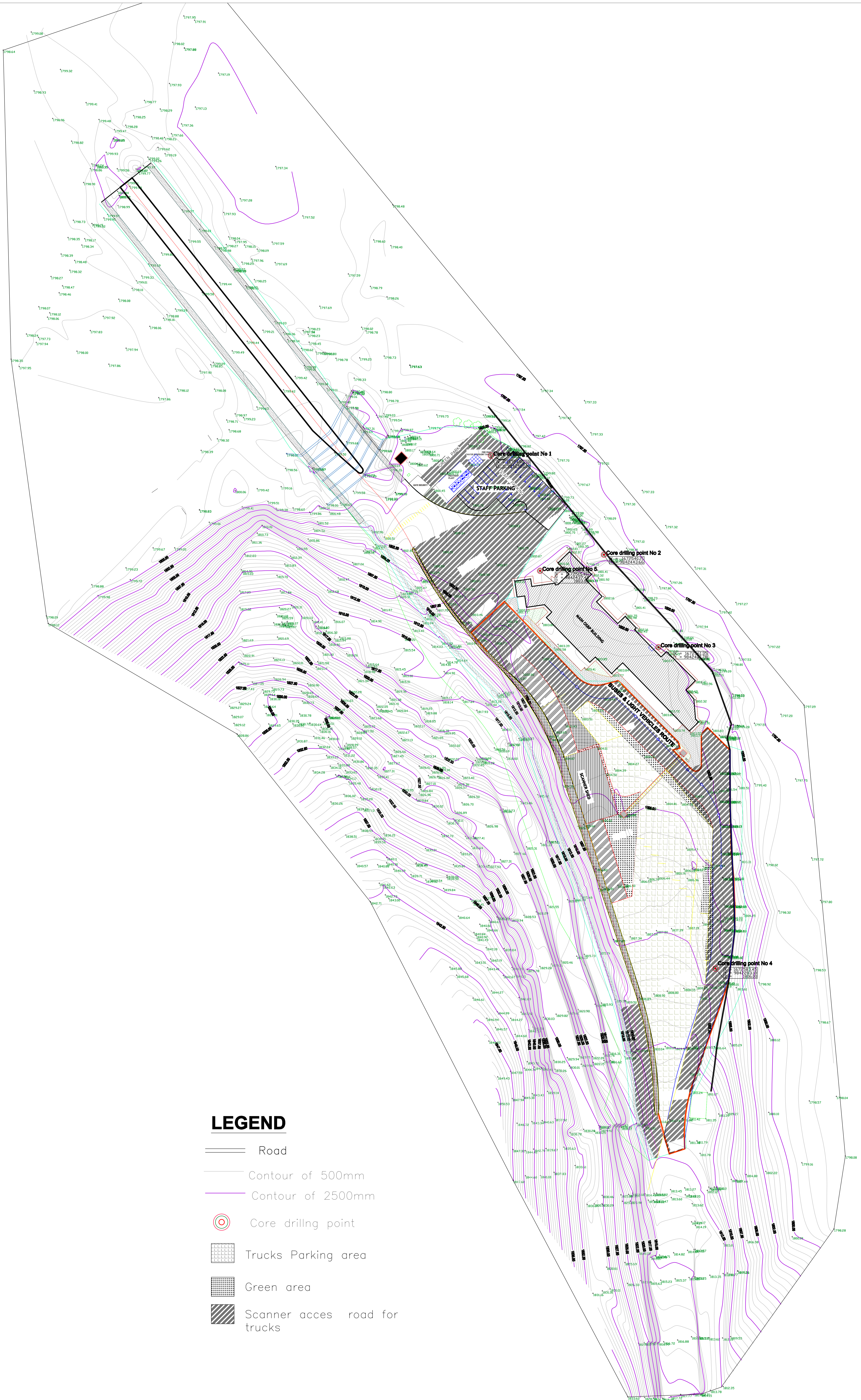
Figure 13 : Table showing Stratigraphic Layer of the drilled boreholes

Appendices 4 : Minimum geotechnical parameter for the subsoil and backfill to achieve the stability with the current proposal

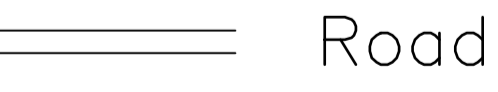








LEGEND

-  Road
-  Contour of 500mm
-  Contour of 2500mm
-  Core drilling point
-  Trucks Parking area
-  Green area
-  Scanner access road for trucks



LEGEND

-  Road
-  Contour of 500mm
-  Contour of 2500mm
-  Core drilling point
-  Trucks Parking area
-  Green area
-  Scanner access road for trucks

Soil	φ_k [°]	c_k [kN/m ²]	γ_k [kN/m ³]	Designation
	28.00	0.00	19.00	Fill material
	20.00	0.00	0.00	Drain layer HaTedrain WAS7
	20.00	8.00	16.00	backfill material
	4.00	6.00	15.00	Subsoil

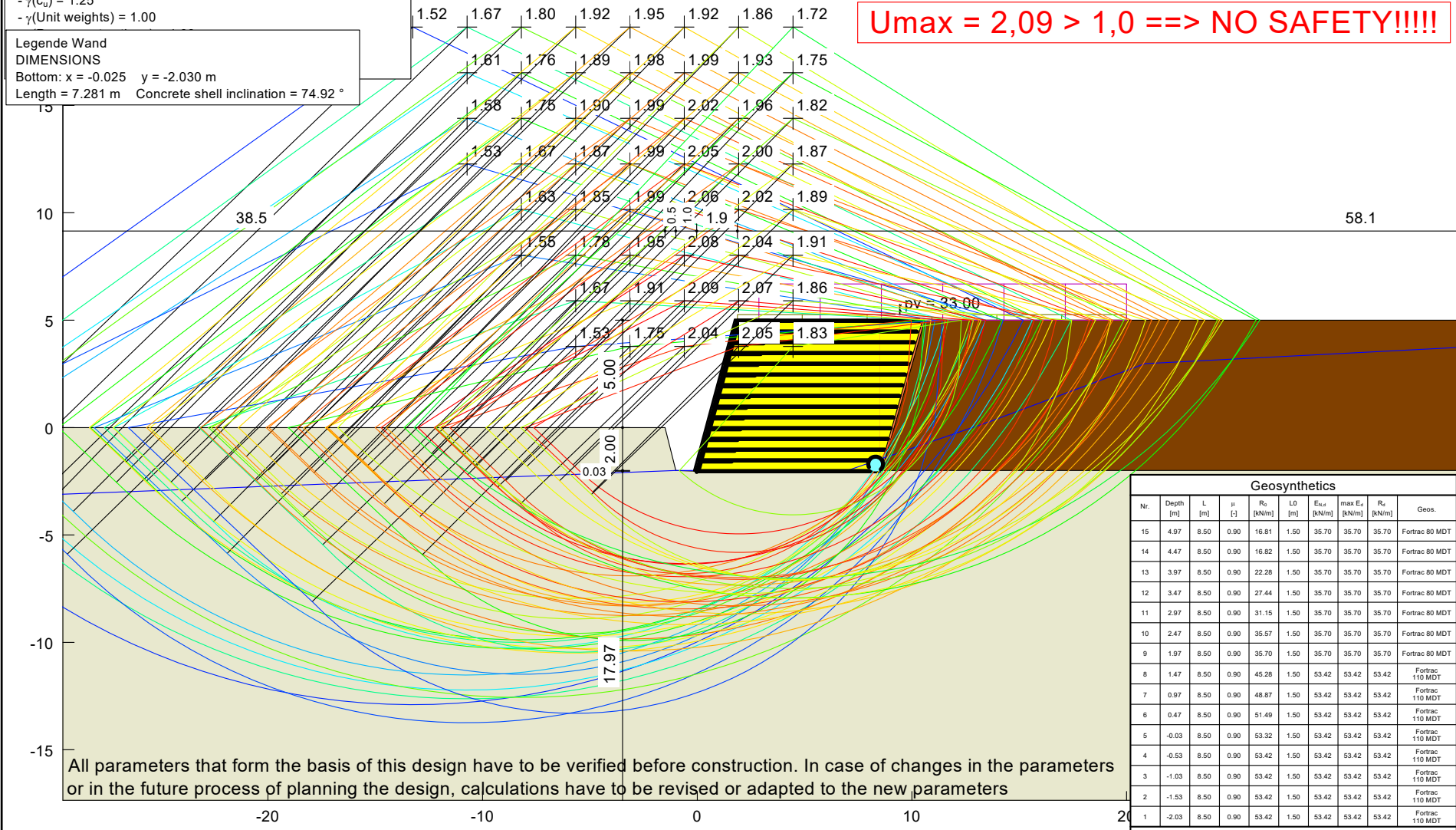
Minimum geotechnical parameter for the subsoil and backfill to achieve the stability with the current proposal

The safety of the building ist not part of this design

U_{max} = 2,09 > 1,0 ==> NO SAFETY!!!!

Standard: EC 7
 Friction reduction due to geosynthetics taken into consideration.
 Unfavourable slip circle:
 $\mu_{max} = 2.09$
 $x_m = -0.60$ m $y_m = 5.90$ m
 $R = 12.27$ m
 Nur Kreise mit: $1.50 < \mu < 2.09$
 — max $\mu = 2.09$
 — min $\mu = 1.50$

Legende Wand
 DIMENSIONS
 Bottom: $x = -0.025$ $y = -2.030$ m
 Length = 7.281 m Concrete shell inclination = 74.92 °



Geosynthetics									
Nr.	Depth [m]	L [m]	μ	R_{01} [kN/m]	L_0 [m]	$E_{v,1}$ [kN/m]	max $E_{v,1}$ [kN/m]	R_{02} [kN/m]	Geos.
15	4.97	8.50	0.90	16.81	1.50	35.70	35.70	35.70	Fortrac 80 MDT
14	4.47	8.50	0.90	16.82	1.50	35.70	35.70	35.70	Fortrac 80 MDT
13	3.97	8.50	0.90	22.28	1.50	35.70	35.70	35.70	Fortrac 80 MDT
12	3.47	8.50	0.90	27.44	1.50	35.70	35.70	35.70	Fortrac 80 MDT
11	2.97	8.50	0.90	31.15	1.50	35.70	35.70	35.70	Fortrac 80 MDT
10	2.47	8.50	0.90	35.57	1.50	35.70	35.70	35.70	Fortrac 80 MDT
9	1.97	8.50	0.90	35.70	1.50	35.70	35.70	35.70	Fortrac 80 MDT
8	1.47	8.50	0.90	45.28	1.50	53.42	53.42	53.42	Fortrac 110 MDT
7	0.97	8.50	0.90	48.87	1.50	53.42	53.42	53.42	Fortrac 110 MDT
6	0.47	8.50	0.90	51.49	1.50	53.42	53.42	53.42	Fortrac 110 MDT
5	-0.03	8.50	0.90	53.32	1.50	53.42	53.42	53.42	Fortrac 110 MDT
4	-0.53	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT
3	-1.03	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT
2	-1.53	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT
1	-2.03	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT

$E_{v,1}$ = Force from failure mechanism
 $\gamma_{soil} = 1.40$ (GEO-2); $\gamma_{fill} = 1.35$; $\gamma_{sub} = 1.50$

All parameters that form the basis of this design have to be verified before construction. In case of changes in the parameters or in the future process of planning the design, calculations have to be revised or adapted to the new parameters

Standard: EC 7
 Friction reduction due to geosynthetics taken into consideration.
 Unfavourable slip circle:
 $\mu_{max} = 0.73$
 $x_m = -0.60$ m $y_m = 8.02$ m
 $R = 13.47$ m
 Partial factors:
 $\gamma(\phi) = 1.25$
 $\gamma(c) = 1.25$
 $\gamma(c_u) = 1.25$
 $\gamma(\text{Unit weights}) = 1.00$
 $\gamma(\text{Permanent actions}) = 1.00$
 $\gamma(\text{Variable actions}) = 1.30$
 $\gamma(\text{Pull-out}) = 1.4000$

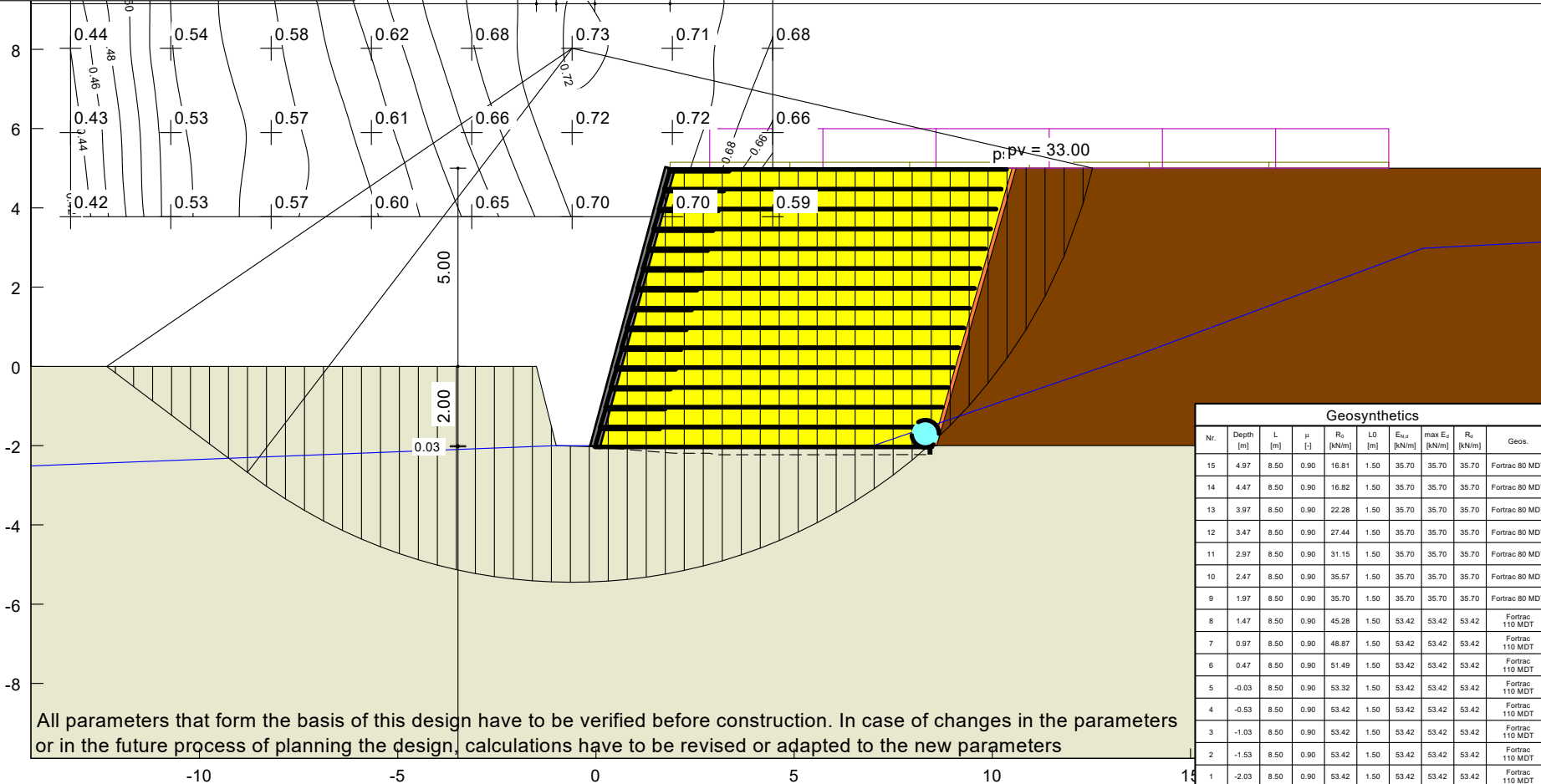
Soil	ϕ_k [°]	c_k [kN/m ²]	γ_k [kN/m ³]	Designation
Fill material	28.00	0.00	19.00	
Drain layer_HaTedral WAS7	20.00	0.00	0.00	
backfill material	20.00	15.00	16.00	
Subsoil	20.00	15.00	15.00	

Minimum geotechnical parameter for the subsoil and backfill to achieve the stability with the current proposal

The safety of the building ist not part of this design

$U_{max} = 0,79 < 1,0 ==> \text{SAFETY}$

Legende Wand
 DIMENSIONS
 Bottom: $x = -0.025$ $y = -2.030$ m
 Length = 7.281 m Concrete shell inclination = 74.92 °



Geosynthetics									
Nr.	Depth [m]	L [m]	μ [-]	R_0 [kN/m]	L_0 [m]	$E_{v,0}$ [kN/m]	$\max E_v$ [kN/m]	R_v [kN/m]	Geos.
15	4.97	8.50	0.90	16.81	1.50	35.70	35.70	35.70	Fortrac 80 MDT
14	4.47	8.50	0.90	16.82	1.50	35.70	35.70	35.70	Fortrac 80 MDT
13	3.97	8.50	0.90	22.28	1.50	35.70	35.70	35.70	Fortrac 80 MDT
12	3.47	8.50	0.90	27.44	1.50	35.70	35.70	35.70	Fortrac 80 MDT
11	2.97	8.50	0.90	31.15	1.50	35.70	35.70	35.70	Fortrac 80 MDT
10	2.47	8.50	0.90	35.57	1.50	35.70	35.70	35.70	Fortrac 80 MDT
9	1.97	8.50	0.90	35.70	1.50	35.70	35.70	35.70	Fortrac 80 MDT
8	1.47	8.50	0.90	45.28	1.50	53.42	53.42	53.42	Fortrac 110 MDT
7	0.97	8.50	0.90	48.87	1.50	53.42	53.42	53.42	Fortrac 110 MDT
6	0.47	8.50	0.90	51.49	1.50	53.42	53.42	53.42	Fortrac 110 MDT
5	-0.03	8.50	0.90	53.32	1.50	53.42	53.42	53.42	Fortrac 110 MDT
4	-0.53	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT
3	-1.03	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT
2	-1.53	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT
1	-2.03	8.50	0.90	53.42	1.50	53.42	53.42	53.42	Fortrac 110 MDT

$E_{v,0}$ = Force from failure mechanism
 $\gamma_{red} = 1.40$ (GEO-2); $\gamma_{red} = 1.35$ ($\gamma_{red} = 1.50$)

All parameters that form the basis of this design have to be verified before construction. In case of changes in the parameters or in the future process of planning the design, calculations have to be revised or adapted to the new parameters

MUR DE SOUTÈNEMENT EN SOL RENFORCE (MSSR)
 Geotechnical Study To Ascertain Longtime Solution At
 Gatuna OSBP Structures Located Close to the Swamp

STABILITY ANALYSIS : Road 1 m < H < 7 m; Facing: Muralax, Nature S, Blocs

STATUS: PRE_DESIGN PERMANENT SITUATION / Bishop

Proj. No: 2014218-1E-POB-LSA

Enclosure: A.1

HUESKER
 Ideen. Ingenieure. Innovationen.

STABILITY ANALYSIS : Road 1 m < H < 7 m; Facing: Murallex, Nature S, Blocs
 STATUS: PRE_DESIGN PERMANENT SITUATION / Blocs Sliding,

Proj. No.: 2014218-1E-POB-LSA

Enclosure: A.2

Soil	φ_k [°]	c_k [kN/m ²]	γ_k [kN/m ³]	Designation
Fill material	28.00	0.00	19.00	
Drain layer_HaTedrain WAS7	20.00	0.00	0.00	
backfill material	20.00	15.00	16.00	
Subsoil	20.00	15.00	15.00	

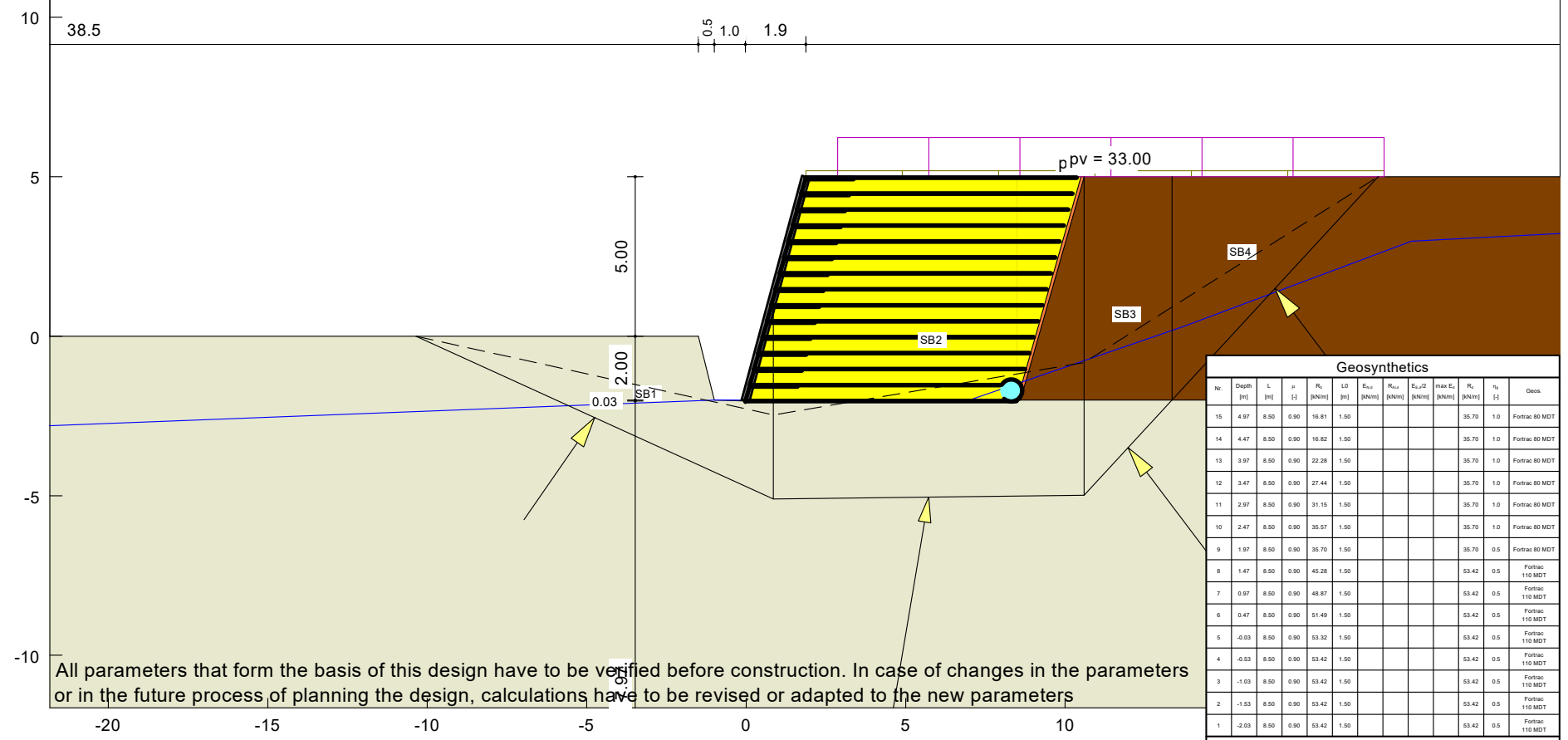
Minimum geotechnical parameter for the subsoil and backfill to achieve the stability with the current proposal

The safety of the building ist not part of this design

Umax = 0,62 < 1,0 ==> SAFETY

Standard: EC 7
 Friction reduction due to geosynthetics taken into consideration.
 Partial factors:
 - $\gamma(\varphi')$ = 1.25
 - $\gamma(c')$ = 1.25
 - $\gamma(c_u)$ = 1.25
 - γ (Unit weights) = 1.00
 - γ (Permanent actions) = 1.00
 - γ (Variable actions) = 1.30
 - γ (Pull-out) = 1.4000
 Slip body no. 2: $\mu = 0.62$
 With shear strength in the slice sides

Legende Wand
 DIMENSIONS
 Bottom: x = -0.025 y = -2.030 m
 Length = 7.281 m Concrete shell inclination = 74.92 °



All parameters that form the basis of this design have to be verified before construction. In case of changes in the parameters or in the future process of planning the design, calculations have to be revised or adapted to the new parameters

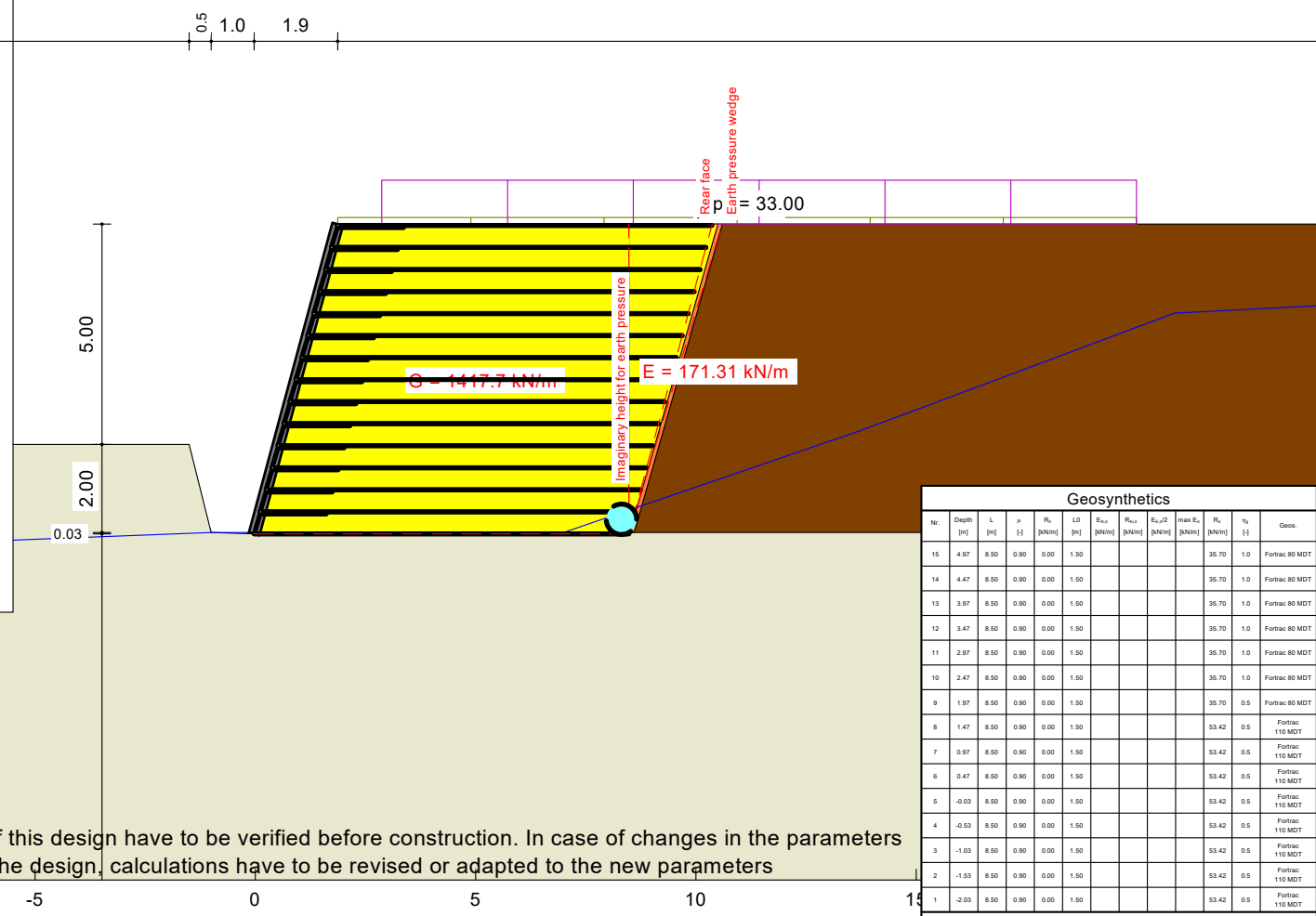
$E_{s,u}$ = Force from failure mechanism
 $R_{s,u}$ = Outer skin pull-out resistance
 $E_{s,z}$ = Earth pressure on outer skin
 $E_{s,z/2}$ because of fold-back
 η_1 = Calibration factor $E_{s,u}$
 Wall friction $\mu = 0.620$ (Outer skin)
 $\gamma_{soil} = 1.40$ (GEO-2); $\gamma_u = 1.35$; $\gamma_s = 1.50$

Standard: EC 7
 Friction reduction due to geosynthetics taken into consideration.
 Partial factors:
 - $\gamma(\phi')$ = 1.25
 - $\gamma(c')$ = 1.25
 - $\gamma(c_u)$ = 1.25
 - γ (Unit weights) = 1.00
 - γ (Permanent actions) = 1.00
 - γ (Variable actions) = 1.30
 - γ (Pull-out) = 1.4000

Soil	ϕ_k [°]	c_k [kN/m ²]	γ_k [kN/m ³]	Designation
Fill material	28.00	0.00	19.00	
Drain layer_HaTedrain WAS7	20.00	0.00	0.00	
backfill material	20.00	15.00	16.00	
Subsoil	20.00	15.00	15.00	

Minimum geotechnical parameter for the subsoil and backfill to achieve the stability with the current proposal
 The safety of the building ist not part of this design

Legende Wand
 DIMENSIONS
 Bottom: x = -0.025 y = -2.030 m
 Length = 7.281 m Concrete shell inclination = 74.92 °
 BASIC DATA
 $H_{k,g+q} / H_{k,g}$ = Earth pressure = 171.31 / 114.89 kN/m
 Batter of rear face = 74.92 °
 Wall friction angle delta/friction angle phi = 0.660
 EP reduction due to rear wall face angle and delta = 0.624
 $V_{k,g+q} / V_{k,g}$ = 1417.69 / 1170.18 kN/m
 Moments around mid-point of wall base:
 $M_{k,g+q} / M_{k,g}$ (from H-forces) = 401.44 / 269.23 kN·m/m
 $M_{k,g+q} / M_{k,g}$ (from V-forces) = -1732.88 / -1140.22 kN·m/m
 b = 8.50 m
 ϕ_k = 20.0 °
 Partial factors (GEO-2):
 γ_g (Permanent actions) = 1.35
 γ_q (Variable actions) = 1.50
 SLIDING
 γ (sliding) = 1.10
 μ_{g+q} / μ_g (sliding) = 0.57 / 0.45
 ϕ_k = 18.1 ° (Geosynthetics in slip surface)
 OVERTURNING
 eccentricity e_{g+q} / e_g = -0.939 / -0.744 m
 Allow e_{g+q} = 2.833 = b/3 / Allow e_g = 1.417 m = b/6
 Overturning safety γ_{g+q} verified
 Overturning safety γ_g verified
 BEARING CAP. for level ground
 c_k = 15.0 kN/m²
 $\gamma_{2,k}$ = 15.0 kN/m³
 γ (Bearing capacity) = 1.40
 μ_{g+q} / μ_g (Bearing capacity) = 0.93 / 0.71



Geosynthetics												
Nr.	Depth [m]	L [m]	μ [°]	R_u [kN/m]	L_0 [m]	$E_{s,1}$ [kN/m]	$R_{u,1}$ [kN/m]	$E_{s,2}$ [kN/m]	$\max E_s$ [kN/m]	$R_{u,2}$ [kN/m]	η_1 [%]	Geos.
15	4.97	8.50	0.90	0.00	1.50					35.70	1.0	Fortrac 80 MDT
14	4.47	8.50	0.90	0.00	1.50					35.70	1.0	Fortrac 80 MDT
13	3.97	8.50	0.90	0.00	1.50					35.70	1.0	Fortrac 80 MDT
12	3.47	8.50	0.90	0.00	1.50					35.70	1.0	Fortrac 80 MDT
11	2.97	8.50	0.90	0.00	1.50					35.70	1.0	Fortrac 80 MDT
10	2.47	8.50	0.90	0.00	1.50					35.70	1.0	Fortrac 80 MDT
9	1.97	8.50	0.90	0.00	1.50					35.70	0.5	Fortrac 80 MDT
8	1.47	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
7	0.97	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
6	0.47	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
5	-0.03	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
4	-0.53	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
3	-1.03	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
2	-1.53	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT
1	-2.03	8.50	0.90	0.00	1.50					53.42	0.5	Fortrac 110 MDT

$E_{s,1}$ = Force from failure mechanism
 $R_{u,1}$ = Outer skin pull-out resistance
 $E_{s,2}$ = Earth pressure on outer skin
 $R_{u,2}$ because of fold-back
 η_1 = Calibration factor $E_{s,1}$
 Wall friction $\delta = 0.660$ (Outer skin)
 $\gamma_{2,k} = 1.40$ (GEO-2); $\eta_1 = 1.35$; $\eta_2 = 1.50$

All parameters that form the basis of this design have to be verified before construction. In case of changes in the parameters or in the future process of planning the design, calculations have to be revised or adapted to the new parameters

Slope stability analysis to EC 7
using circular slip surfaces

Parameter list

phi [°] = friction angle
c [kN/m²] = cohesion
gamma [kN/m³] = unit weight
μ [-] = utilisation factor
xm,ym [m] = x and y values of slip circle centre-point
rad [m] = slip circle radius

Partial factors: (GEO-3)

- gam(phi) = 1.25
- gam(c') = 1.25
- gam(cu) = 1.25
- gam(Unit weights) = 1.00
- gam(Permanent actions) = 1.00
- gam(Variable actions) = 1.30
- gam(Pull-out) = 1.4000 (GEO-2)

Movement direction of slip body to the left

Coordinates of surface points

Nr.	x	y	Nr.	x	y	Nr.	x	y	Nr.	x	y	Nr.	x	y
[-]	[m]	[m]	[-]	[m]	[m]	[-]	[m]	[m]	[-]	[m]	[m]	[-]	[m]	[m]
1	-40.000	0.000	2	-1.500	0.000	3	-1.000	-2.000	4	-0.025	-2.030	5	1.870	5.000
6	60.000	5.000												

Characteristic soil properties

Soil	phi,k	c,k	gamma,k	Designation
[-]	[°]	[kN/m ²]	[kN/m ³]	
1	28.00	0.00	19.00	Fill material
2	20.00	0.00	0.00	Drain layer_HaTedrain WAS7
3	20.00	15.00	16.00	backfill material
4	20.00	15.00	15.00	Subsoil

Design soil properties

Soil	phi,d	c,d	gamma,d	Designation
[-]	[°]	[kN/m ²]	[kN/m ³]	
1	23.04	0.00	19.00	Fill material
2	16.23	0.00	0.00	Drain layer_HaTedrain WAS7
3	16.23	12.00	16.00	backfill material
4	16.23	12.00	15.00	Subsoil

Coordinates of layers and soil numbers

No.	x(left)	y(left)	x(right)	y(right)	Soil	no.
[-]	[m]	[m]	[m]	[m]		
1	8.500	-2.000	10.500	5.000	1	
2	8.500	-2.000	8.600	-2.000	2	
3	0.000	-2.000	8.500	-2.000	1	
4	8.600	-2.000	10.600	5.000	2	
5	7.100	-2.000	60.000	-2.000	3	
6	-40.000	-20.000	60.000	-20.000	4	

Coordinates of pwp polygon course

Nr.	x	y	Nr.	x	y	Nr.	x	y	Nr.	x	y	Nr.	x	y
[-]	[m]	[m]	[-]	[m]	[m]	[-]	[m]	[m]	[-]	[m]	[m]	[-]	[m]	[m]
1	-40.000	-3.504	2	-1.000	-2.000	3	7.000	-2.000	4	13.707	0.304	5	20.860	2.979
6	60.000	5.000												

Permanent loads

No.	Value(left)	Value(right)	x(left)	x(right)	y
[-]	[kN/m ²]	[kN/m ²]	[m]	[m]	[m]
1	5.00	5.00	1.87	20.00	5.00

Live loads

No.	Value(left)	Value(right)	x(left)	x(right)	y
[-]	[kN/m ²]	[kN/m ²]	[m]	[m]	[m]
1	33.00	33.00	2.87	20.00	5.00

Geosynthetics

Friction reduction due to geosynthetics taken into consideration.

Adhesive force f calculated with:

$$f = \mu \cdot \tan(\varphi) \cdot \sigma'$$

 μ [-] = reduction factor for friction between ground and geosynthetics σ' [kN/m²] = effektive Spannung

R0 [kN/m] = design force at connection

R,d [kN/m] = acceptable design force

Influence of live loads considered

Top fold-back [m]: 0.200

Nr.	x1	y1	x2	y2	μ	L0	R0	R,d	Typ
[-]	[m]	[m]	[m]	[m]	[-]	[m]	[kN/m]	[kN/m]	[-]
1	-0.02	-2.03	8.48	-2.03	0.900	1.50	53.42	53.42	Fortrac 110 MDT
2	0.11	-1.53	8.61	-1.53	0.900	1.50	53.42	53.42	Fortrac 110 MDT
3	0.25	-1.03	8.75	-1.03	0.900	1.50	53.42	53.42	Fortrac 110 MDT
4	0.38	-0.53	8.88	-0.53	0.900	1.50	53.42	53.42	Fortrac 110 MDT
5	0.51	-0.03	9.01	-0.03	0.900	1.50	53.32	53.42	Fortrac 110 MDT
6	0.65	0.47	9.15	0.47	0.900	1.50	51.49	53.42	Fortrac 110 MDT
7	0.78	0.97	9.28	0.97	0.900	1.50	48.87	53.42	Fortrac 110 MDT
8	0.92	1.47	9.42	1.47	0.900	1.50	45.28	53.42	Fortrac 110 MDT
9	1.05	1.97	9.55	1.97	0.900	1.50	35.70	35.70	Fortrac 80 MDT
10	1.19	2.47	9.69	2.47	0.900	1.50	35.57	35.70	Fortrac 80 MDT
11	1.32	2.97	9.82	2.97	0.900	1.50	31.15	35.70	Fortrac 80 MDT
12	1.46	3.47	9.96	3.47	0.900	1.50	27.44	35.70	Fortrac 80 MDT
13	1.59	3.97	10.09	3.97	0.900	1.50	22.28	35.70	Fortrac 80 MDT
14	1.73	4.47	10.23	4.47	0.900	1.50	16.82	35.70	Fortrac 80 MDT
15	1.86	4.97	10.36	4.97	0.900	1.50	16.81	35.70	Fortrac 80 MDT

Water level in front of slope left [m] = -2.10

Water level in front of slope right [m] = 0.00

Gamma water [kN/m³] = 10.000

Calculation with consideration of passive earth pressure wedge

Wall

Dimensions

Bottom: x = -0.025 y = -2.030 m

Length = 7.281 m Inclination = 74.92 °

Results

Search grid

Type of search radius

Start and end radius

x/y (start): 3.8436 2.8277

x/y (end): 8.2173 -14.8658

No. of radii = 40

No	xm	ym	Radius	Slices	μ	Numerat.	Denomin.	M(Ti)	M(R)	M(Gi)	M(S)
[-]	[m]	[m]	[m]	[-]	[-]	[kN*m/m]	[kN*m/m]	[kN*m/m]	[kN*m/m]	[kN*m/m]	[kN*m/m]
1	-13.2535	18.6452	24.5298	50	0.5848	10606.753	18138.674	18138.7	0.0	10606.8	0.0
2	-13.2535	16.5211	22.7110	51	0.5628	8659.783	15385.685	15385.7	0.0	8659.8	0.0
3	-13.2535	14.3971	21.0350	51	0.5440	7034.967	12932.145	12932.1	0.0	7035.0	0.0
4	-13.2535	12.2730	25.5616	52	0.4895	28768.731	58769.041	58769.0	0.0	28768.7	0.0
5	-13.2535	10.1489	23.9863	53	0.4570	25880.095	56630.134	56630.1	0.0	25880.1	0.0
6	-13.2535	8.0248	22.2619	55	0.4403	22556.964	51231.691	51231.7	0.0	22557.0	0.0
7	-13.2535	5.9007	20.8084	57	0.4265	19123.486	44833.281	44833.3	0.0	19123.5	0.0
8	-13.2535	3.7767	19.1029	60	0.4171	15219.262	36486.759	36486.8	0.0	15219.3	0.0
9	-10.7230	18.6452	23.2020	50	0.5881	11011.377	18724.357	18724.4	0.0	11011.4	0.0
10	-10.7230	16.5211	21.2424	50	0.5700	8961.237	15720.984	15721.0	0.0	8961.2	0.0
11	-10.7230	14.3971	19.4148	51	0.5509	7250.109	13159.620	13159.6	0.0	7250.1	0.0
12	-10.7230	12.2730	27.1944	51	0.5475	38522.813	70366.777	70366.8	0.0	38522.8	0.0
13	-10.7230	10.1489	25.7238	52	0.5399	35458.345	65680.469	65680.5	0.0	35458.3	0.0
14	-10.7230	8.0248	24.0127	54	0.5423	31890.022	58800.797	58800.8	0.0	31890.0	0.0
15	-10.7230	5.9007	22.1579	56	0.5306	26902.132	50700.792	50700.8	0.0	26902.1	0.0
16	-10.7230	3.7767	20.5867	59	0.5304	23197.384	43732.289	43732.3	0.0	23197.4	0.0
17	-8.1925	18.6452	26.8510	50	0.5813	27346.525	47042.439	47042.4	0.0	27346.5	0.0

18	-8.1925	16.5211	25.1059	50	0.5851	25494.894	43574.635	43574.6	0.0	25494.9	0.0
19	-8.1925	14.3971	23.4369	50	0.5877	23780.552	40465.360	40465.4	0.0	23780.6	0.0
20	-8.1925	12.2730	22.2759	50	0.5858	23380.026	39908.538	39908.5	0.0	23380.0	0.0
21	-8.1925	10.1489	21.6067	51	0.5807	24082.508	41474.065	41474.1	0.0	24082.5	0.0
22	-8.1925	8.0248	21.3903	52	0.5762	25708.071	44613.292	44613.3	0.0	25708.1	0.0
23	-8.1925	5.9007	23.6585	54	0.5680	34710.091	61108.996	61109.0	0.0	34710.1	0.0
24	-8.1925	3.7767	21.9644	58	0.5737	30117.913	52497.846	52497.8	0.0	30117.9	0.0
25	-5.6620	18.6452	25.1356	50	0.6218	23645.548	38027.982	38028.0	0.0	23645.5	0.0
26	-5.6620	16.5211	23.2879	50	0.6298	21889.047	34755.639	34755.6	0.0	21889.0	0.0
27	-5.6620	14.3971	21.9391	50	0.6347	21448.380	33795.206	33795.2	0.0	21448.4	0.0
28	-5.6620	12.2730	20.2330	50	0.6374	19607.941	30762.470	30762.5	0.0	19607.9	0.0
29	-5.6620	10.1489	18.6418	50	0.6388	17842.197	27930.338	27930.3	0.0	17842.2	0.0
30	-5.6620	8.0248	17.6065	51	0.6209	17027.196	27425.336	27425.3	0.0	17027.2	0.0
31	-5.6620	5.9007	16.3598	52	0.6076	15322.183	25216.474	25216.5	0.0	15322.2	0.0
32	-5.6620	3.7767	23.2418	56	0.5965	36386.005	60994.748	60994.7	0.0	36386.0	0.0
33	-3.1316	18.6452	20.9059	50	0.6528	12424.894	19032.820	19032.8	0.0	12424.9	0.0
34	-3.1316	16.5211	22.1206	50	0.6639	20010.601	30142.020	30142.0	0.0	20010.6	0.0
35	-3.1316	14.3971	20.2133	50	0.6777	18164.899	26803.141	26803.1	0.0	18164.9	0.0
36	-3.1316	12.2730	18.3696	50	0.6900	16323.583	23658.255	23658.3	0.0	16323.6	0.0
37	-3.1316	10.1489	17.0547	50	0.6843	15515.469	22672.848	22672.8	0.0	15515.5	0.0
38	-3.1316	8.0248	15.4389	50	0.6807	13694.016	20118.970	20119.0	0.0	13694.0	0.0
39	-3.1316	5.9007	14.0394	51	0.6645	11963.129	18001.924	18001.9	0.0	11963.1	0.0
40	-3.1316	3.7767	12.9537	53	0.6450	10408.390	16137.015	16137.0	0.0	10408.4	0.0
41	-0.6011	18.6452	20.5300	50	0.6590	11827.474	17946.460	17946.5	0.0	11827.5	0.0
42	-0.6011	16.5211	20.7686	50	0.6716	16585.111	24695.735	24695.7	0.0	16585.1	0.0
43	-0.6011	14.3971	18.7529	50	0.6969	14887.485	21361.592	21361.6	0.0	14887.5	0.0
44	-0.6011	12.2730	17.2251	50	0.7038	14187.433	20158.603	20158.6	0.0	14187.4	0.0
45	-0.6011	10.1489	15.2993	50	0.7214	12488.359	17311.306	17311.3	0.0	12488.4	0.0
46	-0.6011	8.0248	13.4730	50	0.7255	10767.348	14841.858	14841.9	0.0	10767.3	0.0
47	-0.6011	5.9007	11.8377	50	0.7179	9021.447	12567.281	12567.3	0.0	9021.4	0.0
48	-0.6011	3.7767	10.5741	50	0.7006	7580.157	10819.734	10819.7	0.0	7580.2	0.0
49	1.9294	18.6452	20.4736	50	0.6421	11706.535	18231.590	18231.6	0.0	11706.5	0.0
50	1.9294	16.5211	18.3726	50	0.6579	10346.270	15725.521	15725.5	0.0	10346.3	0.0
51	1.9294	14.3971	17.6430	50	0.6689	12204.263	18245.543	18245.5	0.0	12204.3	0.0
52	1.9294	12.2730	16.0144	50	0.6852	11472.731	16744.840	16744.8	0.0	11472.7	0.0
53	1.9294	10.1489	13.9463	50	0.7069	9736.016	13771.884	13771.9	0.0	9736.0	0.0
54	1.9294	8.0248	12.3635	50	0.7129	8711.267	12219.626	12219.6	0.0	8711.3	0.0
55	1.9294	5.9007	10.3994	50	0.7176	6860.752	9560.070	9560.1	0.0	6860.8	0.0
56	1.9294	3.7767	8.7132	50	0.7032	5214.066	7414.749	7414.7	0.0	5214.1	0.0
57	4.4599	18.6452	21.1970	50	0.6292	12895.464	20495.441	20495.4	0.0	12895.5	0.0
58	4.4599	16.5211	19.0784	50	0.6571	11788.076	17939.527	17939.5	0.0	11788.1	0.0
59	4.4599	14.3971	16.9610	50	0.6648	10192.507	15331.594	15331.6	0.0	10192.5	0.0
60	4.4599	12.2730	14.8451	50	0.6697	8590.484	12826.782	12826.8	0.0	8590.5	0.0
61	4.4599	10.1489	13.1803	50	0.6904	7923.854	11476.907	11476.9	0.0	7923.9	0.0
62	4.4599	8.0248	11.0717	50	0.6785	6128.461	9032.710	9032.7	0.0	6128.5	0.0
63	4.4599	5.9007	9.4236	51	0.6569	4959.078	7549.572	7549.6	0.0	4959.1	0.0
64	4.4599	3.7767	15.4402	72	0.5872	15372.716	26181.496	26181.5	0.0	15372.7	0.0

Unfavourable slip circle

No	xm	ym	Radius	Slices	μ	Numerat.	Denomin.	M(Ti)	M(R)	M(Gi)	M(S)
[-]	[m]	[m]	[m]	[-]	[-]	[kN*m/m]	[kN*m/m]	[kN*m/m]	[kN*m/m]	[kN*m/m]	[kN*m/m]
46	-0.6011	8.0248	13.4730	50	0.7255	10767.348	14841.858	14841.9	0.0	10767.3	0.0