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DEPARTEMENT DE GENIE CIVIL

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DEPARTMENT CIVIL, ARCHITECTURAL AND ENVIRONMANTAL ENGINEERING

DESIGN AND CALCULATION OF UNDERGROUND TANKS IN DIFFERENT TYPES OF SOILS: CONCEIVED CASE STUDY

A thesis submitted in partial fulfilment of the requirements for the degree of Master in Engineering (MEng) in Civil Engineering

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Academic Year: 2020-2021

DEDICATION

I dedicate this humble endeavour work to my mother, my shield **NYIRANKUNZURWANDA VALENTINE** for her support throughout this journey.

ACKNOWLEDGEMENT

First of all, I thank the Almighty God for giving the courage, strength and patience to complete this modest work. This work could not have been done without the cooperation and contribution of many people to whom I would like to express my deepest gratitude. I would like to thank:

- The President of the jury Prof. **NKENG George ELAMBO** for the honour given to me in accepting to be the president of the jury in my thesis presentation;
- The Examiner of the jury Prof **MBESSA Michel** for accepting to examine this work and bring forth his suggestions and observation to improve this work;
- My Supervisors Prof. **Carmelo MAJORANA**, Dr. Eng. **Guillaume Hervé POH'SIE,** Eng. **GUISEPPE Cardillo** and Eng. **Joseph FOTSO** for all the guidance, the advices and constructive criticisms they provided me during this thesis work;
- Prof. **NKENG George ELAMBO**, Director of the National Advanced School of Public Works, for his great contribution and devotion to the promotion and the success of the new academic status;
- Prof. **Carmelo MAJORANA** for his participation in initiating the engineering curricula in the National Advanced School of Public Works, Yaoundé;
- Dr. **BWEMBA Charles**, Vice-director of the National Advanced School of Public Works, for his perpetual help and advice during our sojourn in this school;
- Prof **MBESSA Michel**, Head of the Civil Engineering department for his disponibility, corrections and valuable advices;
- **All the teaching staff** of the **National Advanced School of Public Works** and the **University of Padua** for their high-quality lessons and the motivations they built and maintained within us;
- **My siblings**: UMUHIRE Evelyn and IRADUKUNDA Valentin for their encouragement and devotion;
- Eng. **MANGUELLE Thierry Claude**, Eng. **TETCHA Nkola Primael**, Eng. **ELOUNGUE Ndonga Paul Gerard** and Eng. **YOGNO Yamba Hycinthe** who supported and contributed to the accomplishment of this work;
- **All my classmates** and friends of the 7th batch of Master's in Engineering (MEng) of The National Advanced School of Public Works, who were a source of motivation and tenacity.

ABBREVIATIONS AND SYMBOLS

$A_{c,eff}$	Effective area of concrete surrounding reinforcement
A_{ct}	Area of concrete in tension just prior to onset of cracking
A_s	Cross-sectional area of reinforcement
$A_{s,min}$	Area of reinforcement to achieve controlled cracking
$A_{s,req}$	Area of reinforcement required
$A_{s,prov}$	Area of reinforcement provided
c	Nominal cover
d	Effective depth
E	Elastic modulus (Young's modulus of elasticity)
E_s	Design value of modulus of elasticity of reinforcing steel
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{ct}	Tensile strength
$f_{ct,eff}$	Mean tensile strength of concrete at the time of cracking
f_{ctm}	Mean tensile strength of concrete
f_{yk}	Characteristic yield strength of the reinforcement
g_k	Characteristic permanent action per unit length or area
h_o	Notional size
K	Allowance for creep
K_a	Active earth pressure coefficient
K_p	Passive earth pressure coefficient
l	Span length
M	Bending moment
q_k	Characteristic variable action per unit length or area
R	Restraint factor

$S_{r,max}$	Maximum spacing of cracks
SLS	Serviceability limit state
T_1	Difference between peak temperature of concrete during hydration and ambient temperature
T_2	Long-term drop-in temperature after concreting
ULS	Ultimate limit state
u	Pore water pressure
V_{Ed}	Design shear action
$V_{Rd,c}$	Shear resistance of section not reinforced for shear
w_k	Crack width
X	Depth to neutral axis
Z	Depth below ground surface
α_c	Coefficient of thermal expansion
α_e	Modular ratio
γ	Unit weight of soil
γ_F	Partial factor for actions
γ_G	Partial factor for permanent actions
$\gamma_{G,fav}$	Partial factor for permanent actions when beneficial
$\gamma_{G,unfav}$	Partial factor for permanent actions when adverse
γ_Q	Partial factor for variable actions
γ_w	Weight density of water
ε_{ca}	Autogenous shrinkage strain
ε_{cd}	Drying shrinkage strain
ε_{cm}	Mean strain in the concrete between cracks
ε_{cr}	Crack-inducing strain in concrete
ε_r	Restrained strain in concrete

ε_{sm}	Mean strain in reinforcement allowing for shrinkage
ρ	Ratio of total area of reinforcement to the gross section in tension
σ_s	Stress in the reinforcement
φ	Angle of internal friction of soil

ABSTRACT

The main objective of this thesis was firstly to provide a properly designed reinforced concrete underground tank to store water for firefighting purposes and, later, to perform a parametric study to observe the effect of different types of soils on the designed tank. The tank was rectangular in shape with three components, namely the top slab, walls, and base slab, and was able to withstand the applied loads without cracks that would permit leakage. The methodology used consisted of meeting the design requirements of a reinforced concrete structure, such as adequate strength, durability, and freedom from excessive cracking. The most critical aspect of design was ensuring that the underground tank maintained its stability under the applied (permanent and variable) actions. The results of the analysis showed that the underground tank was sized and reinforced to retain water without leakage (that is, satisfying the serviceability limit state) and then for strength (the ultimate limit state). This was achieved by providing proper distribution of reinforcement, adequate spacing, and crack width calculations that did not exceed 0.2 mm. For instance, with sand soil, on the top slab, 4 HA12 per unit width at 250 mm spacing in the short span and 3 HA16 per unit width at 300 mm in the long span were needed. 14 HA16 per unit width at 75 mm spacing in the short span and 10 HA16 per unit width at 100 mm in the long span on walls were needed. On the base slab, 10 HA16 per unit width at 100 mm spacing in the short span and 7 HA20 per unit width at 150 mm in the long span were needed. In addition, the parametric study results indicated that increasing the unit weight of soil would result in a linear increase in the active earth pressure coefficient, implying an increase in the maximum solicitations acting on the underground tank and the adequate reinforcement required. Increasing the angle of internal friction of the soil would result in a nonlinear reduction in the active earth pressure coefficient, implying a reduction in the maximum solicitations acting on the underground tank and the need for adequate reinforcement. The interpretation of the results obtained during the parametric study gave us a better understanding of the need to incorporate soil parameters, notably unit weight of soil and angle of internal friction, when designing underground tanks.

Keywords: Underground water tank, Reinforced concrete, Limit state design, Cracking.

RESUME

L'objectif principal de cette thèse était d'abord de fournir un réservoir enterré en béton armé correctement conçu pour stocker de l'eau pour la lutte contre les incendies et, ensuite, de réaliser une étude paramétrique pour observer l'effet de différents types de sols sur le réservoir conçu. Le réservoir était de forme rectangulaire avec trois composants, à savoir la dalle supérieure, les parois et la dalle de base, et était capable de résister aux charges appliquées sans fissures qui permettraient des fuites. La méthodologie utilisée consistait à répondre aux exigences de conception d'une structure en béton armé, telles que la résistance adéquate, la durabilité et l'absence de fissures excessives. L'aspect le plus critique de la conception était de s'assurer que le réservoir enterré conserve sa stabilité sous les actions appliquées (permanentes et variables). Les résultats de l'analyse ont montré que le réservoir enterré a été dimensionné et renforcé pour retenir l'eau sans fuite (c'est-à-dire pour satisfaire l'état limite de service) et ensuite pour la résistance (l'état limite ultime). Ce résultat a été obtenu grâce à une bonne répartition des armatures, un espacement adéquat et des calculs de largeur de fissure ne dépassant pas 0,2 mm. Par exemple, avec un sol sableux, sur la dalle supérieure, 4 HA12 par mètre d'espacement à 250 mm dans la courte portée et 3 HA16 par mètre d'espacement à 300 mm dans la longue portée étaient nécessaires. Sur les murs, 14 HA16 par mètre d'espacement à 75 mm dans la courte portée et 10 HA16 par mètre d'espacement à 100 mm dans la longue portée ont été nécessaires. Sur la dalle de base, 10 HA16 par mètre d'espacement à 100 mm dans la courte portée et 7 HA20 par mètre de largeur à 150 mm dans la longue portée ont été nécessaires. En outre, les résultats de l'étude paramétrique ont indiqué que l'augmentation du poids unitaire du sol entraînerait une augmentation linéaire du coefficient de pression active des terres, ce qui implique une augmentation de la pression maximale agissant sur le réservoir enterré et le renforcement adéquat requis. L'augmentation de l'angle de friction interne du sol entraînerait une réduction non linéaire du coefficient de pression active des terres, ce qui implique une réduction des sollicitations maximales agissant sur le réservoir enterré et la nécessité d'un renforcement adéquat. L'interprétation des résultats obtenus lors de l'étude paramétrique nous a permis de mieux comprendre la nécessité d'intégrer les paramètres du sol, notamment le poids unitaire du sol et l'angle de frottement interne, lors de la conception des réservoirs enterrés.

Mots clés : Réservoir d'eau enterré, béton armé, conception à l'état limite, fissuration.

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GENERAL INTRODUCTION

Tanks play an important role in storing different fluids such as water, petroleum products and hazardous materials. Tanks can be resting on ground, elevated or underground as well as be rectangular, circular or conical in shape. Depending on the stored fluid, the material in which the tank is made can be reinforced concrete, plastic and steel. Selecting the proper construction material depends on various criteria including required storage capacity, service life, structural performance and construction (Meier,2002). These different concepts have their own challenges and considerations especially in terms of space occupied, loading, analysis and design.

The main objective of this thesis is to provide a structurally sound underground tank that will not leak. The stored liquid is water, the material selected is reinforced concrete and the tank is rectangular in shape. Underground water tank is a structure built below the ground level that is used to provide storage of water for various uses such as drinking purposes, irrigation, firefighting purposes and agricultural. Its three basic components are the base slab, side walls, and roof slab. To prevent any leakage, all tanks are built as crack-free constructions. When compared to other structures, underground water tanks are subjected to a variety of pressures, the most common of which are horizontal or lateral loads caused by earth pressure and water pressure. At the bottom of the underground water tank, the side walls will be subjected to a greater load, which will drop linearly towards the top. The underground water tank is subjected to loads both inside and outside the tank, as well as a surcharge above ground level. As a result, the underground tank's roof slab should be able to withstand the surcharge.

To accomplish this work, it will be structured around three chapters. The first chapter will consist of the current state of knowledge on the classification of various water tank types, a general overview of different types of soils, and essential design methods for a liquid-retaining structure that is built below the ground level. The second chapter, entitled methodology, will define the conception of the case study through design assumptions based on existing standards of the rectangular reinforced concrete underground water tank. In addition, the presentation of analytical analysis and limit state design method of the case study will be explained. Finally, presentation and interpretation of the different analyses announced in the methodology will be made in chapter 3.

CHAPTER 1.LITERATURE REVIEW

Introduction

A water tank is a structure built to store water, the stored water is used in many applications such as irrigation agriculture, firefighting and domestic purpose. This chapter presents review on the classification of water tanks. In addition, it provides a general overview of soil through its definitions, composition, classification and properties. Furthermore, the shear strength of the soil is also mentioned. Moreover, a solely focus is made on what is required to provide a structurally sound tank.

1.1. Classification of water tank

The water tanks are classified into 3 types, which are mainly based on the materials, location and their shape.

1.1.1. Based on material

1.1.1.1. Plastic tank

Plastic water tanks have become increasingly popular in recent years. They are both strong and light. They are constructed of polyethylene, which is an ultra-violet resistant plastic. They are also the most affordable, easiest to install, and long-lasting. It is corrosion-resistant and resists rusting. Because they are manufactured in one piece, there are no joints to worry about. It is available in a variety of colors and can be recycled after use. The biggest issue with plastic tanks is that as the plastic degrades over time, it can release harmful compounds. This is especially dangerous if one is drinking from an outdated plastic tank.

Figure 1. 1. Plastic water tank(www.indiamart.com)

1.1.1.2. Steel tank

Steel water tanks are made of galvanized steel, which has a zinc coating that protects it from rust. Steel water storage tanks are strong and suitable for large storage requirements of thousands of liters of water. There is a chance that a steel storage tank will rust, drastically reducing its lifespan. However, there have been techniques developed to keep steel tanks from rusting and to make them last for 15 to 20 years. Steel can be galvanized or coated with corrosion-resistant polyethylene liners.

Figure 1. 2. Steel tank(www.cstindustries.com)

1.1.1.3. Fiber glass tank

Fiberglass refers to a strong, lightweight material that consists of thin fibers of glass that can be transformed into a woven layer or used as reinforcement. It is less brittle, versatile and has shown excellent strength, bendability and dimensional stability. As a result, fiber glass tank is extremely resistant to a wide range of chemicals, cold temperatures, and corrosion.

Figure 1. 3. Fiber glass tank(www.aljassra-fiberplast.com)

1.1.1.4. Concrete tank

Concrete is naturally durable and waterproof if properly constructed, making it an ideal material for water storage tanks, especially when large capacity is required. Concrete, on the other hand, is not invincible, and deterioration can occur due to a variety of factors, including contamination introduced by the water itself. In fact, moisture-borne contamination is a major factor in the corrosion of concrete reinforcing steel and the subsequent cracking that leads to tank leakage.

Figure 1. 4. Concrete tank(www.indiamart.com)

1.1.2. Based on place of construction

1.1.2.1. Resting on ground tank

This type of water tank is built directly on the ground. These tanks are in the residential buildings to store water, as a curing tank in the construction sites and in the agricultural area for irrigation purposes. The wall of these tanks is subjected to water pressure from inside and the base is subjected to the weight of water inside and soil reaction.

Figure 1. 5. Resting on ground tank(www.tuf-bar.com)

1.1.2.2. Overhead tank

Overhead water tank as its name stands for itself these tanks is built on a certain height from ground level. It is generally installed over the rooftop of any house, building, or apartment. Also, the tanks which are constructed over the column or steel structures for the general public water supply or for personal use are included. The location of the tank should be ideal to equalize water pressure in the distribution system. The pressure will not be equal all the time, it depends on the depth of water in the tank. The low levelled water yields less pressure meanwhile the full tank may provide too much pressure. The water pressure can be adjusted by providing stand pipes. The water tank is filled from ground level through pumping.

Figure 1. 6. Overhead tank(www.indiamart.com)

1.1.2.3. Underground tank

These tanks are built below the ground level. In most cases, underground tanks collect and store runoff from ground catchments such as open grasslands, home compounds and roads. They are generally used to store water for drinking water facility, waste water collection. They are mainly used in areas with a high point, or otherwise accompanied by a delivery pump. The walls of these tanks are subjected to water pressure from inside and earth pressure from outside. The base of the tank is subjected to water pressure from inside and soil reaction from underneath. The main advantages they offer is the low variation in their temperature of the water contained and the limitation of acts of vandalism (Hassan,2011).

Figure 1. 7. Underground tank(www.indiamart.com)

1.1.3. Based on shape

1.1.3.1. Conical water tank

This is a cone-shaped vessel used for storage of water. It has a wide variety of applications and is upside down to facilitate easy outflow of liquids and forbid contaminant accumulation. It is suitable for height staging as the tank ‘s hollow shaft can easily be built.

Figure 1. 8. Conical tank(www.indiamart.com)

1.1.3.2. Intze water tank

An Intze tank is circular in shape with a spherical top and conical dome at the bottom. In case of conical bottom tank, the inward forces coming from the conical slab counteract the outward forces coming from the bottom dome which results in less stress. It can be divided into two types based on the support. It can be a column rested water tank or a shaft rested water tank. Generally, column rested water tanks are preferred for easy calculation of loading condition.

Figure 1. 9. Intze water tank(www.centrallibrary.cit.ac.in)

1.1.3.3. Circular water tank

Circular water tank is the simplest form of water tank. Since circular water tanks have no corners, it will be made water tight easily. Circular tanks are usually good for very large storage capacities. The circular side walls are designed for large circumferential hoop tension and bending moment; the theory of thin cylinders is applied for the design of wall thickness and for calculation of maximum hoop tension. Moreover, the circular tanks may be designed either with flexible base connection with wall or with rigid connection between walls and base. Pooja and Pradeep (2016) affirmed that circular tank is the simplest form of water tank.

Figure 1. 10. Circular tank(www.paramvisions.com)

1.1.3.4. Rectangular water tank

The rectangular tanks are easy to construct. However, from the economical point of view Hassan (2011) precised that tank should be preferably square in plan and it is desirable that larger side should not be greater than twice the smaller side for rectangular tanks. Pooja and Pradeep (2016) add by saying that for a given capacity, perimeter is least for circular tank. Therefore, rectangular tank requires more materials for the same capacity as circular tank as indicated by Abba and Abdul Warith (2017). The walls of the tank are subjected to bending moments and shear forces resulting from triangular loads acting on them. The magnitude of the moment will depend on the length, width and height of the tank, and the conditions of support of the wall at the top and bottom edges.

Figure 1. 11. Rectangular tank(www.dreamstime.com)

1.2. Underground concrete water tanks

Underground water tanks are structures built below the ground level that serve as reservoirs for modest residential or commercial constructions. Tanks are very ductile, enabling to withstand seismic forces and varying water backfill. Tanks utilize material efficiently; steel in tension, concrete in compression. Underground water tanks have Low maintenance throughout the life as these are built with concrete, durable material that never corrodes and does not require coatings when in contact with water or the environment.

1.2.1. Basic components of the underground concrete water tanks

Underground tanks have three basic components namely the top slab, side walls, and base slab. In addition, it has an access opening mainly used for maintenance as shown in figure 1.12. These components are subjected to a variety of loads acting differently on each component.

1.2.1.1. Loads acting on the components of the underground tank

When compared to other structures, underground water tanks are subjected to a variety of pressures, the most common of which are horizontal or lateral loads caused by earth pressure and water pressure. At the bottom of the underground water tank, the side walls will be subjected to a greater load, which will drop linearly towards the top. The underground water tank is subjected to loads both inside and outside the tank, as well as a surcharge above ground level. As a result, the underground tank's roof slab should be able to withstand the surcharge.

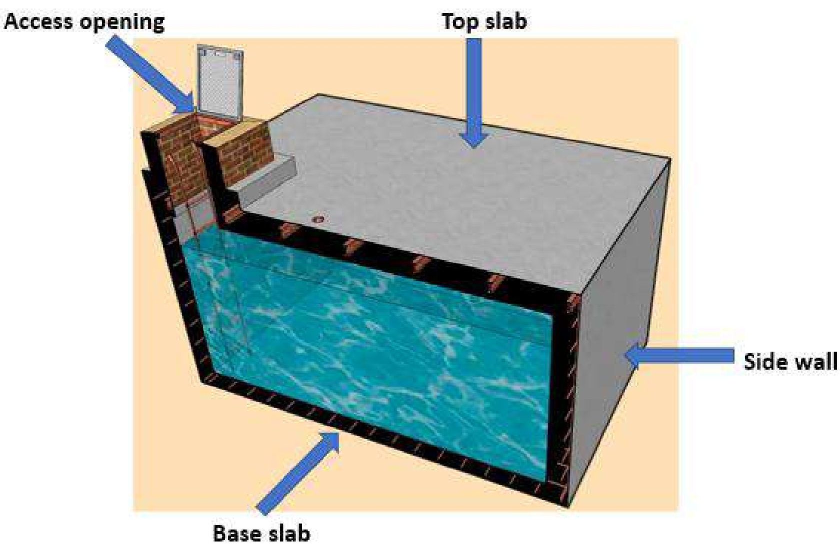

Figure 1. 12. Components of an underground water tank(www.dreamstime.com)

1.2.2. Advantages of concrete water tank

Concrete water tank is a traditional water storage system constructed from strong and long-lasting material, that is concrete. This tank is used to provide storage of water for various uses such as drinking purposes, irrigation, firefighting purposes and agricultural. Its advantages are discussed in the next subsections.

1.2.2.1. Affordable

Concrete is the most cost-effective and efficient material for tanks. It has a large global supply, making it more cost-effective when compared to water tanks built of other materials such as steel and fiberglass.

1.2.2.2. Maintains sanitary conditions

Concrete, if properly maintained, may protect the stored water from a variety of impurities such as germs and bacteria. The material also helps keep water cool all through the year, being a poor conductor of heat. It helps keep intact the water molecules. By the installation of a concrete tank on the property, one is ensuring that water is safe from any bacterial and algae growth.

1.2.2.3. Highly durable

One major reason why concrete tanks are installed in industrial plants, residential buildings and offices is because of its high durability. Such tank requires less maintenance and can function for nearly 50 years. The other tank systems in the market require constant repair and strict maintenance cycles, which involve lots of expenditure.

1.2.2.4. Healthy alternative

It is tough to deal with the copper poisoning emanating from tanks made of plastic or steel. This is due to the reaction between water and steel in the tank. But such issues can be avoided with concrete tanks. The reason is that lime from concrete dissolves in water gradually and is never found to be harmful. But if you drink water contaminated with dissolved copper from plastic or steel, it may result in health issues like headaches, gastric problems, and even liver cirrhosis.

1.2.2.5. Can withstand bushfire

In case a water tank is built near dense forests, it is vulnerable to bush fires. When compared with plastic tanks, concrete tanks may be regarded as the safest option. With regard to quality, in-ground concrete tanks are superior to metal tanks. In contrast, plastic tanks get deformed and burned quickly.

1.2.3. Types of underground tanks

There are many types of underground tank, categorized according to shape, size, capacity, lining material, construction and utilization. The most common types of tanks are given in the following sub-section.

1.2.3.1. Cistern

A cistern is a small underground reservoir with a capacity of around 10 to 500 m^3. The term is sometimes synonymous with underground tank. Cisterns are native water-harvesting systems commonly found in arid regions. They are typically used for water consumption by humans and livestock and are mostly found on or near homesteads. In many places they are dug into the rock or could be constructed as underground tanks lined with concrete. This system collects runoff water from catchment areas such as rooftops, residential complexes, rocky surfaces, roads or open spaces. Stills are sometimes needed to reduce sediment input. Because the water is stored underground, a lifting device such as a Pump, bucket and rope is used to bring water to the surface for use (Bancy mati).

Figure 1. 13. Cylindrical cistern for surface rain water harvesting system (Alemu seifu,2011)

1.2.3.2. Cylindrical underground tank

A cylindrical underground tank is made similar to a surface tank. However, it requires a lot of digging to achieve any appreciable storage volume. In general, cylindrical tanks are made of concrete or brick and have good hydraulic properties. They are easy to construct with local labour and construction material can be minimized compared to surface tanks (Bancy mati).

Figure 1. 14. Cylindrical tank partially buried (www.jkuat.ac.ke)

1.2.3.3. Rectangular underground tank

One of the easier ways to build underground tanks is in the shape of a rectangle. This shape combines good storage with simple design and construction features as builders use straight lines. The tank can be lined with geomembrane plastics, concrete, brick and other waterproof materials. Lined underground tanks have the advantage that they can be used on almost all types of soil. The design also makes it easy to roof or cover the tank with galvanized iron sheets, grass, polyethylene, wood or other materials. The tank is especially popular for runoff harvesting for agricultural purposes, especially for supplemental irrigation of small plots. The larger tanks are built with reinforced concrete. Some of these tanks are usually covered with a concrete slab which can also serve as a catchment area for rainwater harvesting (Bancy mati).

Figure 1. 15. Roofed rectangular underground tank (Bancy mati)

1.3. Soils

According to the subject of study, pedology, soil geology, soil biology, agrology, botany, geochemistry, ecology, and geotechnics, soil can be perceived in a variety of ways. Distinct scientists have provided many different definitions to soil over time to demonstrate the evolution of the modern concept of soil.

1.3.1. Definitions of soils

According to Johnson and DeGraff (1988), soil is a mass of solid particles formed by the physical and/or chemical disintegration of bedrock located in various thicknesses mantling the ground surface. The soil is any uncemented or weakly cemented aggregation of mineral particles created by the weathering of rocks, the void area between the particles containing water and/or air (Herrmann & Bucksch 2014). Carbonates or oxides deposited between the particles, as well as organic debris, can cause weak cementation. The material formed by in-situ weathering of rocks that remain in the same location of origin with little or no migration of individual soil particles is referred to as residual soil.

1.3.2. Composition of soils

Soils are a three-phase substance made of rock or mineral particles, water, and air in geotechnical engineering. As shown in Figure 1.1, the void in the soil is the space between mineral particles that contains water and air. Gravel and sand fractions (coarse particles or granular particles), silt or clay fractions (fine particles or cohesive) are found alone or mixed with organic components in an unlimited variety of compositions.

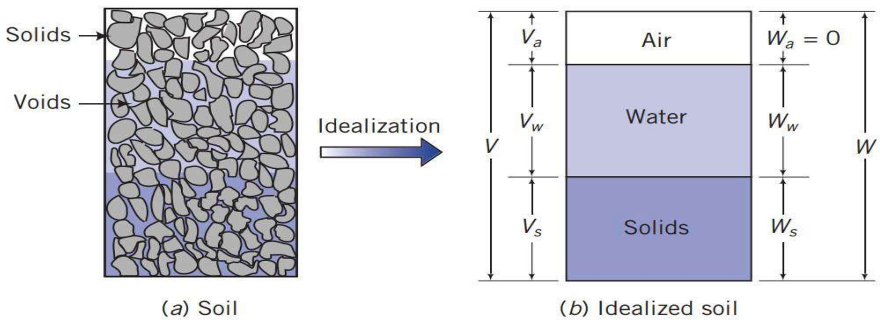

Figure 1. 16. Phase diagram of soil (Budhu, 2015)

1.3.3. Properties of soil

All soils contain mineral particles, organic matter, water, and air. The combination of these determines the soil's properties. Physical and mechanical properties of soil are discussed in the next subsection.

1.3.3.1. Physical properties

Physical properties play an important role in determining soil's suitability for geotechnical engineering and environmental uses. The next subsection will discuss on soil texture, soil structure, soil color and finally unit weight of soil.

a. Soil texture

Soil is made up of different-sized particles. Soil texture corresponds to the size of the particle that makes up the soil and depends on the proportion of sand, silt, clay-sized particles, and organic matter in the soil. Sand particles are the largest and clay particles the smallest. Soils are made up of different combinations of sand, silt, and clay particles. Soils that are a mixture of sand, silt, and clay are called loams.

b. Soil Structure

Soil structure describes the way the sand, silt, and clay particles are clumped together. Organic matter and soil organisms like earthworms and bacteria influence soil structure. Good quality soils are friable and have fine aggregates so the soil breaks up easily if you squeeze it. Poor soil structure has coarse, very firm clods or no structure at all.

c. Soil color

Soil color is influenced primarily by soil mineralogy and organic matter. Soil colors range from black to red to white. Soils high in iron are deep orange-brown to yellowish-brown. Soils that are high in organic matter are dark brown or black. Color can also tell us how a soil behaves, a soil that drains well is brightly colored and one that is often wet and soggy will have a mottled pattern of greys, reds, and yellows.

d. Unit weight of soil

The unit weight also known as the weight density of a soil is related to the mineral composition of a soil and to soil structure. It is defined as the dry weight of soil per unit volume of soil and is typically expressed as kilonewtons per cubic meter (KN/m^3). The typical values of unit weight of soils are given in Table 1.1, and can be used in the absence of reliable soil data.

Table 1. 1. Typical values of soil unit weight (Table.2.10; Reynolds and Steedman ,2005)

Unit weights of soils (and similar materials)						
Granular materials	Moist bulk weight γ_m kN/m³		Saturated bulk weight γ kN/m³		Cohesive soils	Weight kN/m³
	Loose	Dense	Loose	Dense		
Gravel	16.0	18.0	20.0	21.0	Peat (very variable)	12.0
Well graded sand and gravel	19.0	21.0	21.5	23.0	Organic clay	15.0
Coarse or medium sand	16.5	18.5	20.0	21.5	Soft clay	17.0
Well graded sand	18.0	21.0	20.5	22.5	Firm clay	18.0
Fine or silty sand	17.0	19.0	20.0	21.5	Stiff clay	19.0
Rock fill	15.0	17.5	19.5	21.0	Hard clay	20.0
Brick hardcore	13.0	17.5	16.5	19.0	Stiff or hard glacial	
Slag fill	12.0	15.0	18.0	20.0	clay	21.0
Ash fill	6.5	10.0	13.0	15.0		

1.3.3.2. Mechanical properties

The mechanical properties of soil are those that will characterise the response of soil to the application of different load actions. Among the properties which evaluate the response of a soil, it can be distinguished, failure criterion in soil and the shear strength of soil, discussed in the next subsection.

1.3.3.3. Failure criterion in soil

The characteristics at failure of a soil corresponds to the combination of the most unfavorable load actions to which the soil can withstand without failure. This is mostly related to cohesion and angle of internal friction.

a. Cohesion

Cohesion is the measure of the resistance due to intermolecular forces. This corresponds to the shear strength at a zero-shear stress resistance. Cohesion between soil particles comes from three major sources cementation, electrostatic and electromagnetic attraction, and primary valence bonding and adhesion. The adhesion due to cohesion is observed in fine-grained soil(clay) and partially saturated sand having a zero value in dry or saturated sand and normally consolidated clay. Examples of cohesive soils include sandy clay, silty clay, clayey silt, and organic clays.

b. Angle of internal friction

The angle of friction is a measure of the ability of a unit of soil to withstand a shear stress. This is also called the angle of shearing resistance. It is the angle measured between the normal force and resultant force, that is attained when failure just occurs in response to a shearing stress. It depends on a few factors, primarily grain size distribution, angularity and particle interlocking. An angular and coarse sand has a higher friction angle than a fine grained and well-rounded sand. Table 1.2 shows values of angle of internal friction for different types of soils. These values can be used in the absence of reliable soil data.

Table 1. 2. Typical values of angle of internal friction (Narayanan and Goodchild ,2012)

Soil type	Angle of internal friction(degrees)
Sand (rounded grains)	
Loose	$27-30$
Medium	$30-35$
Dense	$35-38$
Sand (Angular grains)	
Loose	$30-35$
Medium	$35-40$
Dense	$40-45$
Gravel with sand	$34-48$

1.3.3.4. Shear strength of soil

The term ‘‘strength of a soil’’ is defined as the ability of the soil to resist imposed forces and normally refers to the shearing strength or shear strength. The shear strength of soil is the maximum internal shear resistance to applied shearing forces. It is the level of shear stresses a material can resist without fracture. Shear stresses are forces applied tangentially along the face of the soil. The shear strength is a function of the normal stress and is measured in the laboratory from the direct shear and triaxial tests.

a. The Mohr-Coulomb criterion

The basic principles in the description of strength properties are the failure criterion and the effective stress principle. The Mohr-Coulomb criterion is widely used to define failure in geotechnical applications. The Mohr-Coulomb criterion assumes that failure is controlled by the maximum shear stress and that this failure shear stress depends on the normal stress. This can be represented by plotting Mohr's circle for states of stress at failure in terms of the maximum and minimum principal stresses. The Mohr-Coulomb failure line is the best straight line that touches these Mohr's circles (Figure 1.13). The Mohr-Coulomb shear strength equation of a saturated soil presented by Terzaghi (1936) is given by equation (1.1).

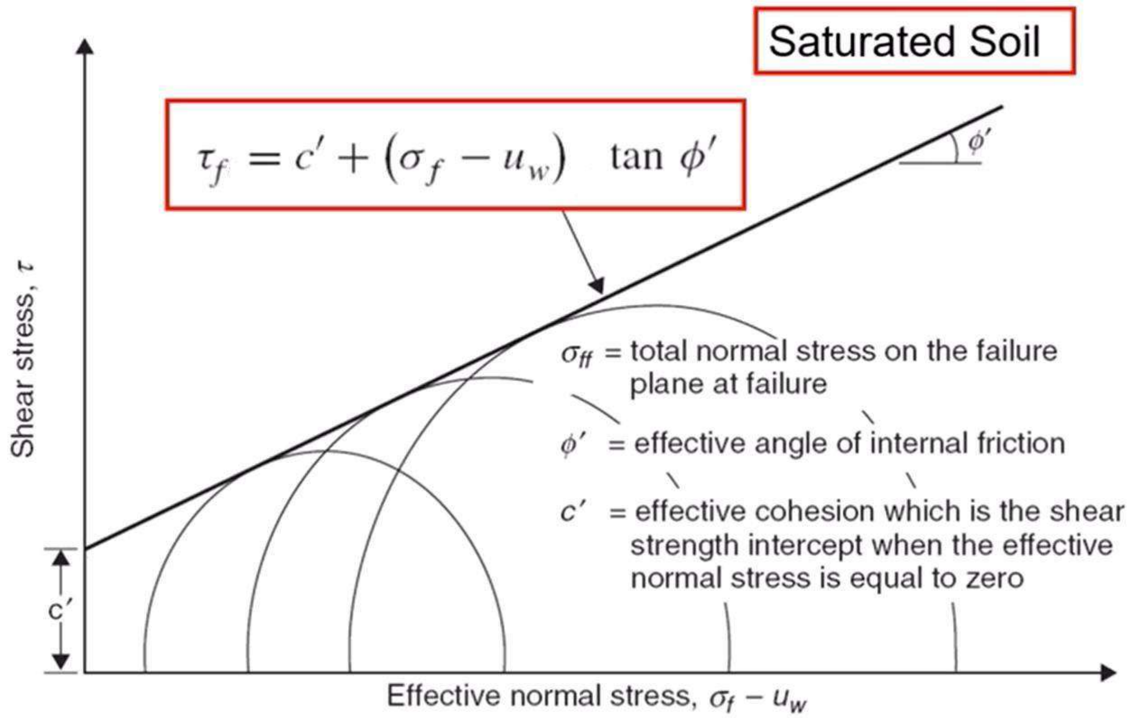

Figure 1. 17. Mohr-Coulomb failure envelope for saturated soil. (Fredlund et al., 1978)

$$\tau_f = c' + (\sigma_f - u_w) \tan \phi' \qquad (1.1)$$

Where:

-τ_f is the shear stress on failure at failure;

-c' is the effective cohesion;

-$(\sigma_f - u_w)$ is the effective stress on failure plane at failure;

- u_w is the pore water pressure on the failure plane at failure;

- $\phi'$is the effective angle of internal friction.

The Mohr-Coulomb criterion, unlike the Drucker-Prager criterion, assumes that failure is independent of the value of the intermediate principal stress. The failure of typical geotechnical materials generally includes some small dependence on the intermediate principal stress, but the Mohr-Coulomb model is generally considered to be sufficiently accurate for most applications. This failure model has vertices in the deviatoric stress plane (Figure 1.14).

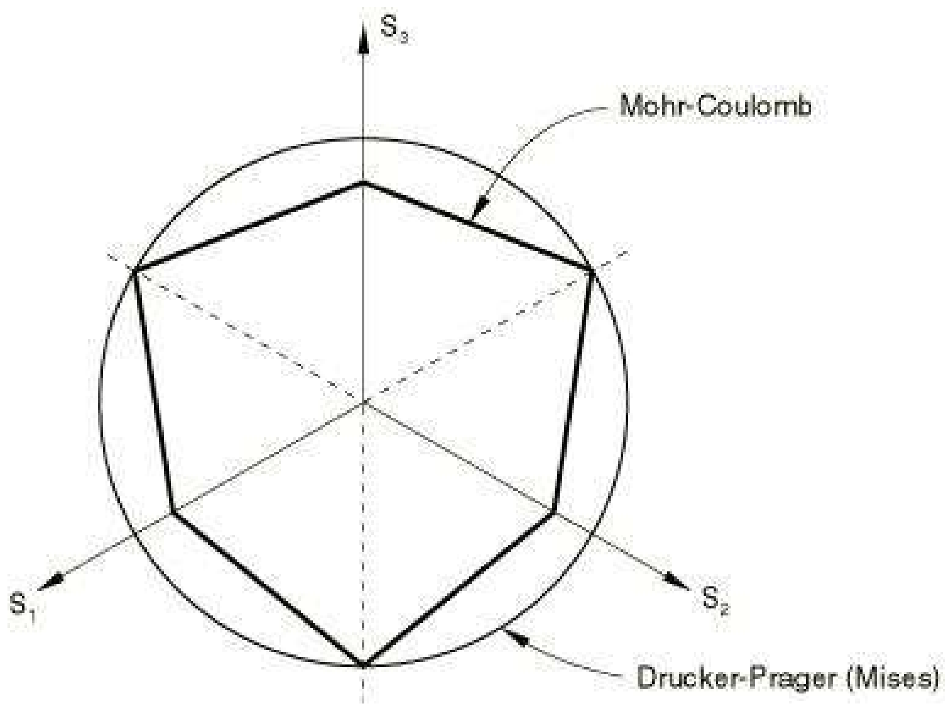

Figure 1. 18. Mohr-Coulomb and Drucker-Prager models in the deviatoric plane (Pan et al., 2011)

1.3.4. Different types of soils

Specific textures in soils are identified by gravel, sand silts, and clays, which are referred to as soil types. Coarse-grained soils include sand and gravel, and fine-grained soils include clays and silts. Knowing the distribution of particle sizes, which is a fundamental technique of defining coarse-grained soils, determines the coarseness of the soil. The following are some examples of soil types and descriptions:

- Fine particles that have been eroded from rock and transported by water that has settled on riverbeds are known as alluvial soil;
- Colluvium soil is a type of soil found near the base of mountains that has been eroded by water and gravity;
- Gypsum clay is calcium sulphate that forms from sediments in ocean brine under heat and pressure;
- Lacustrine soil is made up largely of silts and clays that have been formed in glacial lakes;
- Lateritic soil is a type of residual soil found in tropical areas that is cemented with iron oxides;
- Loam is a sand, silt, and clay mixture that may contain organic material;
- Loess is a fine-grained, wind-blown soil;
- Mud is a viscous fluid made mostly of clay and silt mixed with water.

1.3.5. Classification of soils

A soil classification system is a way for engineers to communicate with one another. It offers a systematic way of classifying soils based on their likely engineering behavior. AASHTO, USCS, and USDA are the most common of these systems. To identify and classify soils, each system has its own method and nomenclature.

1.3.5.1. AASHTO Classification System

The AASHTO system was developed specifically for highway construction and is still widely used for that purpose. With skill and expertise, reasonably accurate field classification can be determined. However, it is necessary to run sieve analyses and plasticity determinations to precisely classify a soil with this method. Table 1.3 presents the basic AASHTO soil classification system.

Table 1. 3. AASHTO soil classification system (Wisconsin Department of Transportation, 2017)

General Classification	Granular Materials								Silt-Clay Materials						
	35 percent or less of total sample passing No. 200 (75 µm)								More than 35 percent of total sample passing No. 200 (75 µm)						
Group Classification	A-1		A-3 [1]		A-2				A-4		A-5	A-6		A-7	
	A-1-a	A-1-b	A-3	A-3a	A-2-4	A-2-5	A-2-6	A-2-7	A-4a	A-4b		A-6a	A-6b	A-7-5	A-7-6
Sieve analysis, percent passing:						*				**	*			*	
No. 10 (2 mm)	50 max														
No. 40 (425 µm)	30 max	50 max	51 min	[2]					[3]	[4]					
No. 200 (75 µm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	35 max	36 min	50 min	36 min	36 min		36 min	
Characteristics of fraction passing No. 40															
Liquid limit	—	—	Non-	—	40 max	41 min	40 max	41 min	40 max		41 min	40 max		41 min	
Plasticity index	6 max	6 max	Plastic	6 max	10 max	10 max	11 min	11 min	10 max		10 max	11 – 15	16 min	≤LL-30	>LL-30
Group Index	0						4 max		8 max		12 max	10 max	16 max	20 max	
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Sand	Silty or clayey gravel and sand				Silty soils			Clayey soils			
General rating as subgrade	Excellent to good								Good to fair						

Notes

With the test data available, the classification of a soil is found by proceeding from left to right on the chart. The first classification that the test data fits is the correct classification.

* A-2-5 is not allowed under 703.16.B. A-5 and A-7-5 is not allowed under 703.16.A. See "Natural Soil and Natural Granular Soils" (203.02.H) in this manual

** A-4b is not allowed in the top 3 feet (1.0 m) of the embankment under 203.03.A.

[1] The placing of A-3 before A-2 is necessary in the "left to right" process, and does not indicate superiority of A-3 over A-2.

[2] A-3a must contain a minimum 50 percent combined coarse and find sand sizes (passing No. 10 but retained on No. 200, between 2 mm and 75 µm).

[3] A-4a must contain less than 50 percent silt size material (between 75 µm and 5 µm).

[4] A-4b must contain 50 percent or more silt size material (between 75 µm and 5 µm).

According to the AASHTO classification system, there are two general soil groups: Coarse-grained or granular and fine-grained or cohesive soils. The distinction between coarse and fine-grained soil is 35% passing the 0.075mm sieve. The system also contains eight classes to identify soils and granular materials of which the classes range from A-1 to A-3 are coarse-grained materials, A-4 to A-7 being fine-grained materials, and lastly A-8 which are organic soils.

1.3.5.2. Unified Soil Classification System

The unified soil classification system is based on the engineering properties of soil and it is most appropriate for earthwork construction. The USCS has been through several transitions since it was developed. Upon recognizing a USCS symbol of a classification group, one can immediately deduce the approximate permeability, shear strength, and volume change potential of soil and how it may be affected by water, frost, and other physical conditions. It can also be used in estimating excavation and compaction characteristics, potential dewatering situations, and workability. Table 1.4 represents the basic Unified soil classification system.

Table 1. 4. Unified soil classification system (Wisconsin Department of Transportation, 2017)

Major Divisions			Group Symbol	Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

1.3.5.3. USDA Classification System

The USDA system was developed for agricultural purposes. It has some engineering applications in that it provides a relatively easy method for the general field classification of soils. However, "loamy", while descriptive, is not an engineering term and should be avoided when discussing the engineering properties of a soil. Figure 1.19 presents the basic USDA soil classification system.

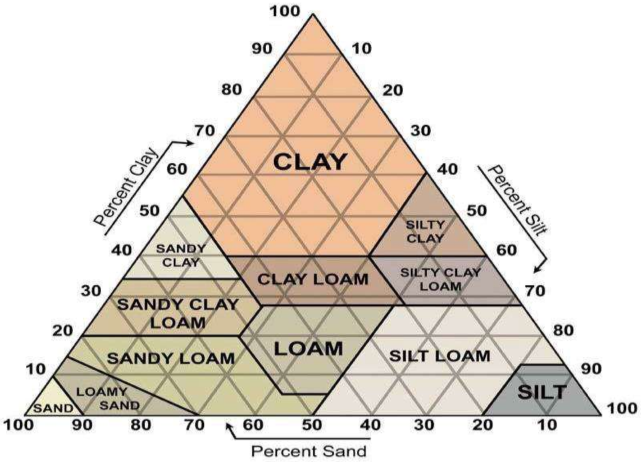

Figure 1. 19. USDA Soil Classification System (Wisconsin Department of Transportation, 2017)

1.4. Design of underground reinforced concrete water tank

Concrete for underground tank must have low permeability. This is necessary to prevent leakage through the concrete and also to provide adequate durability and protection against corrosion for the reinforcement and other embedded steel (Forth and Martin, 2014). A properly designed tank must be able to withstand the applied loads without cracks that would permit leakage. The goal of providing a structurally sound tank that will not leak is achieved by providing proper reinforcement and distribution, proper spacing and detain of construction joints, and use of quality concrete placed using proper construction procedures .

1.4.1. Design approach

When designing normal building structures, the most critical aspect of design is to ensure that the structure maintains its stability under the applied (permanent and variable) actions. When designing structures to contain liquids, it is usually found that if the structure has been sized and reinforced to retain the liquid without leakage (that is, satisfying the Serviceability Limit State, SLS), then the strength (the Ultimate Limit State, ULS requirements) is more than adequate.

1.4.1.1. Codes of practice

Structural design is often governed by a code of practice appropriate to the location of the structure. The design of liquid retaining structures, that are underground is done according to the following codes:

- BS EN 1990: Basis of structural design;
- BS EN 1991-1-1: General actions – Densities, self-weight, imposed loads for buildings;
- BS EN 1991-1-6: Actions on structures during execution;
- BS EN 1992-3, Eurocode 2: Design of concrete structures, Part 3: Liquid retaining and containing structures;
- BS EN 1997-1, Eurocode 7: Geotechnical design, Part 1: General rules;
- BS EN 206-1, Concrete, Part 1: Specification, performance, production and conformity;
- BS 8500 Parts 1 & 2: Concrete – Complementary British Standard to BS EN 206-1;
- BS 8002: Code of practice for earth retaining structures;
- BS 8102: Code of practice for protection of structures against water from the ground;
- CIRIA. Report C660, early–age thermal crack control in concrete. (This document is quoted as NCCI in the UK National Annex to BS EN 1992-3.) .

1.4.1.2. Actions on underground water tank

The forces acting on underground water tank come primarily from retained soil at the back of the wall, groundwater, stored liquid and self-weight. They can be categorised based on the structural components of the tank. On the walls of the tank there is lateral earth pressure due to retained soil (backfill), lateral ground water pressure, lateral water pressure due to the stored water**,** lateral pressures caused by placing and compacting backfill (compaction pressure) and lateral pressures caused by surcharge from loads at surface level. On the base slab there is weight of stored water, downward load from walls and self-weight of the base slab and finishes, and upward ground water pressure.

a. Categories of lateral earth pressure

There are three categories of lateral earth pressure and each depends upon the movement experienced by the retaining wall on which the pressure is acting as shown in figure 1.20. It can be categorised as at rest earth pressure, active earth pressure and passive earth pressure;

- The **at rest earth pressure** develops when the wall experiences no lateral movement. This typically occurs when the wall is restrained from movement such as along a basement wall that is restrained at the bottom by a slab and at the top by a floor framing system prior to placing soil backfill against the wall.
- The **active earth pressure** develops when the wall is free to move outward such as a typical retaining wall and the soil mass stretches sufficiently to mobilize its shear strength.
- The **passive earth pressure** develops when the base of the wall rotates enough to cause inward movement, the soil mass is compressed and mobilizes its shear strength.

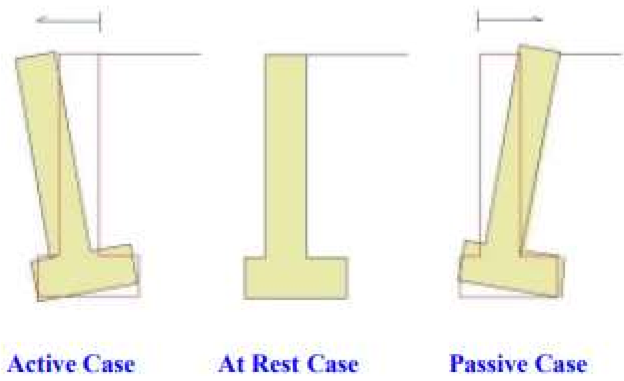

Figure 1. 20. Categories of lateral earth pressure (Richard P. Weber, P.E,2012)

b. Surcharge pressure

Surcharge loads exist on the walls of structures constructed under the ground. Surcharge can arise from a number of sources including:

- loads from adjacent roads, buildings and pavements;
- loads due to construction activities;
- variations in surface levels in undulating ground.

Surcharge loads should be properly accounted for in the design of underground tanks. In the design of walls in underground structures with depth greater than 3 m, a minimum surcharge of 10 KN/m^2 should be assumed (Narayanan and Goodchild, 2012). For shallower depths, the surcharge load may be reduced if one is confident that a surcharge of 10 KN/m^2 will not occur during the life of the structure. The surcharge discussed in this section should be treated as variable actions unless otherwise stated during the design. Some surcharge loads are permanent in nature.

c. Water pressure

Water stored in a tank exerts lateral pressure and axial tension on the walls of the tank. In the assessment of water pressure on wall of the tank (when the tank is full), the beneficial effect of the soil retained at the back of the tank should be neglected. Also, even though the maximum water storage level of tanks is usually established using discharge/flow pipes, it offers more reliability to design the water level in the tank using the full depth of the tank (Narayanan and Goodchild, 2012). Typically, liquid-retaining structures are loaded by pressure from the retained liquid. The nominal densities of materials are given in BS EN 1992-1-1, Table 1.3 gives the densities for typically retained liquids notably that of water.

Table 1. 5. Nominal density of retained liquids (Robert D. Anchor,1992).

Liquid	*Weight (kN/m³)*
Water	10.0
Raw sewage	11.0
Digested sludge aerobic	10.4
Digested sludge anaerobic	11.3
Sludge from vacuum filters	12.0

1.4.1.3. Design for ultimate limit state

For design of underground water tank, the ultimate limit states that must be verified are equilibrium, strength of structural elements and soil resistance. It is very important to consider the loads that could act on the structure during the construction stage, and during the normal use. This is particularly important for underground structures where loads and forces are influenced by the construction method adopted.

a. Design situations

In the design of tanks, two situations are considered namely when the tank is empty and when the tank is full as shown in figure 1.23.

i. Tank is empty

When the tank is empty, we consider the adverse effects of groundwater and earth pressure on the walls and the base. the active soil pressure and any vehicle pressure surcharge must be fully taken into account. The tank may also be subject to uplift forces from hydrostatic pressure at the bottom, this will tend to move the tank upwards in the ground, or float.

- **Floatation**

An empty tank constructed in water-bearing soil will tend to move upwards in the ground, or float. The ability of the structure to resist this uplift can be checked by comparing the permanent stabilising actions (self-weight of the tank) to the permanent and variable destabilising actions from the groundwater and possibly other sources. Simplistically; the tendency towards uplift must be counteracted, either by ensuring that the weight of the empty tank structure is greater than the uplift equal to the weight of the groundwater displaced by the tank as shown in figure 1.22, or by providing a heel on the perimeter of the floor to mobilise extra weight from the external soil as shown in figure 1.23.

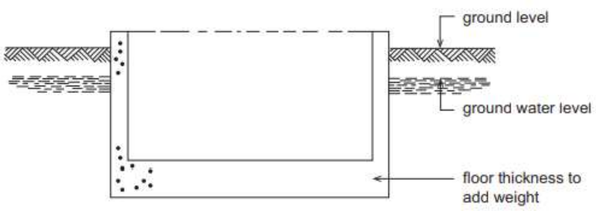

Figure 1. 21. Method of preventing flotation by additional dead weight settlement (Robert D. Anchor,1992)

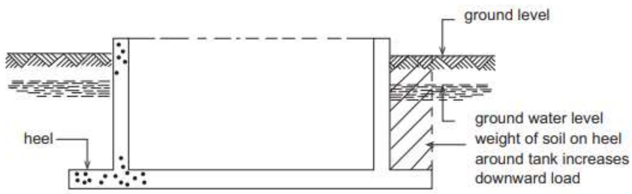

Figure 1. 22. Method of preventing flotation by provision of a heel settlement (Robert D. Anchor,1992)

ii. Tank is full

When the tank is full, we consider the pressure of the stored water on the walls of the tank and the base. No beneficial consideration should be given as regards the earth being retained on the other side wall. It is important to note that when designing for the full tank condition, no relief from the passive pressure of the soil fill should be allowed. This is due to the different elastic moduli of soil and concrete, which prevent the soil's passive resistance from developing before the concrete is fully loaded by the pressure of the fluid it contains.

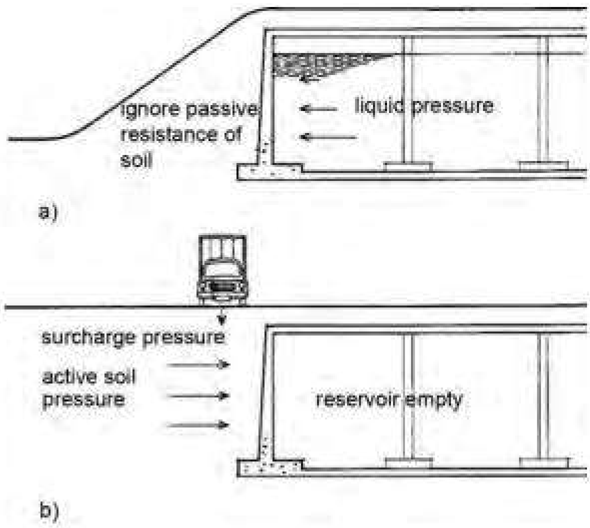

Figure 1. 23. Design loadings for external walls with soil fill (a) Reservoir full (b) Reservoir empty (Robert D. Anchor,1992).

b. Partial factors and load combinations

In the design of underground water tanks, all possible actions that could act on the structure must be fully considered. The prominent forces encountered are usually grouped into permanent and variable actions. According to EN 1997-1, ultimate limit states soil resistance and strength of structural resistance must be verified using one of three design approaches. Simpson 2007, Bond et al, 2013 proposed to adopt design approach 1; in which, two different combinations of actions are required and it involves the application of two sets of partial factors. In principle, the two different combinations can be relatively summarised as:

- ❖ Combination 1 which apply partial factors to actions (loads) and use characteristic values of soil properties;
- ❖ Combination 2 which use the characteristic values of actions, and applying partial factors to the soil properties. Partial factor of 1.3 is however applied on variable loads in this combination.

The structure must satisfy these two sets of combinations, even though combination 1 is usually more critical for the structural members, while combination 2 will likely be more critical for the soil. For each combination of loads, partial factors (γ_F) are applied to representative values of actions and soil properties. For soil properties, these are obtained by applying partial factors (γ_M)) to values of soil parameters as shown in table 1.6.

Table 1. 6. Partial factors for ULS verification of underground structures (EN 1997)

Parameter	Symbol	Combination 1	Combination 2
Actions γ_F			
Permanent actions	$\gamma_{G,unfav}$	1.35	1.00
	$\gamma_{G,fav}$	1.00	1.00
Variable actions	γ_Q	1.5	1.3
Soil properties γ_M			
Angle of internal friction φ'	γ_φ	1.00	1.25
Effective cohesion c'	γ_c	1.00	1.25
Undrained shear strength C_u	γ_{cu}	1.00	1.4

c. Categorising earth pressure and ground water

Earth pressures and ground water are treated as permanent actions. According Narayanan and Goodchild (2012); it is recommended that for ULS verification $\gamma_F(\gamma_{G,unfav})$ is equal to 1.35 should be applied to 'normal' ground water levels and $\gamma_F(\gamma_Q)$ is equal to 1.20 should be applied to pressure from water at the most unfavourable level that could occur during the lifetime of the structure (that is at surface level or, where the ground water level is known with confidence, at the ground water level plus a margin based on knowledge of the site and soil conditions).

In the simplest term, where the drainage condition of the soil cannot be fully established or during site investigation ground water was encountered but the water table could not be established, a partial $\gamma_F(\gamma_Q)$ is equal to 1.20 should be applied on the hydrostatic lateral pressure from the water. The reference height should be taken as the ground surface. If the depth water table can be firmly established, γ_F should be taken as 1.35.

1.4.2. Site conditions

The choice of location for a reservoir or tank is usually dictated by requirements beyond the responsibility of the structural engineer, but soil conditions can radically affect the design. A well-drained site with underlying soils that have even safe bearing pressure at foundation level is ideal. Where the subsoil strata dip, so that a level excavation intersects more than one type of subsoil, the effects of differential settlement must be considered. A soil survey is always required unless accurate subsoil records are available. Typically, boreholes of at least 150mm diameter should be drilled to a depth of 10m and soil samples taken and tested to determine the sequence of strata and allowable bearing pressure at various depths. The information from boreholes should be supplemented by digging test pits with a small excavator to a depth of 34 m (Robert D. Anchor,1992).

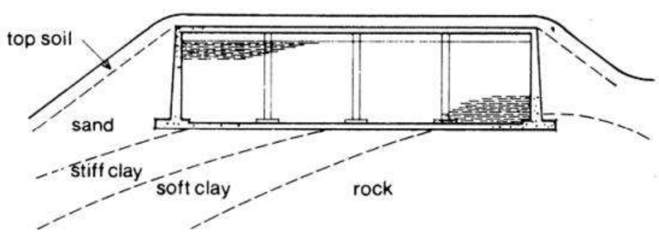

Figure 1. 24. Effect of varying strata on settlement (Robert D. Anchor,1992).

The soil investigation must also include chemical tests on the soils and ground water to detect the presence of sulphates or other chemicals in the ground that could attack the concrete and eventually cause corrosion of the reinforcement (Newman and Choo, 2003). A careful analysis of the subsoil is particularly important when the site has previously been used for industrial purposes, or where groundwater from an adjacent tip may flow through the site.

1.4.3. Constructive disposition

Design of liquid retaining structure is different from an ordinary R.C Structure as it is required that the concrete should not crack; it should be of high quality and strength also it should be leak proof.

1.4.3.1. Concrete

The Concrete for liquid-retaining structures must have low permeability in order to provide adequate durability, resistance to frost damage, and protection against corrosion for the reinforcement and other embedded steel (Forth and Martin, 2014). An uncracked concrete slab of adequate thickness will be impervious to the flow of liquid if the concrete mix has been properly designed and compacted into position. Practically, the minimum thickness of poured in-situ concrete for satisfactory performance in most structures is 300 mm. thinner slabs should only be used for structural members of very limited dimensions or under very low liquid pressures (ibid).

The design of the concrete mix shall be such that the resultant concrete is sufficiently impervious. Efficient compaction preferably by vibration is essential. The permeability of the thoroughly compacted concrete is dependent on water cement ratio. Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact. Generally concrete mix weaker than M-30 is not used. Depending up on the exposure conditions, the grade of concrete is decided for the class exposure. As British Standard European Norm 1992-3 requires that all liquid-retaining structures should be designed for at least severe conditions of exposure.

1.4.3.2. Reinforcement

Although the service tensile stress in the reinforcement in liquid-retaining structures is not always very high, it is standard practice to specify high-strength steel with a ribbed or deformed surface either in single bar form or as mesh. BS EN 1992-1-1 Annex C permits a range of characteristic yield strengths between 400 and 600 MPa; The specified characteristic strength is a statistical measure of the yield or proof stress of a type of reinforcement. The proportion of bars that fall below the characteristic strength level is defined as 5% (Figure 1.20). A material partial safety factor (for Persistent and Transient loading, γ_m = 1.15) is applied to the specified characteristic strength to obtain the ultimate design strength.

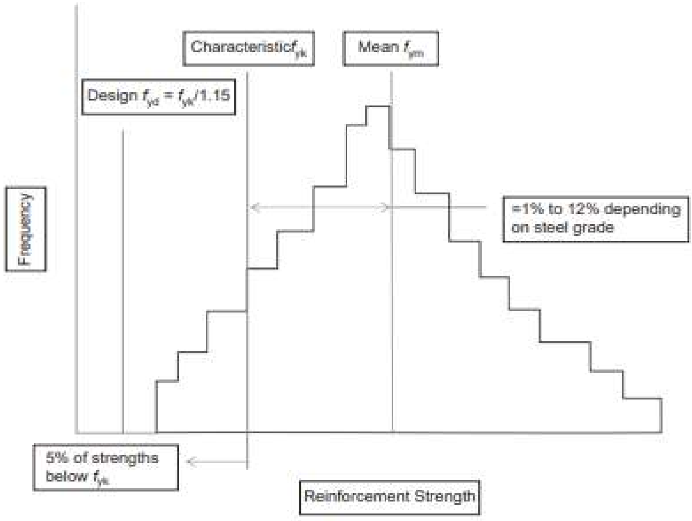

Figure 1. 25. Graphical definition of characteristic strength (Robert D. Anchor,1992).

Reinforcement embedded in concrete is protected from corrosion by the alkalinity of the cement. As time passes, the surface of the concrete reacts with carbon dioxide from the air and carbonates are formed that remove the protection. As time passes, the surface of the concrete reacts with carbon dioxide from the air and carbonates are formed which remove the protection. The minimum concrete cover for reinforcement in the tank should be at least 50 mm for normal conditions, but where particularly aggressive conditions apply, it is worth considering the use of a special type of reinforcement such as galvanized bars, epoxy-coated bars and stainless-steel bars.

1.4.3.3. Water tightness materials and crack control

Primary considerations in water tanks, besides, strength, is water tightness of tank. Complete water-tightness can be obtained by using a high strength concrete. In addition, water proofing materials can be used to further enhance the water tightness. To make concrete leak proof or water tight, internal water proofing or water proof linings are frequently used. The object of using them is to fill the pores of the concrete and to obtain a dense and less permeable concrete. According to BS EN 1992-3 (2006) there is four tightness classes as shown in the following table 1.7.

Table 1. 7. Tightness classification (Robert D. Anchor,1992).

Tightness class	*Requirements for leakage*
0	Some degree of leakage acceptable, or leakage of liquids irrelevant.
1	Leakage to be limited to a small amount. Some surface staining or damp patches acceptable.
2	Leakage to be minimal. Appearance not to be impaired by staining.
3	No leakage permitted

For the serviceability limit state, the maximum (limiting) crack width is between 0.05 mm and 0.2 mm, depending on the ratio of the hydrostatic pressure to wall thickness. Batty and Westbrook (1991) mentioned that External loading and changes of temperature during the working life of the structure, moisture content, inadequate reinforcement associated with poor constructions techniques, differential settlement are often identified as cause of cracking. Then it is strongly advised to:

- ❖ Use aggregate with a low thermal expansion or which are not shrinkable;
- ❖ Use the minimum cement content (325kg/m^3), respect requirement for durability (concrete cover not less than 40 mm);
- ❖ Use cement with a low rate of heat evolution;
- ❖ Provision movement joint;
- ❖ Localise cracking within a particular member between movement joint by using reinforcement.

1.4.4. Water tightness and waterproofing

Watertightness is the most important serviceability limit state consideration when designing water-retaining structures. According to BS 8102 there are three types of waterproof protection for concrete structures and they are as follows:

- Type A – Barrier protection;
- Type B – Structurally Integral Protection;
- Type C – Drained Protection.

1.4.4.1. Type A – Barrier Protection

Type A is a type of waterproof construction where a continuous barrier of waterproof membrane is applied to the outside or inside of walls and floors. Most waterproofing membranes do not have sufficient load-bearing properties and therefore cannot be applied to floors and left uncovered. It is usually very common to cover them with screed to hold them in place and form a protective layer.

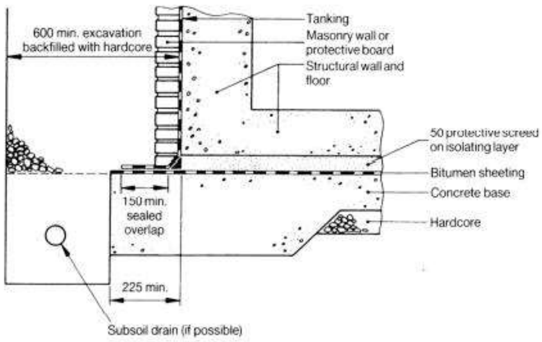

Figure 1. 26. External membrane protection (Robert D. Anchor,1992).

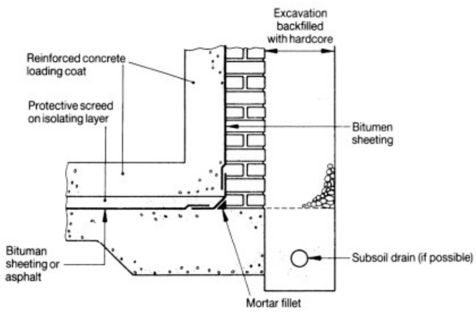

Figure 1. 27. Internal membrane protection (Robert D. Anchor,1992).

1.4.4.2. Type B – Structurally Integral Protection

With type B, the structural system is designed for water tightness and is not dependent on membranes or barriers. Such constructions are designed to BS EN 1992-3 so crack control can be used to minimize the risk of water ingress. The crack width limits are usually defined before the design begins. Despite the watertight construction of the concrete box, however, the construction joint is a potential source of leakage, which is why water stops must be provided, regardless of whether there is a risk of groundwater or not. Good workmanship is also important for construction type B, since deficiencies such as honeycombs, poor compaction, foreign matter in the concrete, poor installation of water rails can jeopardize the entire effort.

a. Types of water stops

Water bars (also called water stops) are barriers provided in construction joint so as to stop any potential source of leakage into the reinforced concrete water retaining structures. The most popular types of water stops are:

i. Preformed strips water stops

Preformed strips are made from impervious but durable materials that are embedded into the concrete when pouring. Typically, they are installed at construction joints to provide a watertight seal against movement within the joint. These are popularly called water bars. Water bars are usually made of flexible PVC (polyvinyl chloride) material or steel, with the former being more popular. When inserting, however, care must be taken to ensure that the flexible PVC water bar does not collapse during placement of concrete. This is the advantage that the steel water bar has. Steel water bars are typically made of 1.5mm thick black steel, which due to its stiffness stays in place permanently during concreting.

Figure 1. 28.PVC water stops seals(www.deep-jyotti.com)

ii. Swellable Water Stops

These water stops, produced with hydrophilic material designed to stop water infiltration through the cast in place concrete construction joints by expanding upon contact with water to form a positive seal against the concrete. Appropriate care must also be taken at the joints to ensure that there will be no room for water penetration.

Figure 1. 29. Water stops swellable hydrophilic(www.arconsupplies.co.uk)

iii. Cementitious crystalline water stops

The water stopping action of this material is from salt crystallization in the presence of water within the pores and capillaries of the concrete. They are prepared by mixing cement, fillers, and chemicals on site as slurry, which is then applied on the surface of the old concrete before pouring the new concrete. They are not suitable for use in expansion joints but construction joints only.

Figure 1. 30. Cementitious crystalline water stops(www.kryton.com)

iv. Injectable water bars

With this method, resins or other proprietary fluid in injected under pressure through perforated or permeable tubes that have been preinstalled on the old concrete before the pour of the new concrete. This is done after temperature and shrinkage movements have stabilized sufficiently, and the resin flows out of the tube into cracks, fissures or holes in the joint, thereby sealing the water paths in the joint. This suitable for construction joints only.

Figure 1. 31. Resin injection tube water bar(www.newtonwaterproofing.com)

1.4.4.3. Type C – Drained Protection

Type C construction demands a drained cavity within the structure and relies on this cavity to collect any water from seepage and drain it to sumps for pumping. A dry internal environment can be achieved with confidence using a drained cavity wall and floor construction provided that any defects are corrected and the system is maintained.

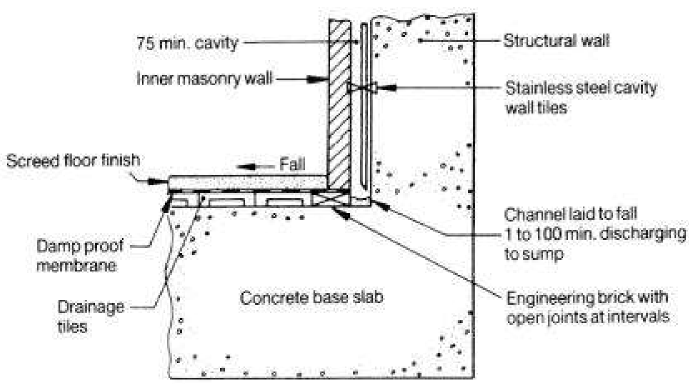

Figure 1. 32. Drained cavity construction (Robert D. Anchor,1992).

1.4.5. Factors affecting construction of underground liquid containing structures

It is very important that the contractor use best practices to ensure concrete mixes, transport, handling, placement and consolidation are all carefully carried out to standards to ensure the required level of watertightness. The location and spacing of reinforcements should be in accordance with the working drawings, and the installation of water barriers should be done under close supervision. The factors that influence good water retaining structure construction are briefly highlighted as follows;

1.4.5.1. Movement and Construction Joints

There are basically two types of joints in water retaining structures; they are movement joints and construction joints.

a. Movement joints

Movement joints are designed to reduce the risk of cracking and are therefore usually classified as expansion and contraction joints. Expansion joints are provided when reversible movements are expected, and contraction joints are appropriate when only one contraction needs to be accommodated. At expansion joints, the two concrete surfaces tend to move closer together, while at contraction joints, they tend to move away from each other.

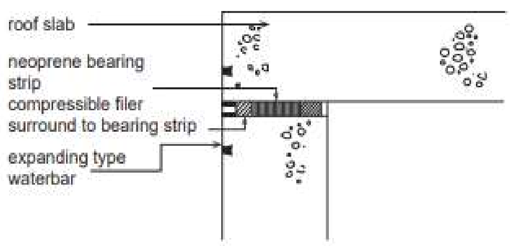

Figure 1. 33. Detail for movement joint between wall and roof slab (Robert D. Anchor,1992).

i. Construction joint

Contraction joints may or may not have reinforcements going all the way across the joints. The most common form of contraction joint is achieved by creating a plane of weakness such that cracks occur at preferred lines at intervals. This level of weakness can be achieved by making a saw cut in the concrete section so cracks would likely propagate from there.

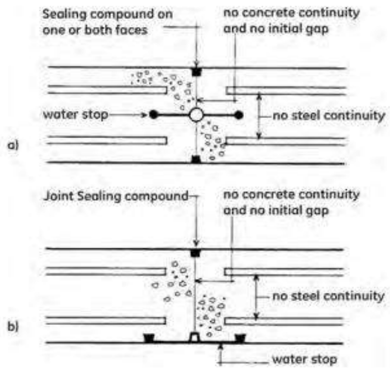

Figure 1. 34. Complete contraction joints (a) wall (b) floor (Robert D. Anchor,1992).

ii. Expansion joint

Expansion joints are constructed in such a way that there is a gap between the two concrete surfaces. A good idea of the movement to be expected is important in order to select adequate filling material. Usually, a compressible layer of material is used to fill the gap, and for water-retaining structures, such a material should be able to withstand high pressures, be non-toxic, and preferably possess hydrophobic properties, otherwise the joint must be sealed for water tightness. In addition, the sealing material should be able to compress by the specified amount and return to its original shape after the concrete has contracted.

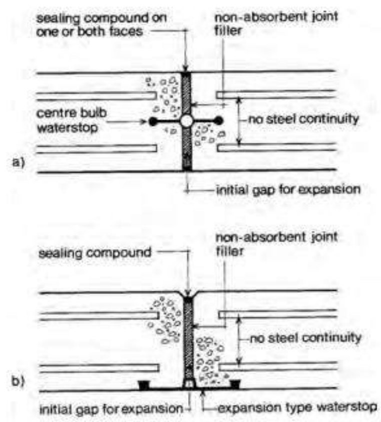

Figure 1. 35. Expansion joints (a) floor (b) wall (Robert D. Anchor,1992).

b. Construction Joints

It is rarely possible to build a reinforced concrete structure in one piece. It is therefore necessary to design and locate joints that will allow the contractor to construct the elements of the structure in appropriate sections. In normal structures, the location of construction joints is generally determined by the designer and the number of joints and their exact location may be decided by the contractor subject to final approval by the designer. Construction joints are monolithic strong joints in reinforced, watertight concrete that join works carried out on two different days. The first section has starter bars that stick out so the new reinforcement overlaps with the old. When building tanks, it is convenient to provide a short 'kicker' at the transition between a floor slab and a wall, which will allow the formwork for the walls to be placed accurately and easily.

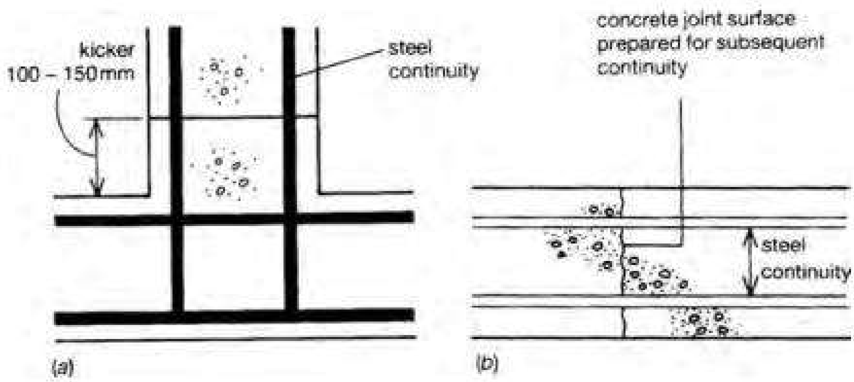

Figure 1. 36. Construction joints (a) horizontal joint between base slab and wall (b)vertical joint (Robert D. Anchor,1992).

1.4.5.2. Methods of construction

There are various methods of constructing underground water retaining structures. One of the most difficult situations in the construction of underground structures is the groundwater problem. In the absence of groundwater, construction can be carried out without much trouble at minimal cost. In this section, we will discuss some of the common methods used in the construction of underground tanks.

a. Open Excavation and Bottom-Up Construction

If the construction site conditions permit, an open excavation of the construction site can be carried out to the required depth. If this method is chosen, the contractor should consider the following points such as impact of the open excavation on neighboring buildings or other nearby properties, removal of excavated material, supporting or appropriately sloping sides of excavated trenches to avoid collapse (very important) and the control of groundwater and drainage.

In the case of open construction, all adjacent buildings and property boundaries must be taken into account. This construction method may result in a loss of bearing capacity of foundation for other nearby structures and should be considered. On the other hand, there should be adequate provision for removing excavated materials before they become problematic for the site. The sides of the excavation should be shored up or sloped once the depth of the foundation exceeds the critical depth. This is to avoid the problem of caving in.

b. Top-down construction (Sinking of tanks)

To save money on building underground tanks in areas with high water tables, contractors can cast the walls of the tank onto the ground surface and excavate inside the tank until it is sunk to the required depth. After the tank has been sunk and leveled to the required depth, the bottom slab is cast in situ to depth on site to bond it to the walls of the tank. In this case, this means that reinforcement and water bars that extend into the base slab must be provided and protected throughout the process. Groundwater is also continuously pumped throughout the process.

It should be noted that the tank walls are subjected to stresses during the sinking process, which can damage the tank shell. Therefore, sinking of the tanks should be done 28 days after the required casting and input from the structural engineer is needed. In order to effectively achieve the watertightness of the structure, this process must be carried out under strict supervision from start to finish and is suitable for smaller tank shells.

1.4.5.3. Construction Defects – Honeycombing

Honeycombs or lack of compaction is the most common constructions defect that could compromise the watertightness of a water retaining structure. This can be caused by a number of reasons such as incorrect mix design consistency, lack of proper consolidation, poor reinforcement placement, incorrect aggregate specification, too much water in mix causing segregation and improper build sequencing.

It is the contractor's responsibility to ensure that the concrete supplied is of the required mix design with the appropriate consistency class. He is also responsible for placing, compacting and curing the concrete and must ensure that this is carried out in accordance with standards. In the case of honeycombs due to improper mix design, the concrete manufacturer will conduct a site inspection to see and evaluate the poured concrete for defects associated with poor or insufficient compaction hardening methods. Any deficiencies would normally be highlighted and an agreed remediation would be required. Typical remediation works for honeycombs are crack injection, mortar filling and epoxy coatings.

Conclusion

This chapter aimed to give the current state of knowledge on the classification of various water tank types, a general overview of different types of soils, and essential design methods for a liquid-retaining structure that is built below the ground level. It clearly indicates that reinforced concrete underground tanks are highly recommended for water storage. When designing this structure, not only strength requirements but also serviceability requirements must be considered. A properly designed tank must be able to withstand the loads applied without cracking that would allow leakage. One of the applied loads is the earth pressure, which depends on the type of soil, its calculation is based on the mechanical properties of the soil, that is, the angle of friction and the shear strength of a given soil. The goal of providing a structurally sound tank that will not leak is achieved by providing the correct amount and distribution of reinforcement, the correct spacing and detail of construction joints, and the use of quality concrete placed using proper construction practices. The next chapter focuses solely on the methodology used to meet the design requirements of an underground reinforced concrete tank.

CHAPTER 2.METHODOLOGY

Introduction

Methodology is very important part which consist to present the processes, the methods that will be used to achieve the work objectives. This chapter will define the conception of the case study through design assumptions based on existing standards, of the rectangular reinforced concrete underground water tank. After that, the presentation of analytical analysis and limit state design method of the case study will be explained. This is to achieve a structurally sound tank that will not leak.

2.1. Conception of the case study

The process of conceiving an underground water tank involves the presentation of the assumptions made for the conception and pre-design of the structural element of the water tank to be studied. The purpose of this paragraph is to present the codes, specifications that will be used for the design of the underground tank, then the actions and combinations of actions that result in section 2.2.

2.1.1. Determination of the tank capacity

For tanks, it is important to highlight that the capacity of the tank is based on considerations such as individual/household water demand, firefighting purpose and miscellaneous uses such as washing of cars. From these considerations, the preliminary tank dimensions can be fixed such as depth of tank (in-to-in), length (in-to-in), width (in-to-in), thickness of walls and thickness of base.

The use of the conceived underground water tank is to store water for firefighting purpose. The determination of the quantity of water necessary for a fire extinction depends on the building type, its characteristics and other parameters like the surface area of the local according to the D9 practical guide for design of water requirements necessary for external defence against fire. Table 2.1 shows how to determine the water needs for some types of buildings receiving the public.

Table 2. 1. Water Needs (FFA, MET, and al, 2020)

Risk	**Class 1** **N: Restaurants, bars** **O and OA: Hotels** **R : Class rooms**	**Class 2** **L: Meeting Rooms** **P: Dance Halls** **Y : Museums**
Surface	Water needs (m^3/h)	
≤ 500 m²	60	60
≤ 1000 m²	60	75
≤ 2000 m²	120	150
Number of Fire water points	According to the global flow rate required and the partition according to the geometry of the buildings	
Maximum Distance between the fire water points	200 m	200 m
Maximum distance between the 1st water point and the principal entrance	150 m (Dry column = 60 m when required)	150 m (Dry column = 60 m when required)
Minimum Duration	Apart from particular dispositions, the minimum duration is 2 h	
Autonomous duration, complete and design according to the storage and the existing activity, with respect to the existing references; Installation maintained and regularly controlled; Installation in permanent service		

2.1.2. Materials

Materials that will be considered in the design should be in accordance with the recommendations of Eurocode 2. It is about concrete and reinforcement steel.

2.1.2.1. Concrete

The compressive strength, tensile strength and modulus of elasticity of the concrete are presented.

a. Compressive strength

Compressive strength of the concrete is given by equation 2.1.

$$f_{cd} = \frac{0.85\, f_{ck}}{\gamma_c} \qquad 2.1$$

Where:

-f_{ck} is the characteristic compressive strength of concrete at 28 days;

-γ_c is the partial safety factor for concrete. (See table A1.1 in annex 1)

b. Tensile strength

Tensile strength of the concrete is given by equation 2.2.

$$f_{ctm} = 0.3(f_{ck})^{\frac{2}{3}} \qquad 2.2$$

c. Modulus of elasticity

Modulus of elasticity is given by equation 2.3.

$$E_{cm} = 2200\left(\frac{f_{ck}+0.8}{10}\right)^{0.3} \qquad 2.3$$

2.1.2.2. Reinforcement steel

Characteristic yield strength of the steel reinforcement is given by equation 2.4

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \qquad 2.4$$

Where:

-f_{yk} is the yield strength of steel

- γ_s is the partial safety factor of steel. (See table A1.1 in annex 1)

2.1.3. Concrete cover

For concrete structures, the design working life is set by Eurocode 0 (see table A1.2 in Annex 1). In order to ensure this working life of the structure Eurocode 2 set protection of structural element against the environmental action by the definition of a concrete cover. It takes into account the structural class of the structure and the exposure class. This concrete cover is defined as the distance between the surface reinforcement and the concrete surface. The nominal value of the concrete cover is defined by equation 2.5.

$$C_{nom} = C_{min} + \Delta C_{dev} \qquad 2.5$$

Where:

-ΔC_{dev} is the allowance in design for deviation with a recommended value of 10 mm

-C_{min} is the minimum concrete cover defined in equation 2.6

The minimum cover C_{min} is defined by:

$$C_{min} = max(C_{min,b}; C_{min,dur}; 10\ mm) \qquad 2.6$$

Where:

-$C_{min,b}$ is the minimum cover due to bond requirement, equal to the diameter of the bars or the equivalent diameter in the case of bundled bars

-$C_{min,dur}$ is the minimum cover due to environmental conditions obtained from the table A1.2 of the annex 1

2.2. Analysis and design of the structural elements of the rectangular underground water tank

For analysis and design of tanks, elastic analysis is normally necessary as a basis for checking serviceability cracking (Reynolds et al, 2008). The elastic analysis of rectangular tanks uses coefficients in order to obtain the internal forces in the members of the tank. The bending moments and shear forces in the walls of a rectangular tank can be obtained based on the assumed support conditions and span ratio.

2.2.1. Check for uplift when tank is empty

This consist of calculating and comparing stabilizing and destabilizing actions. In case the latter is greater than the former, a remedial solution is proposed so as to avoid the flotation of the tank.

2.2.1.1. Stabilising action

As this is a favorable load, the stabilizing action at ULS is determined as in equation 2.7.

$$\boldsymbol{G_{stb,d}} = \gamma_{G,inf}.W_T \qquad 2.7$$

where:

-$G_{stb,d}$ is the favorable stabilising load actions at ULS

-W_T is the total weight of the tank when empty

-$\gamma_{G,inf}$ is the partial factor for equilibrium verification read in table 2.2

2.2.1.2. Destabilising action

As this is an unfavorable load due to the uplift pressure of water under the base slab, the destabilising action at ULS is determined as in equation 2.8.

$$U_{dst,d} = \gamma_{G,inf}.W_b.U_P \qquad 2.8$$

where:

-$U_{dst,d}$ is the unfavorable destabilising load actions at ULS

-W_b is the weight of the the base slab when tank empty

-U_P is the uplift pressure of water under the base slab

-$\gamma_{G,inf}$ is the partial factor for equilibrium verification read in table 2.2

2.2.1.3. Equilibrium verification

Equilibrium should be verified in accordance with BS EN 1997-1-1 as shown in equation 2.9. The relevant partial factors of safety for the verification are shown in table 2.2.

$$G_{stb,d} > U_{dst,d} \tag{2.9}$$

Where:

$G_{stb,d}$ is the favourable stabilising load actions at ULS;

$U_{dst,d}$ is the destabilising uplift action at ULS.

a. Partial safety factors

The partial safety factor to be applied to the permanent stabilising action of the water is 0.9 and the partial safety factor for the destabilising action is 1.1.

Table 2. 2. Partial factors for equilibrium verification (EN 1997-1-1)

Action	Favorable(stabilizing)	Unfavorable(destabilizing)
Permanent γ_G	0.9	1.1
Variable γ_Q	0.0	1.5

2.2.1.4. Failure of the uplift verification

If the structure fails the uplift verification, the remedial action to increase the weight of the structure. This is done by extending the base beyond the slab. Extending the base offers the best advantage because after backfilling, the weight of the soil on the extended base offers beneficial action in resisting uplift. This remedial solution to resisting uplift of underground tanks is shown in figure 2.1.

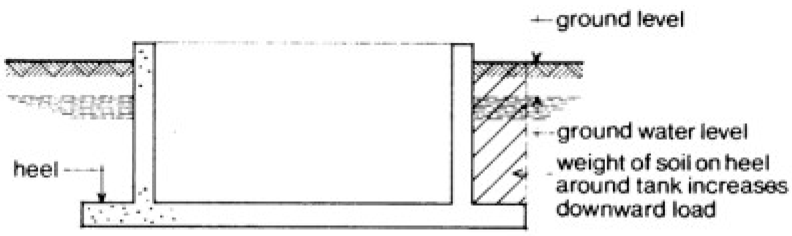

Figure 2. 1. Provision of a heel by extending the base slab (Robert D. Anchor,1992)

2.2.2. Span ratio and support conditions

In this section the span ratio is determined with its corresponding support condition for each structural element of the rectangular tank.

2.2.2.1. Span ratio

Span ratio is the ratio between the length of the longer side to the shorter side of the any structural element of an underground rectangular tank, as shown in equation 2.10. The obtained values are used to read the corresponding coefficients for horizontal bending moments, vertical bending moments and shear forces. The length of either shorter side or longer side is shown in figure 2.2.

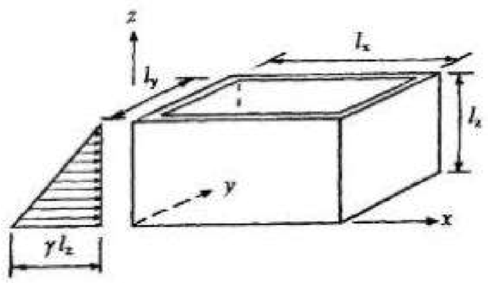

Figure 2. 2. Schematic representation of a rectangular tank (Reynolds and Steedman ,2005)

$$\text{Span ratio} = \frac{l_x}{l_z} \qquad 2.10$$

Where:

-l_x is the length of the longer side

-l_z is the length of the shorter side

2.2.2.2. Support conditions

Reinforced concrete tanks are usually covered with monolithic top slabs, therefore, the top of the wall is usually treated as hinged or fixed. If a wall is monolithic with the top slab, the joint will exhibit a behavior that tends towards hinged when the tank is full as the joint rotates. However, when there is earth pressure on the walls of the tank when empty, a closing corner moment is developed at the joint. At the base of the tank, the joint can be treated as hinged or fixed.

2.2.3. Analysis and design of top slab of the underground tank

The top slab of the underground tank should be able to resist a characteristic variable action and characteristic permanent action. It is analysed and designed as a two-way slab with all edges considered discontinuous.

2.2.3.1. Loadd per unit area calculation

At ULS, the total load per unit area acting on the top slab is determined based on equation 2.11.

$$n = 1.35g_k + 1.5q_k \tag{2.11}$$

where:

-g_k is the characteristic permanent action;

-q_k is the characteristic variable action;

- n is the total load per unit area.

2.2.3.2. Bending moments on the top slab of the underground tank

From the value of the span ratio obtained from equation 2.10, the design bending moment of the top slab can be evaluated using the equations 2.12 and 2.13.

$$m_{sx} = \beta_{sx} n l_x^2 \tag{2.12}$$

$$m_{sy} = \beta_{sy} n l_x^2 \tag{2.13}$$

Where:

-m_{sx} is the design bending moment on short span;

-m_{sy} is the design bending moment on long span;

-β_{sx} and β_{sy} are the bending moment coefficients for short and long span respectively from table 2.3;

-n is the total load per unit area;

-l_x is the length of the shorter side.

Table 2. 3. Bending moment coefficients for short and long span (Table 3.14; BS 8110-1997)

Type of panel and moments considered	Short span coefficients, β_{sx}								Long span coefficients, β_{sy} for all values of l_y/l_x
	Values of l_y/l_x								
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Interior panels									
Negative moment at continuous edge	0.031	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
Positive moment at mid-span	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
One short edge discontinuous									
Negative moment at continuous edge	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.037
Positive moment at mid-span	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	0.028
One long edge discontinuous									
Negative moment at continuous edge	0.039	0.049	0.056	0.062	0.068	0.073	0.082	0.089	0.037
Positive moment at mid-span	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.028
Four edges discontinuous									
Positive moment at mid-span	0.055	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

2.2.3.3. Shear forces on the top slab of the underground tank

From the value of the span ratio obtained from equation 2.10, the design shear forces of slab can be evaluated using the equations 2.14 and 2.15.

$$v_{sx} = \beta_{vx} n l_x \qquad 2.14$$

$$v_{sy} = \beta_{vy} n l_x \qquad 2.15$$

where:

-v_{sx} is the design shear force on short span;

-v_{sy} is the design shear force on long span;

-β_{vx} and β_{vy} are the shear coefficients for short span and long span respectively from table 2.4;

-n is the total load per unit area;

-l_x is the length of the shorter side

Table 2. 4. Shear force coefficient for short and long span (Table 3.15; BS 8110-1997)

Type of panel and location	β_{vx} for values of l_y/l_x								β_{vy}
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Four edges continuous									
Continuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
One short edge discontinuous									
Continuous edge	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
Discontinuous edge	—	—	—	—	—	—	—	—	0.24
One long edge discontinuous									
Continuous edge	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
Discontinuous edge	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	—
Four edges discontinuous									
Discontinuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

2.2.4. Analysis and design of the walls of the underground tank

In rectangular tanks, there is direct axial tension in the plane of the walls due to the lateral loads supported by adjacent contiguous walls. This means that when considering horizontal spans, the shear forces at the vertical edges of one wall will result to axial forces in the adjacent walls.

2.2.4.1. Tension in walls of the tank

When the tank is full, the shear force due to water pressure on the long wall will result to axial tension in the shorter wall and vice versa as shown in figure 2.3. The interaction of these forces should be included in the design.

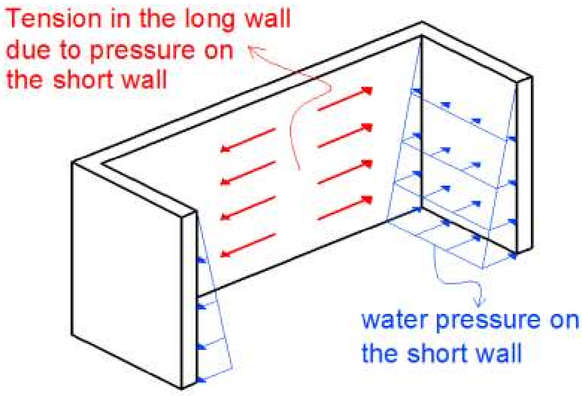

Figure 2. 3. Tension in walls of rectangular water tank (Ubani Obinna,2018)

2.2.4.2. Pressure calculation on the walls of the underground tank

There are different procedures of calculating lateral pressures acting on the walls of the underground tank. In the next subsection, we are going to focus on the simple methods that are suitable for hand calculations.

a. The lateral earth pressures and earth pressure coefficients

Lateral earth pressure is related to the vertical earth pressure by a coefficient termed the at rest earth pressure coefficient (K_o), active earth pressure coefficient (K_a) and passive earth pressure coefficient (K_p).

i. Earth pressure coefficients

The earth pressure coefficients are calculated as follows in equation 2.16,2.17 and 2.18 respectively.

- **At rest coefficient**

One common earth pressure coefficient for the "at rest" condition in granular soil is:

$$K_o = 1 - \sin\varphi \quad 2.16$$

Where:

-K_o is the earth pressure at rest

-φ is the angle of internal friction

- **Active and passive earth pressure coefficients**

These coefficients are obtained based on the Rankine's theory; which assumes that, there is no adhesion or friction between the wall and soil, lateral pressure is limited to vertical walls and failure (in the backfill) occurs as a sliding wedge along an assumed failure plane defined by angle of shearing resistance. The Rankine active and passive earth pressure coefficient for the specific condition of a horizontal backfill surface is calculated as follows in equation 2.17 and equation 2.18 respectively.

$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} \quad 2.17$$

$$K_p = \frac{1+\sin\varphi}{1-\sin\varphi} \quad 2.18$$

Where:

-K_a is the active pressure coefficient

-K_p is the passive pressure coefficient

ii. The lateral earth pressures

The active lateral earth pressure and passive lateral earth pressure at a depth z below the ground surface is given by equation 2.19 and equation 2.20 respectively.

$$P_a = K_a\,\sigma'_v + u \quad 2.19$$

$$P_p = K_p\,\sigma'_v + u \quad 2.20$$

Where:

-P_a is the active lateral earth pressure

-P_p is the passive lateral earth pressure

- σ'_v is the effective vertical pressure

-u is the pore water pressure

b. Lateral pressures caused by surcharge loading

Lateral load due to surcharge is treated as a uniformly distributed surcharge (q KN/m^2) on the walls of the retaining wall. This is calculated by considering the surcharge pressure as an initial overburden (pressure load) at the ground surface level. The overburden surcharge pressure is multiplied by the appropriate earth pressure coefficient to obtain the lateral thrust shown in equation 2.21.

$$P_{ah} = K_h \, q \qquad 2.21$$

Where:

-K_h is the design coefficient of active or at rest pressure as appropriate

-q is the surcharge load (KN/m^2)

Generally, it will be satisfactory to use the active pressure coefficient for calculation of surcharge pressure, while some scholars are of the opinion that at rest pressure coefficient should be used.

c. Water pressure

The pressure due to water stored in a tank is given by:

$$P_w = \gamma_w Z \qquad 2.22$$

Where:

-P_w is the water pressure

-γ_w is the unit weight of water

-Z is the depth of the tank

Typical lateral pressure distribution on a retaining wall is shown in figure 2.4.

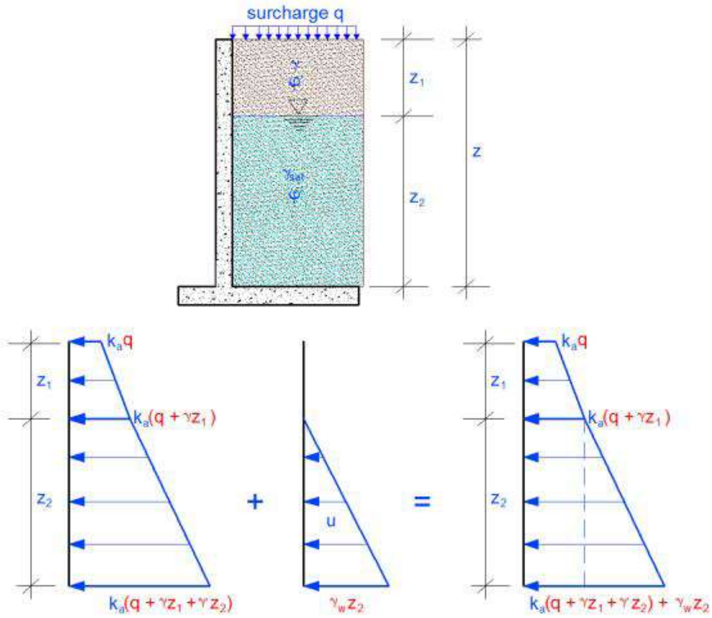

Figure 2. 4. Lateral pressure distribution on a retaining wall (Ubani Obinna,2018)

2.2.4.3. Analysis of the trapezoidal load on the walls of the underground tank

When analyzing lateral earth and water pressure on tank, surcharge load is considered at the surface of the ground. Hence, there will be a trapezoidal load on the walls of the tank instead of a triangular load.

a. Method for evaluating trapezoidal loads

The approximate approach of handling this problem is to separate the trapezoidal load into its uniformly distributed load and triangular components as shown in figure 2.5. The two load components are added; with the coefficients readily available, the bending moments and shear force are obtained. The walls are considered as one long edge discontinuous slab.

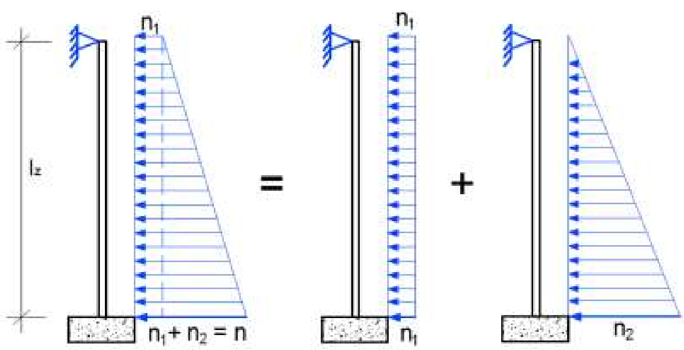

Figure 2. 5. Approximate method for evaluating trapezoidal loads on walls of tanks (Ubani Obinna,2018)

i. Bending moments on the walls of the underground tank

From the value of the span ratio obtained from equation 2.10, the design bending moment of the top slab can be evaluated using the equation below,

$$m_{sz} = \beta_z n_1 l_z^2 \; + \; a_z n_2 l_z^2 \qquad 2.23$$

$$m_{sx} = \beta_x n_1 l_x^2 \; + \; a_x n_2 l_z^2 \qquad 2.24$$

Where:

-m_{sz} is the design bending moment on short span;

-m_{sx} is the design bending moment on long span;

-β_z is the uniformly distributed load bending moment coefficients for short span from table 2.5;

- β_x is the uniformly distributed load bending moment coefficients for long span from table 2.5;

-a_z is the triangular load bending moment coefficients for short span from table 2.6;

-a_x *is the* triangular load bending moment coefficients for long span from table 2.6;

-n_1 is the uniformly distributed load per unit area;

-n_2 is the triangular distributed load per unit area;

-l_z is the length of shorter side;

-l_x is the length of longer side.

Table 2. 5. Bending moments coefficients for a wall supporting a uniformly distributed load (Narayanan and Goodchild ,2012)

Type of panel and moments considered		Short span coefficients, β_z			Long span coefficient, β_x
		Values of k = width l_x/height l_z			
		1	1.5	2	
Interior panel					
	Negative moments at continuous edge	0.031	0.053	0.063	0.032
	Positive moment at mid-span	0.024	0.040	0.048	0.024
One long edge discontinuous					
	Negative moments at continuous edge	0.039	0.073	0.089	0.037
	Positive moment at mid-span	0.03	0.055	0.067	0.028

Table 2. 6. Bending moment coefficients for a wall supporting a triangular load (Narayanan and Goodchild ,2012)

Type of panel and moments considered	Coefficients α_x, α_z — Values of k = width l_x/height l_z: 0.5	1	2	3	4
3 sides fixed, top pinned					
Negative moments at edge, α_x	0.012	0.029	0.037	0.037	0.037
Positive moment at mid-span for span l_x, α_x	0.006	0.012	0.01	0.009	0.009
Negative moments at bottom edge, α_z	0.011	0.035	0.062	0.066	0.067
Positive moment at mid-span for span l_z, α_z	0.003	0.011	0.026	0.029	0.029
3 sides fixed, top free					
Negative moments at edge, α	0.012	0.03	0.066	0.091	0.099
Positive moment at mid-span for span l_x, α	0.006	0.013	0.028	0.024	0.017
Negative moments at bottom edge, α_z	0.011	0.035	0.086	0.127	0.149
Positive moment at mid-span for span l_z, α_z	0.003	0.01	0.016	0.011	0.007

ii. Shear forces on the walls of the underground tank

From the value of the span ratio obtained from equation 2.10, the design shear forces of slab can be evaluated using the equation below;

$$v_{sz} = \beta_{vz} n_1 l_z + \alpha_{vz} n_2 l_z \quad 2.21$$

$$v_{sx} = \beta_{vx} n_1 l_z + \alpha_{vx} n_2 l_z \quad 2.22$$

Where:

-v_{sz} is the design shear force on short span;

-v_{sx} is the design shear force on long span;

-β_{vz} is the uniformly distributed load shear force coefficients for short span from table 2.4;

-β_{vx} is the uniformly distributed load shear force coefficients for long span from table 2.4;

-α_{vz} *is the* triangular distributed load shear force coefficients for short span from table 2.7;

-α_{vx} *is the* triangular distributed load shear force coefficients for long span from table 2.7.

Table 2. 7. Triangular distributed load shear force coefficients for short and long span (Table 2.53; Reynolds et al, 2008)

Type of panel with moments and shears considered		Coefficients for values of l_x/l_z								
		0.5	0.75	1.0	1.25	1.5	2.0	2.5	3.0	4.0
1. Top hinged, bottom fixed										
Negative moment at side edge	α_{mx}	0.012	0.022	0.029	0.033	0.036	0.037	0.037	0.037	0.037
	α_{mz}	0.002	0.004	0.006	0.007	0.007	0.007	0.007	0.007	0.007
Positive moment for span l_x	α_{mx}	0.006	0.010	0.012	0.013	0.012	0.010	0.009	0.009	0.009
Negative moment at bottom edge	α_{mz}	0.011	0.023	0.035	0.045	0.053	0.062	0.065	0.066	0.067
	α_{mx}	0.002	0.005	0.007	0.009	0.011	0.012	0.013	0.013	0.013
Positive moment for span l_z	α_{mz}	0.003	0.007	0.011	0.016	0.021	0.026	0.028	0.029	0.029
Shear force at side edge	α_{vx}	0.17	0.22	0.24	0.25	0.26	0.26	0.26	0.26	0.26
Shear force at bottom edge	α_{vz}	0.20	0.26	0.32	0.36	0.38	0.40	0.40	0.40	0.40
Shear force at top edge	α_{vz}	0.03	0.05	0.07	0.09	0.11	0.11	0.11	0.11	0.10

2.2.4.4. Analysis of the base slab

The base of tank is subjected to earth pressure reaction due to the self-weight of the tank and the weight of the liquid stored. The analysis of tank base is done using rigid approach; under rigid approach, we assume that the base and the structure are stiff enough to span over the weaknesses and complexities of the supporting soil. The base is treated as a slab subjected to uniformly distributed pressure load at the base. It can be analysed as a two-way slab under the relevant load cases and combination. The junction between the base slab and the wall is subjected to unbalanced moment, but in this analysis, the maximum moment between the two elements is taken to design the top and bottom reinforcement for the base slab. The inner span is assumed to act as a plate that is continuous at all edges. The bending moments and shear forces are calculated based on equations in section 2.2.3 (top slab). Figure 2.6 shows bending moment distribution in the walls and base of the tank.

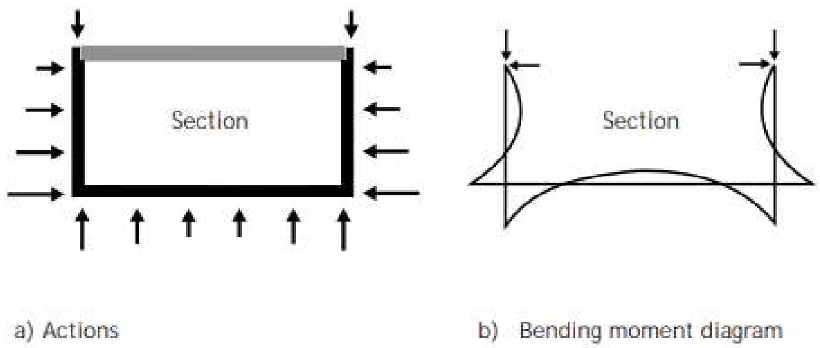

Figure 2. 6. Typical bending moment distribution in the walls and base of a tank (Narayanan and Goodchild ,2012)

In case the obtained span ratio does not have any corresponding bending moment or shear coefficients, an interpolation is made using equation so as to obtain the required coefficients

$$f(x) = f(x_0) + \frac{f(x_1) - f(x_0)}{x_1 - x_0}(x - x_0) \qquad 2.23$$

2.2.4.5. Design for serviceability limit states

The Serviceability Limit State of cracking will often govern the amount of reinforcement provided in underground water tank. In particular, it is necessary to verify the control of cracking due to restraint to early thermal and longer-term shrinkage movements. Deflection is usually not very serious, but either way, it must be within acceptable limits (Narayanan and Goodchild ,2012).

a. Cracking of reinforced concrete sections

Cracking is normal in reinforcement concrete structures subject to bending or shear resulting from either direct loading or restraint to imposed deformations (EN 1992 1-1, clause 7.3.1). The tensile stress in the concrete must be transferred to the steel if cracking must be controlled. To achieve this, a minimum amount of reinforcement must be provided in order to have small cracks occurring at intervals instead of having one single large crack.

i. Minimum reinforcement

Minimum reinforcement is provided to ensure that yielding does not occur, and by so doing, cracking is adequately controlled in a concrete section. This calculation is to obtain the minimum area of steel that is required to prevent early thermal cracking in the section. The minimum area of reinforcement required in BS EN 1992-1-1 and is given by equation 2.24.

$$A_{s,min} = k_c k A_{ct}\left(f_{ct,eff}/f_{yk}\right) \qquad 2.24$$

Where:

-k_c is the coefficient to account for stress distribution (1.0 for pure tension and 0.4 for pure bending);

-k is the coefficient to account for self-equilibrating stresses (1.0 for thickness $h < 300\,mm$ and 0.65 $h > 800\,mm$);

-A_{ct} is the area of concrete in the tension zone just prior to onset of cracking based on full thickness of the section;

-$f_{ct,eff} = f_{ctm}$ the mean tensile strength when cracking may be first expected to occur;

-f_{yk} is the characteristic yield strength of the reinforcement.

b. Crack width calculations

Crack control in water retaining structures is verified by carrying out direct calculation of the crack widths and compliance with the stated limits. According to expression 7.8 of EN 1992-1-1, crack width w_k is given by equation 2.25.

$$w_k = S_{r,max}\varepsilon_{cr} \qquad 2.25$$

where:

-$S_{r,max}$ is the maximum crack spacing;

--ε_{cr} is the crack-inducing strain in concrete.

i. The maximum crack spacing

The maximum crack spacing is given in the equation 2.26.

$$S_{r,max} = 3.4\,C + 0.425\left(k_1 k_2 \phi \frac{A_{c,eff}}{A_S}\right) \qquad 2.26$$

Where:

-$S_{r,max}$ is the maximum crack spacing;

-C is the nominal cover in mm in accordance with BS EN 1992-1;

-$k_1 = 0.8$ for high bond bars;

-$k_2 = 1.0$ for tension (from restraint) and 0.5 for bending;

-ϕ is the diameter of the bar in mm;

-A_S is the cross-sectional area of reinforcement;

-$A_{c,eff}$ is the effective area of concrete surrounding reinforcement given in equation 2.27

The effective area of concrete surrounding reinforcement is defined by:

$$A_{c,eff} = b \times min[0.5h; 2.5(C + 0.5\phi); (h - X)/3] \qquad 2.27$$

Where:

-b is the thickness of section;

-h is the height of section;

-X depth to neutral axis.

ii. Crack inducing strain

The crack inducing strain is derived according whether the element is subject to edge restraint (early thermal effects or long-term effects), end restraint and flexure and applied tension.

- **Crack inducing strain due to edge restraint and early thermal effects**

At the early age of freshly poured concrete (within 3 days), the crack inducing strain due to edge restraint in concrete element is given in equation and it as follows:

$$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}]R_1 - 0.5\varepsilon_{ctu} \qquad 2.28$$

Where:

-K is the allowance for creep;

-α_c is the coefficient of thermal expansion (See table A1.3 in annex 1);

-T_1 is the difference between the peak temperature of concrete during hydration and ambient temperature, $°C$ (See table A1.4 in annex 1);

-ε_{ca} is the autogenous shrinkage strain (See table A1.5 in annex 1);

-R_1 is the restraint factor for the short-term thermal situation;

-ε_{ctu} is the tensile strain capacity of the concrete (See table A1.6 in annex 1).

- **Crack inducing strain due edge restraint and long-term effects**

It is given by the equation 2.29.

$$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca})R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\,\varepsilon_{ctu} \qquad 2.29$$

Where:

-ε_{ca} is the autogenous shrinkage strain;

-T_2 is the long-term drop-in temperature after concreting, $°C;$

-R_1, R_2, R_3 are the restraint factors for short-term, long-term thermal and long-term drying situations (See table A1.7 in annex 1);

-ε_{cd} is the drying shrinkage strain (See table A1.8 in annex 1);

-ε_{ctu} is the tensile strain capacity of the concrete.

- **Crack inducing strain due end restraint**

It is given in BS EN 1992-3 EXP. (M.1) and it is as follows

$$\varepsilon_{cr} = 0.5\alpha_e k_c k f_{ct,eff}[1 + (1/\alpha_e \rho)]/E_s \qquad 2.30$$

Where:

-k_c is the coefficient to account for stress distribution;

-k is the coefficient to account for self-equilibrating stresses;

-$f_{ct,eff} = f_{ctm}$ is the mean tensile strength;

-α_e is the modular ratio defined in equation 2.31;

-ρ is the ratio of total area of reinforcement to the gross section in tension;

-E_s is the modulus of elasticity of reinforcing steel;

The modular ratio is defined by:

$$\alpha_e = E_s/E_c \qquad 2.31$$

iii. Crack inducing strain due flexure and applied tension

The crack inducing strain due to flexure and applied tensile force is given in expression 7.9 of BS EN 1992-1-1 and it is as follows

$$\varepsilon_{cr} = (\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - k_t\left(\frac{f_{ct,eff}}{\rho_{p,eff}}\right)\left(1+\alpha_e\frac{A_s}{A_{c,eff}}\right)}{E_s} \geq \frac{0.6\,\sigma_s}{E_s} \qquad 2.32$$

Where:

-ε_{sm} is the mean strain in reinforcement;

-ε_{cm} is the mean strain in the concrete between cracks;

-σ_s is the stress in the reinforcement based on cracked section properties under quasi permanent load combination;

-$k_t = 0.6$ is the for short term loading and 0.4 for long term loading;

-0.6 is the factor that limits the effect of tension stiffening.

2.3. Parametric study

To observe the effect of different type of soils on the underground tank, the calculations of lateral earth pressure are made. The resulting maximum bending moments are obtained and the reinforcement for design given. The parameters of soil that affect the solicitation on the underground tank are unit weight of soil (γ) and angle of internal friction (φ). Based on the given formulas throughout, an excel spreadsheet is generated so as to study the bending moments and their corresponding area of steel reinforcement required on the walls and base slab of the tank.

2.3.1. Varying unit weight of soil

This done by fixing the value of angle of internal friction at 27°**,** then varying unit of weight of soil between values that ranges from 16-22.

2.3.2. Varying angle of internal friction

This done by fixing the value of unit of weight of soil at 19.5 KNm**,** then varying the angle of internal friction between values that ranges from 0-44.

Conclusion

The aim of this chapter was to present a methodological approach for the design of an underground water tank. The study was divided into several sections, the first was the conception of the case study through design hypotheses based on existing standards and specifications. In addition, a procedure for the calculation of load actions was detailed. After that, a solely focus was made at the design and verification under limit states where the crack width calculation is detailed. The following chapter will present the results of this methodology applied in the conceived case study.

CHAPTER 3.PRESENTATION AN INTERPRETATION OF RESULTS

Introduction

This chapter consists of a description of the conceived case study, the materials used and the actions and combinations of actions considered. This will be followed by the presentation of a static analysis results of the case study at different limit state and verifications of the structural elements. After that, the reinforcement plan of each structural element is presented. Finally, a parametric study is made so as to understand the effect of different type of soils on the tank.

3.1. Conception of the case study

The conception of the case study consists of the brief description, presentation of materials properties and concrete cover used, Actions and load combinations considered

3.1.1. The capacity of the tank

The tank is designed to meet up the fire water requirements for the risks associated to the type of building open to the public. According to Table 2.1, the need in water for external fire fighting for class 1 type and for the surface area less or equal to 500 m² of the building reported is 60m^3/h and this for a minimum of 2 hours. Consequently, the volume of water needed as reserve is at least 120 m^3. The tank is supplied with water from the public water network. Therefore, an underground rectangular tank of sides (6 m × 5 m × 4 m), presented in table 3.1 is designed and has an access opening of 750 mm × 750 mm.The thickness of top slab, walls and base slab is assumed and presented in table 3.2.

Table 3. 1. Preliminary tank dimensions

Depth of tank (in to in)	Length (in to in)	Width (in to in)
4 m	6 m	5 m

Table 3. 2. Thickness of structural elements

Structural elements	Thickness(mm)
Top slab	200
Walls and base slab	400

3.1.2. Design Parameters

The design of a rectangular water tank that is built under the ground subjected to a surcharge of 10 KN/m^2 at the surface.

- Soil type: uniform sand
- Unit weight,$\gamma = 19.5KN/m^3$
- Submerged unit weight $\gamma' = \gamma - \gamma_w = 19.5 - 10 = 9.5\ KN/m^3$
- Angle of internal friction (design value) $\varphi = 27°$
- Ground water exists at 1m below natural ground surface of the site

3.1.2.1. Check for uplift when tank is empty

This consist of calculating and comparing stabilizing and destabilizing actions. In case the latter is greater than the former, a remedial solution is proposed so as to avoid the flotation of the tank.

a. Stabilizing action

- Weight of base = $(6.8\ \times 5.8 \times 0.4) \times 25 = 394.4$ KN
- Weight of top slab = $(6.8\ \times 5.8 \times 0.2) \times 25 = 197.2$ KN
- Weight of longitudinal wall = $2(6.8\ \times 5 \times 0.4) \times 25 = 680\ KN$
- Weight of lateral wall= $2(5 \times 4 \times 0.4) \times 25 = 400$ KN
- Total weight, $\boldsymbol{W_T} = 394.4 + 197.2 + 680 + 400 = 1671.6\ KN$

As this is a favourable load, the stabilizing action at ULS is

$$\boldsymbol{G_{stb,d}} = \gamma_{G,inf}.W_T = 0.9 \times 1671.6 = \mathbf{1504.44\ KN}$$

b. Destabilizing action

Uplift pressure of water under the base due to 4.4m head water (assuming water table at ground surface) is calculated as follows:

- Uplift pressure = $10 \times 4.4 = 44KN/m^2$
- Destabilizing uplift action at ULS is $\boldsymbol{U_{dst,d}} = 1.1 \times 6.8 \times 5.8 \times 44 = \mathbf{1908.9\ \boldsymbol{KN}}$
- The stabilizing actions and the destabilizing uplift actions are compared $U_{dst,d} > G_{stb,d}$ **Not OK**

In order to solve this, the base of the tank is extended by projecting it beyond the walls of the tank and its width(b) calculated. The following is done to get the results:

- Additional force required for stability $= U_{dst,d} - G_{stb,d} = 1908.9 - 1504 = 404.46$ KN
- The submerged unit weight of soil = 9.5 KN/m^3
- Pressure due to 4m high submerged soil = 4×9.5=$38\ KN/m^2$
- submerged weight of slab = $(25 - 10) \times 0.4 = 6\ KN/m^2$
- $\left[\left((6.8 + 2b) \times (5.8 + 2b) - 6.8 \times 5.8\right) \times (38 + 6)\right] \times 0.9 = 404.46$ KN
- $b = 0.382$ m
- Therefore, the base extension(b) can be estimated to $\mathbf{0.4}\ \boldsymbol{m}$

The adopted design dimension in meter of the underground tank is presented in figure 3.1,3.2 and 3.3.

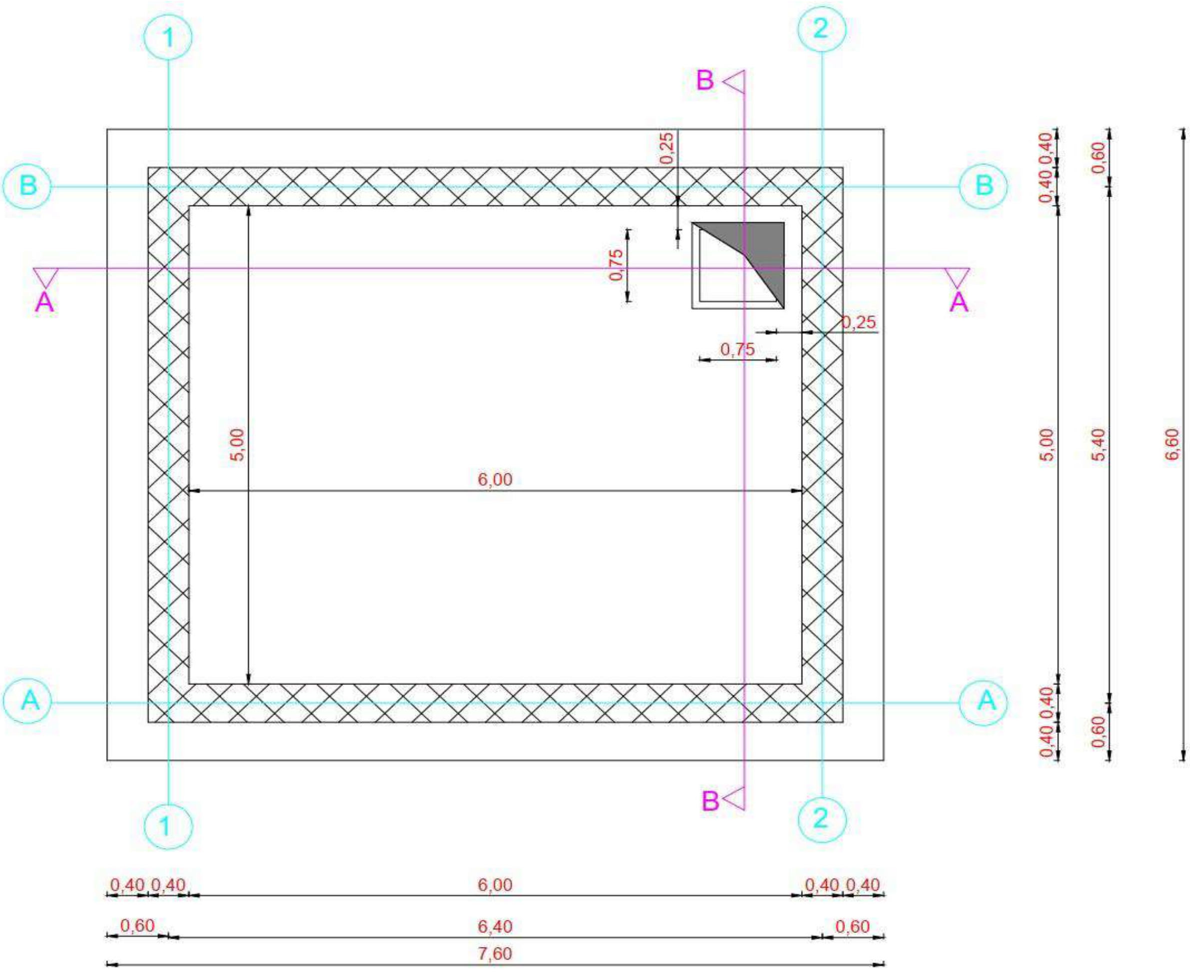

Figure 3. 1. Plan view of the underground rectangular tank

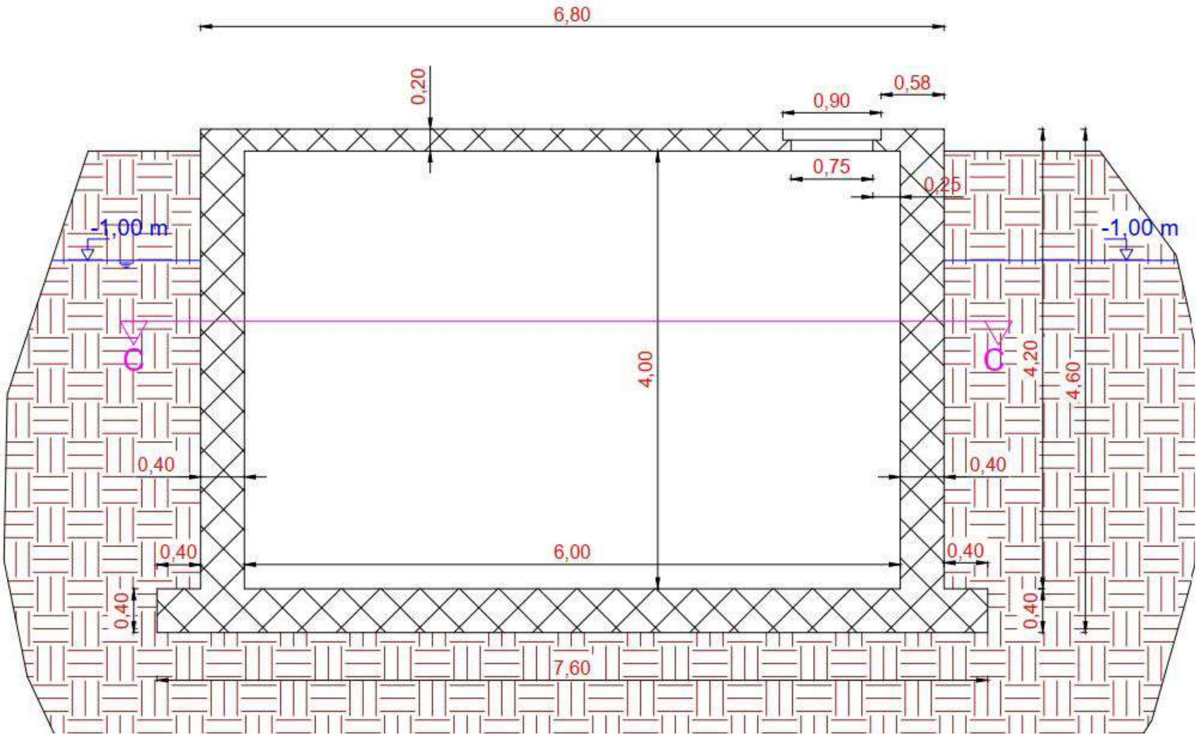

Figure 3. 2. Cut B-B (Longitudinal view of the underground rectangular tank)

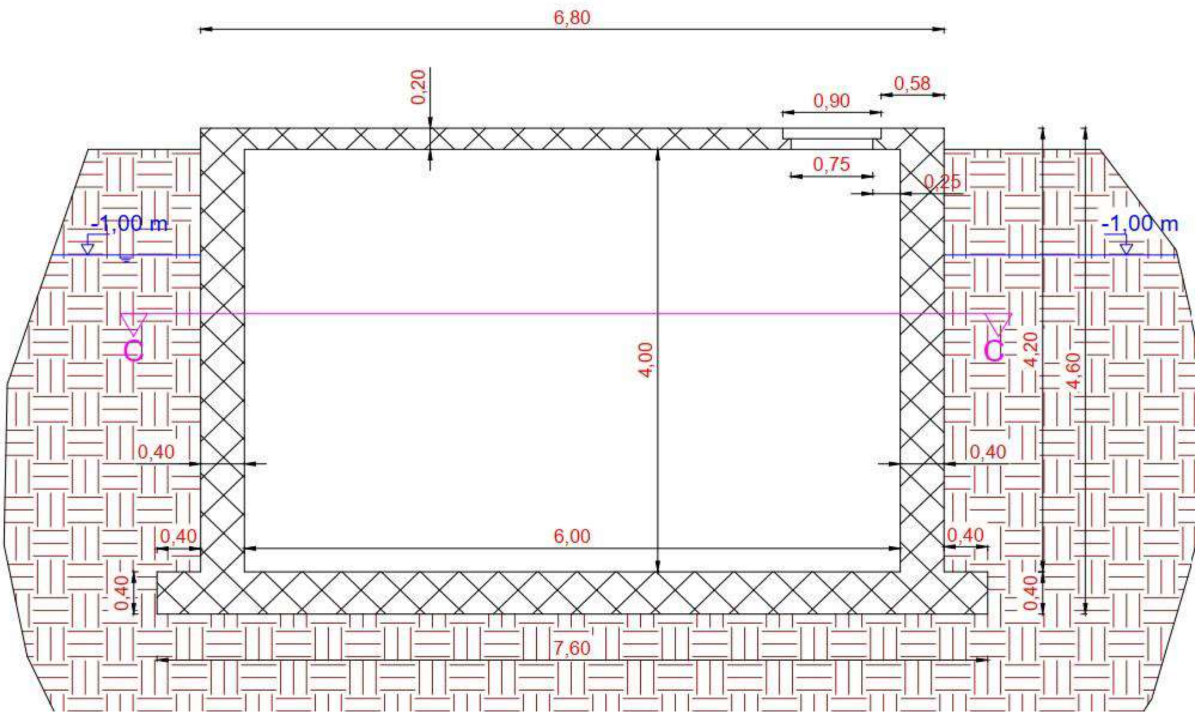

Figure 3. 3. Cut A-A (Lateral view of the underground rectangular tank)

3.1.2.2. Materials properties

In order to comply with the requirements prescribed for underground tank the concrete class chosen is C30/37 dosed at 400 kg/m^3 in order to have a good tightness. the longitudinal steel reinforcement is the B500C. The main characteristics of these materials defined for linear analysis and structural design come from equation 2.1 to 2.4 and are given in table 3.3. for the concrete and table 3.4. for the steel reinforcement.

Table 3. 3. Concrete characteristics

Property	Value	Unit	Description
Class	C30/37	-	Concrete class
f_{ck}	30	N/mm²	Characteristic compressive strength of concrete at 28 days
f_{cm}	38	N/mm²	Mean value of concrete cylinder compressive strength
γ_c	1.5	-	Partial factor for concrete
f_{ctm}	2.89	N/mm²	Mean value of axial tensile strength of concrete
f_{ctd}	1.35	N/mm²	Design resistance in traction
E_{cm}	32837	N/mm²	Secant modulus of elasticity
υ	0.2	-	Poisson's ratio
γ	25	kN/m^3	Specific weight of the concrete

Table 3. 4. Steel reinforcement characteristics

Property	Value	Unit	Description
Class	B500C	-	Steel class
f_{yk}	500	N/mm²	Characteristic yield strength
γ_s	1.15	-	Partial safety factor for steel
γ	78.5	kN/m^3	Specific weight of steel
υ	0.3	-	Poisson ratio
n	6.09	-	Steel-concrete Homogenization coefficient

3.1.2.3. Concrete cover

The walls and base slab of the underground tank are exposed to severe conditions. A big concrete cover is needed compared to the top slab so as to ensure that moisture and air do not cause carbonation in the concrete cover to reinforcement. So, for the top slab structural class S3 and exposure class XC2/XC3 are taken. For the walls and base slab, structural class S4 and exposure class XD2/XS2 are taken. The design working life of the structure is 50 years. Equations 2.5 and 2.6 permits to obtain the value of concrete cover as following:

a. For top slab

C_{min}= max {20mm ;20mm ;10mm} = 20 mm

C_{nom} = 20 mm + 10 mm= 30 mm

b. For the walls and base slab

C_{min}= max {20mm ;40mm ;10mm} = 40 mm

C_{nom} = 40 mm + 10 mm= 50 mm

3.2. Static design

The results below come from hand calculation of each component of the underground tank at different limit state ULS and SLS.It concerns the top slab, walls and base slab. From the procedure explained in section 2.2; the solicitations, reinforcement and verifications are presented.

3.2.1. Top slab

The top slab with a thickness of 200 mm of the water tank should be able to resist a characteristic variable action (q_k) of 4KN/m^2(assumed) and characteristic permanent action (g_k). It is analysed and designed as a two-way slab with all edges considered discontinuous.

3.2.1.1. Calculation of actions on the top slab

- self-weight of the slab =$0.2 \times 25 = 5\ KN/m^2$
- self-weight of tiles and screed at the suffit of the tank =1 KN/m^2
- $\mathrm{g_k} = 5 + 1 = 6\ \mathrm{KN/m^2}$
- At ULS: $\mathrm{P_{Ed}} = 1.35\mathrm{g_k} + 1.5\mathrm{q_k} = \mathbf{14.1\ KN/m^2}$

3.2.1.2. Analysis of short span

From the span ratio, coefficients for bending moment and shear forces are determined as follows;

- $\text{Span ratio} = \frac{\text{long span}}{\text{short span}} = \frac{6.4}{5.4} = 1.2$
- Bending moment coefficient; $\beta_{\mathrm{sx}} = 0.074$
- Shear force coefficient, $\beta_{\mathrm{vx}} = 0.39$

a. Calculation of sollicitations

The bending moment and shear force are calculated as follows:

- $\boldsymbol{M_{Ed}} = 0.074\ \times 14.1 \times 5.4^2 = \mathbf{30.426\ KNm}$
- $\mathbf{V_{Ed}} = 0.39 \times 14.1 \times 5.4 = \mathbf{29.69kN/m}$

3.2.1.3. Analysis of long span

From the span ratio, coefficients for bending moment and shear forces are determined as follows;

- Span ratio $= \frac{\text{long span}}{\text{short span}} = \frac{6.4}{5.4} = 1.2$
- Bending moment coefficient; $\beta_{sy} = 0.056$
- Shear force coefficient, $\beta_{vy} = 0.33$

a. Calculation of sollicitations

The bending moment and shear force are calculated as follows;

- $M_{Ed} = 0.056 \times 14.1 \times 5.4^2 = \mathbf{23.05\ KNm}$
- $V_{Ed} = 0.33 \times 14.1 \times 5.4 = \mathbf{25.13\ kN}$

Table 3. 5. Summary of solicitations on top slab at ULS

Sollicitation	M_{Ed}(KNm)	V_{Ed}(kN)
Short span	30.426	29.69kN
Long span	23.05	25.13

3.2.1.4. Reinforcement of the top slab

The reinforcement of the top slab is done for the short span and long span.

a. Flexure design of short span

- $M_{Ed} = 30.426$ KNm
- Assuming ϕ 12 mm bars will be used and $b = 1000$ mm(designing per unit width)
- $d = h - c_{nom} - \frac{\phi}{2} = 200 - 30 - \frac{12}{2} = 164$ mm
- $K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{30.426\times10^6}{30\times1000\times164^2} = 0.0377$
- Since $K < 0.167$ no compression moment reinforcement required
- $Z=d\left[0.5 + \sqrt{(0.25 - 0.882K)}\right]$ =0.97d
- $A_{sreq} = \frac{M_{Ed}}{0.87f_{yk}Z} = 439.68\text{mm}^2/\text{m}$

Provide HA12 at 250 mm spacing, $A_{sprov} = \mathbf{452}\ \boldsymbol{mm^2/m}$

$A_{smin} = 0.26\frac{f_{ctm}}{f_{yk}}bd = 246.97\text{mm}^2/\text{m}$, where $f_{ctm} = 0.3f_{ck}^{2/3} = 2.896Mpa$

$\mathbf{A_{sprov}} > A_{smin}$ so, the provided reinforcement is adequate.

b. Flexure design of long span

- $M_{Ed} = 23.025$ KNm
- d = 164 mm
- $K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{23.025\times10^6}{30\times1000\times164^2} = 0.0285$
- Since K < 0.167 no compression moment reinforcement required
- $Z = d\left[0.5 + \sqrt{(0.25 - 0.882K)}\right]$ =0.97d
- $A_{sreq} = \frac{M_{Ed}}{0.87f_{yk}Z} = 332.75\ mm^2/m$

Provide HA12 at 300 mm spacing, $A_{sprov} = 377\ mm^2/m$

$A_{sprov} > A_{smin}$ so, the provided reinforcement is adequate.

3.2.1.5. Verifications

The verifications of the top slab are done for the short span and long span.

a. For short span

i. Check for deflection

- k = 1 for discontinuous slab
- $\rho = \frac{A_{prov}}{bd} = \frac{452}{1000\times164} = 2.756\times10^{-3}$
- $\rho_0 = 10^{-3}\sqrt{f_{ck}} = 10^{-3}\sqrt{30} = 5.48\times10^{-3}$
- Since $\rho < \rho_0$; $\frac{L}{d} = k\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}(\frac{\rho_0}{\rho} - 1)^{3/2}\right]$

$$= 1\left[11 + 1.5\sqrt{30}\frac{5.48}{2.756} + 3.2\sqrt{30}(\frac{5.48}{2.756} - 1)^{3/2}\right] = 44.56$$

- Modification factor, $\beta_s = \frac{310}{\sigma_s}$
- $\sigma_s = \frac{310f_{yk}}{500A_{sprov}} = \frac{310\times500\times439.68}{500\times452} = 301.55\text{N/mm}^2$
- So $\beta_s = \frac{310}{301.55} = 1.03 < 2$ **verified**

Taking the distance between center of support as effective span the allowable deflection and actual deflection are compared.

The allowable deflection = $\beta_s \times 44.56 = 1.03 \times 44.56 = 45.897$

Actual deflection, $\frac{L}{d} = \frac{5400}{164} = 32.93 < 45.897$ so the deflection is **OK**

ii. **Shear verification**

- $\mathbf{V_{Ed} = 29.69\ kN}$
- $V_{Rdc} = \max(V_{Rdc1}, V_{Rdc2})$
- $V_{Rdc1} = \left[C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d = 79.63\ kN$
- $C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$
- $k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{164}} = 2.1 > 2$ so, k = 2
- $\rho_1 = \frac{A_{sprov}}{bd} = \frac{452}{1000\times164} = 2.76\times10^{-3} < 0.2$
- $k_1 = 0.15$
- $\sigma_{cp} = \frac{N_{Ed}}{A_c} = 0$
- $V_{Rdc2} = (V_{min} + K_1\sigma_{cp})b_w d = 88.89\ kN$
- $V_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.0352^{3/2}30^{1/2} = 0.542 N/mm^2$
- $\boldsymbol{V_{Rdc}} = \max(V_{Rdc1}, V_{Rdc2}) = \mathbf{88.89\mathit{kN}}$
- Since $V_{Rdc} > V_{Ed}$ no shear reinforcement is required.

b. For long span

i. **Check for deflection**

- $\rho = \frac{\text{Asprov}}{bd} = \frac{377}{1000x164} = 2.3\times10^{-3}$
- $\rho_0 = 5.48x10^{-3}$
- Since $\rho < \rho_0$; $\frac{L}{d} = 1\left[11 + 1.5\sqrt{30}\frac{5.48}{2.48} + 3.2\sqrt{30}(\frac{5.48}{2.3} - 1)^{3/2}\right]$ =59
- $\sigma_s = \frac{310x500x332.75}{500x377} = 273.61\ N/mm^2$
- $\beta_s = \frac{310}{273.61} = 1.133 < 2$ **verified**

Taking the distance between center of support as effective span the allowable deflection and actual deflection are compared.

The allowable deflection = $\beta_s \times 44.56 = 1.133 \times 59 = 66.85$

Actual deflection, $\frac{L}{d} = \frac{5400}{164} = 39.024 < 66.85$ so the deflection is **OK**

ii. **Shear verification**

- $\boldsymbol{V_{Ed} = 25.13KN}$
- $V_{Rdc1} = \left[0.12 \times 2(100 \times 2.29 \times 10^{-3} \times 30)^{1/3}\right] 1000 \times 164 = 74.92kN$
- $\rho_1 = \frac{377}{1000\times164} = 2.299 \times 10^{-3} < 0.2$
- $V_{Rdc2} = 88.89KN$
- $\boldsymbol{V_{Rdc}} = \max(V_{Rdc1}, V_{Rdc2}) = \mathbf{88.89kN}$
- Since $V_{Rdc} > V_{Ed}$ no shear reinforcement is required

3.2.2. Walls of the tank

The trapezoidal loads due to earth pressure, water pressure and surcharge is analysed. The needed reinforcement is given and all the necessary verifications carried out.

3.2.2.1. Pressure calculation on the walls of the tank

Earth pressure coefficient using Rankine's theory

- $K_a = \frac{1-sin\varphi}{1+sin\varphi} = \frac{1-sin27}{1+sin27} = 0.3755$
- $K_P = \frac{1+sin\varphi}{1-sin\varphi} = \frac{1+sin27}{1-sin27} = 2.663$

In order to get the critical load action acting on the walls of the tank, different load cases are brought up and their corresponding actions both at ULS and SLS.It is as follows.

a. Load case 1: Tank is empty, ground water at ground surface level

Pressure due to surcharge,P_s(this is uniformly distributed on the wall)

$P_s = K_a q_k = 0.376 \times 10 = 3.76\ KN/m^2$

- **At the ground surface**

Lateral pressure is equal to pressure due to surcharge

At ULS: $\mathbf{P_{Ed,1}} = 1.5 \times 3.76 = \mathbf{5.64\ kN/m^2}$

At SLS: $\mathbf{P_1} = \mathbf{3.76kN/m^2}$

- **At 4.2m below the ground surface**

The lateral pressure is given by the sum of the pressures due to surcharge, submerged earth fill and ground water.

At ULS: $\mathbf{P_{Ed,2}} = 1.5(3.76) + 1.35(0.3755 \times 4.2) + 1.2(10 \times 4.2) = \mathbf{76.27\ kN/m^2}$

At SLS: $\mathbf{P_2} = 3.76 + (0.3755 \times 9.5 \times 4.2) + (10 \times 4.2) = \mathbf{60.79\ kN/m^2}$

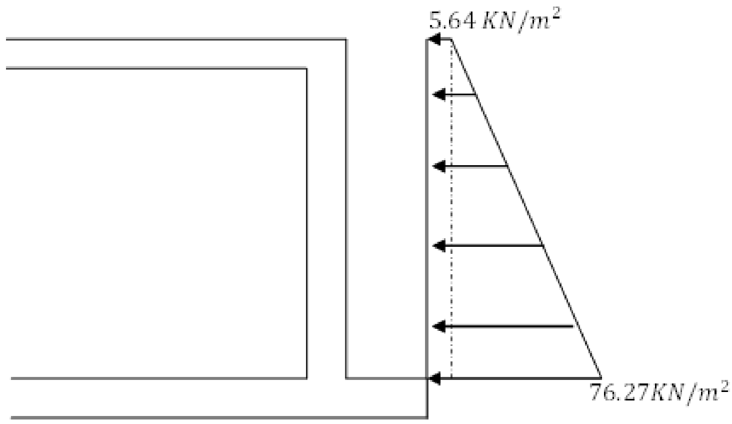

Figure 3. 4. Load case 1-Pressure distribution at ULS

b. Load case 2: Tank is empty, ground water below base level

$P_s = 3.76\ \mathrm{kN/m^2}$

- **At the ground surface**

Lateral pressure is equal to pressure due to surcharge

At ULS: $P_{Ed,1} = 1.5 \times 3.76 = \mathbf{5.64\ kN/m^2}$

At SLS: $P_1 = 3.76\ \mathrm{kN/m^2}$

- **At 4.2m below the ground surface**

Lateral pressure is given by the sum of the pressures due to surcharge pressure and earth fill.

At ULS: $P_{Ed,2} = 1.5(3.76) + 1.35(03755 \times 19.5 \times 4.2) = \mathbf{47.1422\ \mathit{kN/m^2}}$

At SLS: $P_2 = 3.76 + (0.3755 \times 19.5 \times 4.2) = 34.5\ \mathrm{kN/m^2}$

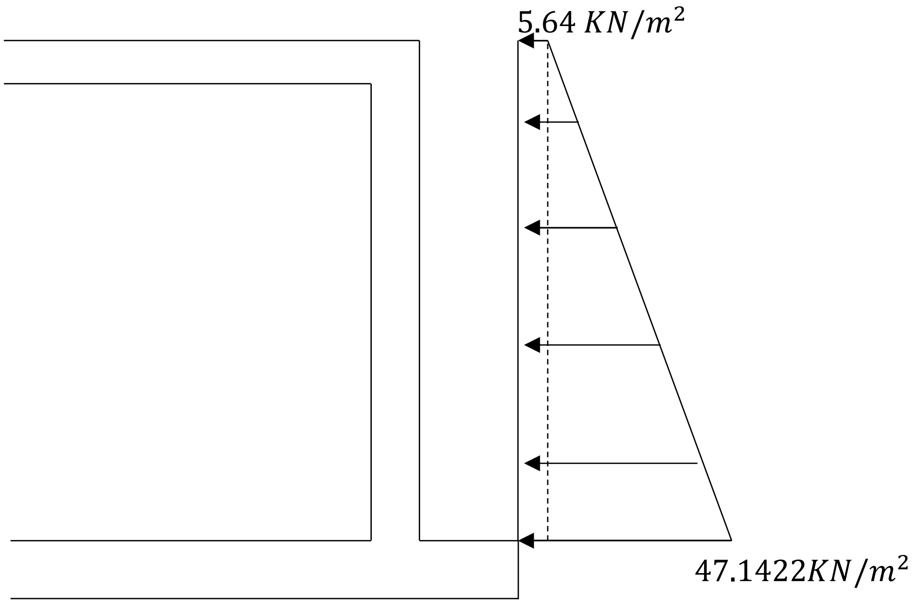

Figure 3. 5. Load case 2-Pressure distribution at ULS

c. Load case 3: Tank is full ignoring earth pressure and ground water

At ULS: $P_{Ed,1} = 1.2(10 \times 4) = 48\ \mathrm{kN/m^2}$

At SLS: $P_1 = 10 \times 4 = 40\ \mathrm{kN/m^2}$

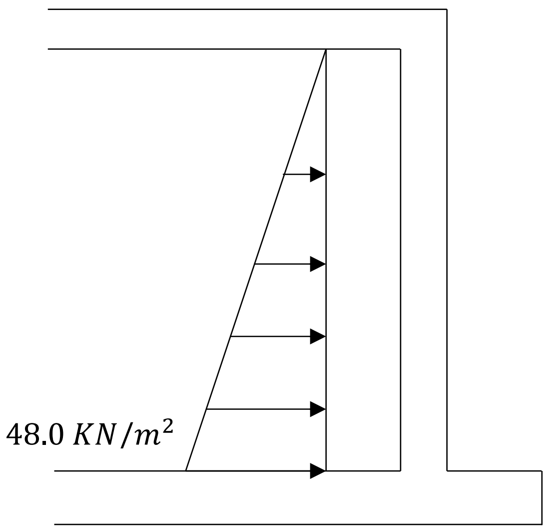

Figure 3. 6. Load case 3-Pressure distribution at ULS

It results that the critical load action is load case 1and it is as follows:

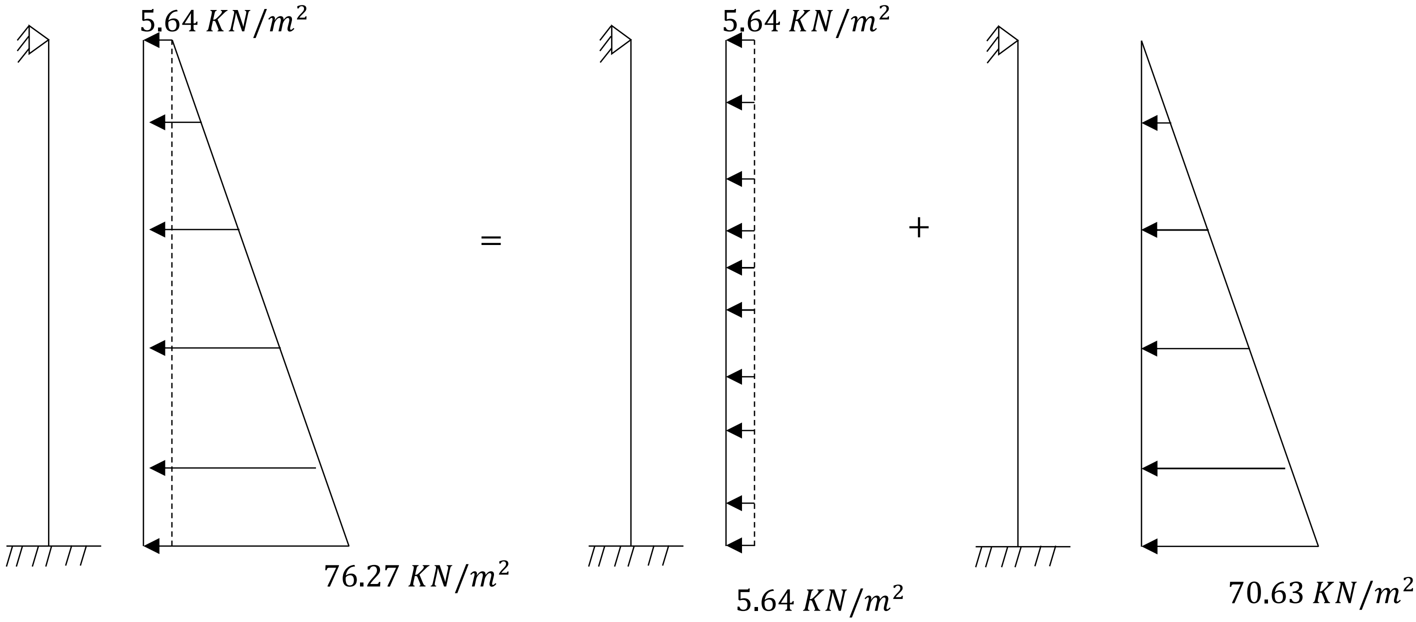

Figure 3. 7. Trapezoidal load distribution on the walls at ULS

3.2.2.2. Longitudinal wall

It is considered as a two-way slab and one long edge discontinuous

$$l_x / l_z = 6.4/4.3 = 1.5$$

a. Calculation of solicitations in the short span (l_z)

i. Bending moment

Negative moment at base = $(0.073 \times 5.64 \times 4.3^2) + (0.0485 \times 70.63 \times 4.3^2)$

$= 70.95$ KNm

Positive moment at mid span= $(0.055 \times 5.64 \times 4.3^2) + (0.0185 \times 70.63 \times 4.3^2)$

$= 29.896$ KNm

$\mathbf{M_{Ed}} = \max(70.95 \text{ KNm}; 29.896 \text{ KNm}) = \mathbf{70.95\ KNm}$

ii. Vertical shear force at base

$\mathbf{V_{Ed}} = (0.51 \times 5.64 \times 4.3) + (0.38 \times 70.63 \times 4.3) = \mathbf{127.77\ KN}$

b. Calculation of solicitations in the long span (l_x)

i. Bending moment

Negative moment at continuous edge = $(0.037 \times 5.64 \times 6.4^2) + (0.033 \times 70.63 \times 4.3^2)$

= 51.64 KNm

Positive moment at mid span = $(0.028 \times 5.64 \times 6.4^2) + (0.011 \times 70.63 \times 4.3^2)$

=20.83 KNm

$\mathbf{M_{Ed}} = \max(51.64\ \text{KNm}; 20.83\ \text{KNm}) = \mathbf{51.64\ KNm}$

ii. Horizontal shear force at the edge

$\mathbf{V_{Ed}} = (0.36 \times 5.64 \times 4.3) + (0.26 \times 70.63 \times 4.3) = \mathbf{87.7\ KN}$

Table 3. 6. Summary of maximum solicitations on longitudinal wall at ULS

Sollicitation	M_{Ed}(KNm)	V_{Ed}(kN)
Short span	70.95	127.77
Long span	51.64	87.7

3.2.2.3. Lateral wall

It is considered as a two-way slab and one long edge discontinuous

$$l_x / l_z = 5.4/4.3 = 1.26$$

a. Calculation of solicitations in the short span (l_z)

i. Bending moment

Negative moment at base = $(0.0567 \times 5.64 \times 4.3^2) + (0.042 \times 70.63 \times 4.3^2)$

= 60.76 KNm

Positive moment at mid span= $(0.043 \times 5.64 \times 4.3^2) + (0.0149 \times 70.63 \times 4.3^2)$

= 23.94 KNm

$\mathbf{M_{Ed}} = \max(60.76\ \text{KNm}; 23.94\ \text{KNm}) = \mathbf{60.76\ KNm}$

ii. Vertical shear force at base

$\mathbf{V_{Ed}} = (0.458 \times 5.64 \times 4.3) + (0.36 \times 70.63 \times 4.3) = \mathbf{120.44\ KN}$

b. Calculation of solicitations in the long span (l_x)

i. Bending moment

Negative moment at continuous edge = $(0.037 \times 5.64 \times 5.4^2) + (0.031 \times 70.63 \times 4.3^2)$

= 46.57 KNm

Positive moment at mid span = $(0.028 \times 5.64 \times 5.4^2) + (0.011 \times 70.63 \times 4.3^2)$

=18.97 KNm

$\mathbf{M_{Ed}} = \max(46.57\ \text{KNm}; 18.97\ \text{KNm}) = \mathbf{46.57\ KNm}$

ii. Horizontal shear force at the edge

$\mathbf{V_{Ed}} = (0.36 \times 5.64 \times 4.3) + (0.25 \times 70.63 \times 4.3) = \mathbf{84.66\ KN}$

Table 3. 7. Summary of maximum solicitations on lateral wall at ULS

Sollicitation	M_{Ed}(KNm)	V_{Ed}(kN)
Short span	60.76	120.44
Long span	46.57	84.66

Following the analysis of both longitudinal and lateral wall, it can be seen that there is no huge difference in solicitations amongst the walls. The maximum between them is taken and used for the design of walls. They are shown in Table 3.8

Table 3. 8. Summary of maximum solicitations on walls

Solicitation	M_{Ed}(KNm)	V_{Ed}(kN)
Short span	70.95	127.77
Long span	51.64	87.7

3.2.2.4. Reinforcement of the walls

The reinforcement of the walls is done for the short span and long span.

a. Flexure design of short span

- $M_{Ed} = 70.95$ KNm
- Assuming ϕ 12 mm bars will be used and $b = 1000$ mm(designing per unit width)
- $d = h - c_{nom} - \frac{\phi}{2} = 400 - 50 - \frac{12}{2} = 344$ mm
- $K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{70.95 \times 10^6}{30 \times 1000 \times 344^2} = 0.02$
- Since $K < 0.167$ no compression moment reinforcement required
- $Z = d\left[0.5 + \sqrt{(0.25 - 0.882K)}\right] = 0.98d$
- $A_{sreq} = \frac{M_{Ed}}{0.87 f_{yk} Z} = 483.81\ \mathrm{mm^2/m}$

Provide HA12 at 200 mm spacing, $A_{sprov} = 566\ mm^2/m$ both faces in the vertical direction

$\mathbf{A_{sprov}} > A_{smin}$ so, the provided reinforcement is adequate.

b. Flexure design of long span

- $M_{Ed} = 51.64$ KNm
- $K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{51.64\times10^6}{30\times1000\times344^2} = 0.015$
- Since $K < 0.167$ no compression moment reinforcement required
- $Z = d\left[0.5 + \sqrt{(0.25 - 0.882K)}\right] = 0.99d$
- $A_{sreq} = \frac{M_{Ed}}{0.87f_{yk}Z} = 348.58\ mm^2/m$

Provide HA12 at 250 mm spacing, $\mathbf{A_{sprov}} = 452\ mm^2/m$ both faces in the horizontal direction

$\mathbf{A_{sprov}} > A_{smin}$ so, the provided reinforcement is adequate.

c. Direct tension in walls

In load case, there is direct tension in the horizontal direct tension in the horizontal direction in the wall due to water pressure on the shorter walls from load case 3; $P_{Ed,1} = 48\ kN/m^2$.

The shear force at the horizontal edge due to water pressure on the shorter wall; which corresponds as well to the axial tension in the horizontal direction (N_{Ed}) is given by;

$V_{Ed} = 0.38 \times 48 \times 4.3 = 78.432\ kN$

Steel area to resist this force, $A_s = \frac{N_{Ed}}{0.87f_{yk}} = \frac{78.432\times10^3}{0.87\times500} = 180.3\ mm^2/m$

$A_s = \frac{N}{0.87.f_{yk}} = \frac{78.432}{0.87*500}*10^3 = 180.3\ mm^2/m.$

On each face = 180.3/2 = 90.15mm^2/m

$A_{sreq} = 90.15 + 34.58 = 438.73\ mm^2/m$

$\boxed{A_{sprov} \text{ is sufficient}}$

3.2.2.5. Verifications

The verifications on the walls of the tank are done as follows;

a. shear verification

i. Vertical shear force at base (short span, l_z)

- $V_{Ed} = 127.77$ KN
- Axial force from top slab, N_{Ed}= 29.69 KN
- $V_{Rdc} = \max(V_{Rdc1}, V_{Rdc2})$
- $V_{Rdc1} = \left[C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp}\right] b_w d = 127.79$ kN
- $C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$
- $k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{344}} = 1.762 < 2$ so, $k = 1.762$
- $\rho_1 = \frac{A_{sprov}}{bd} = \frac{566}{1000\times344} = 1.65 \times 10^{-3} < 0.2$
- $k_1 = 0.15$
- $\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{29.69\times10^3}{1000\times400} = 0.0742\ N/mm^2$
- $V_{Rdc2} = (V_{min} + K_1\sigma_{cp})b_w d = 98.1$ kN
- $V_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.0352^{3/2}30^{1/2} = 0.2744\ N/mm^2$
- $\boldsymbol{V_{Rdc}} = \max(V_{Rdc1}, V_{Rdc2}) = \mathbf{127.79\ KN}$

Since $V_{Rdc} > V_{Ed}$ no shear reinforcement is required.

ii. Horizontal shear force at the edge (Long span, l_x)

- $V_{Ed} = 87.7$ KN
- Axial force from direct tension in wall, N_{Ed}= 78.432 KN
- $V_{Rdc1} = \left[C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + k_1\sigma_{cp}\right] b_w d = 134.1$ kN
- $\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{78.432\times10^3}{1000\times400} = 0.0742\ N/mm^2$
- $V_{Rdc2} = (V_{min} + K_1\sigma_{cp})b_w d = 104.37$ kN
- $V_{min} = 0.035k^{3/2}f_{ck}^{1/2} = 0.0352^{3/2}30^{1/2} = 0.2744\ N/mm^2$
- $\boldsymbol{V_{Rdc}} = \max(V_{Rdc1}, V_{Rdc2}) = \mathbf{134.1\ KN}$

Since $V_{Rdc} > V_{Ed}$ no shear reinforcement is required

b. Thermal cracking

Design crack width $w_k = 0.2\,mm$

$w_k = S_{r,max}.\varepsilon_{cr}$

Maximum crack spacing,

$$S_{r,max} = 3.4C + 0.425\left(\frac{k_1 k_2}{\rho_{P,eff}}\right)$$

$\mathrm{k_1 = 0.8 \quad k_2 = 1 \quad C = 50\ mm \quad h = 400\ mm \quad b = 1000\ mm \quad A_{sprov} = 452\ mm^2/m}$

$$\rho_{P,eff} = \frac{\mathrm{A_{sprov}}}{A_{c,eff}} = \frac{452}{140000} = 3.23 \times 10^3$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\phi}{2}\right)\right] = 1000 \times 140 = 140000\ \mathrm{mm^2/m}$

$\Rightarrow S_{r,max} = 1433.16\ \mathrm{mm}$

i. Early age cracking

$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}]R_j - 0.5\varepsilon_{ctu}$

$\mathrm{K} = 0.65 \qquad \alpha_c = 10 \times 10^{-6} \qquad T_1 = 25^{\circ}C \qquad \varepsilon_{ca} = 15 \times 10^{-6} \qquad \varepsilon_{ctu} = 76 \times 10^{-6}$

$$R_j = \frac{1}{1 + \frac{A_n}{A_o}\frac{E_n}{E_o}} = \frac{1}{1 + 0.8 \times 0.5} = 0.714$$

- $\frac{E_n}{E_0} = 0.8 \qquad \frac{A_n}{A_0} = \frac{h_n}{2\,h_0} = \frac{0.4}{2 \times 0.4} = 0.5$

$\varepsilon_{cr} = 0.65[10 \times 10^{-6} \times 35 + 15 \times 10^{-6}] \times 0.714 - 0.5 \times 76 \times 10^{-6} = 1.314 \times 10^{-4}$

$w_k = 1433.16 \times\ 1.314 \times 10^{-4} = 0.188\ \mathrm{mm}$

$w_k < 0.2\,mm$ $\Rightarrow$ **OK**

ii. long term restraint cracking

$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca})R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\,\varepsilon_{ctu}$

$\varepsilon_{cd} = 150 \times 10^{-6};\ \ \varepsilon_{ca} = 50 \times 10^{-6}\ ;\ \varepsilon_{ctu} = 104 \times 10^{-6}$

$R_1 = R_2 = R_3 = 0.67$

$$\varepsilon_{cr} = 0.65[(10 \times 10^{-6} \times 35 + 50 \times 10^{-6})][(10 \times 10^{-6} \times 35 + 50 \times 10^{-6}) + (10 \times 10^{-6} \times 25 \times 0.67) + (150 \times 10^{-6} \times 0.67)] - 0.5(104 \times 10^{-6})$$

$$= 2.964 \times 10^{-6}$$

$$w_k = 1433.16 \times \ 2.964 \times 10^{-4} = 0.423 \text{ mm}$$

$w_k > 0.2\ mm$ **not ok**

Increase the area of steel to
HA16 at 100 mm spacing
$A_{sprov} = 2010\ mm^2/m$
Both faces in the horizontal direction

- $P_{p,eff} = \frac{2010}{1000 \times 40} = 0.01436$

$$S_{r,max} = 3.4 \times 50 + 0.425 \left(\frac{0.8 \times 1 \times 16}{0.01436}\right) = 548.83\ mm$$

$$w_k = 548.83 \times 2.964 \times 10^{-4}$$

$$w_k = 0.163\ mm$$

$w_k < 0.2\ mm \ \Longrightarrow$ **OK**

c. Cracking due to loading

Load case 1 is the critical one, the bending moment at SLS is given by;

$$M_{SLS} = (0.073 \times 3.76 * 4.3^2) + (0.0485 \times 56.88 \times 4.3^2) = 56.17 \text{ KNm}$$

$$\varepsilon_{cr} = (\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - k_t\left(\frac{f_{ct,eff}}{\rho_{p,eff}}\right)\left(1+\alpha_e\frac{A_s}{A_{c,eff}}\right)}{E_s} \geq \frac{0.6\,\sigma_s}{E_s}$$

$$\alpha_e = 7 \quad K_t = 0.4 \qquad f_{ct,eff} = f_{ctm} = 0.3\ f_{ck}^{2/3} = 0.3 \times 30^{2/3} = 2.89\ Mpa$$

σ_s: SLS stress in the reinforcement = f_s

$$\rho_{P,eff} = \frac{\mathrm{A_{sprov}}}{A_{c,eff}}$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\phi}{2}\right), \frac{(\mathrm{h-x})}{3}\right]$

The depth to neutral axis(x) is given by;

$$MA_{sprov}\alpha_e(d - x) - A_{sprov}\alpha_e(x - d')M = b\frac{x^2}{2}M \Leftrightarrow 566 \times 7 * (344 - x) - 566 * 7 * (x - 56) = \frac{1000x^2}{2}$$

$$\Leftrightarrow 3962\,(344 - x) - 3962\,(x - 56) = 500x^2$$

$$\Leftrightarrow 500\,x^2 + 7924\,x - 1584800 = 0$$

$$x = \frac{-7924 \pm\sqrt{7924^2+4*500*1584800}}{2*500} \Leftrightarrow \boldsymbol{x = 48.93}$$

Calculate the compressive stress in concrete

$$bx\frac{f_c}{2}\left(d - \frac{x}{3}\right) = M \;\Leftrightarrow 1000 \times \frac{f_c}{2} \times 48.93\left(344 - \frac{48.93}{3}\right) = 56.17 * 10^6$$

$$\Leftrightarrow 8.02 \times 10^6\, f_c = 56.17 \times 10^6$$

$$\Leftrightarrow \boldsymbol{f_c = 7\ Mpa}$$

Calculate the tensile stress in steel

$$f_s = \alpha_e f_c\frac{d-x}{x} \Leftrightarrow f_s = 7 * 7 * \frac{344-48.93}{48.93}$$

$$\Leftrightarrow \boldsymbol{f_s = 295.49\ Mpa}$$

$$\Longrightarrow \rho_{p,eff} = \frac{566}{1000 \times 117} = 4.84 \times 10^{-3}$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\phi}{2}\right), \frac{(\mathrm{h-x})}{3}\right] = 1000 \times 117 = 117000\ \mathrm{mm^2/m}$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{295.49-0.4\left(\frac{2.89}{4.84*10^{-3}}\right)(1+7*4.84*10^{-3})}{200000} = 2.43 \times 10^{-4}$$

We have 2.43$\times 10^{-4} \geq \frac{0.6*295.49}{200000} \Leftrightarrow 2.43 \times 10^{-4} \geq 8.86 \times 10^{-4}$

Take $\varepsilon_{cr} = (\varepsilon_{sm} - \varepsilon_{cm}) = 8.86 \times 10^{-4}$

$S_{r,max} = 3.4\, c + 0.425\, (k_1 k_2 \phi / \rho_{p,eff})$, where $k_1 = 0.8 \quad k_2 = 0.5$ (for bending)

$$S_{r,max} = 3.4 \times 50 + 0.425 \left(\frac{0.8 \times 0.5 \times 12}{4.84 \times 10^{-3}}\right) = 591.49\, mm$$

$$w_k = 591.49 \times 8.86 \times 10^{-4}$$

$w_k = 0.52\, mm > 0.2\, mm$ **not Ok**

Increase the area of steel to H16 at 75 mm spacing, $A_{sprov} = 2680\ mm^2/m$ .Both faces in the vertical direction.

The new depth to neutral axis is calculated as follows;

$$2680 * 7 * (344 - x) - 2680 * 7 * (x - 56) = 500x^2$$

$$18760(344 - x) - 18760(x - 56) = 500x^2$$

$$6453440 - 18760x - 18760x + 1050560 = 500x^2$$

$$500x^2 + 37520x - 7504000 = 0$$

$$x = \frac{-37520 \pm \sqrt{37520^2 + 4 * 500 * 7504000}}{1000}$$

$$\boldsymbol{x = 90.6\ mm}$$

Calculate the new compressive stress in concrete

$$1000 * \frac{f_c}{2} * 90.6 \left(344 - \frac{90.6}{3}\right) = 56.17 * 10^6 \Leftrightarrow 1422 * 10^6 f_c = 56.17 * 10^6$$

$$\Leftrightarrow \boldsymbol{f_c = 3.95\ Mpa}$$

Calculate the new tensile stress in steel

$$f_s = 7 * 3.95 * \frac{344 - 90.6}{90.6} \Leftrightarrow \boldsymbol{f_s = 77\ Mpa}$$

$$\Longrightarrow \rho_{p,eff} = \frac{2680}{103130} = 0.026$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\phi}{2}\right), \frac{(\mathrm{h-x})}{3}\right] = 1000 \times 103.13 = 103130\ \mathrm{mm^2/m}$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{77 - 0.4 * \left(\frac{2.89}{0.026}\right)(1 + 7 * 0.026)}{200000} = 1.22 \times 10^{-4}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = 1.22 \times 10^{-4} \geq \frac{0.6 \times 77}{200000} \Leftrightarrow 1.22 \times 10^{-4} \geq 2.54 \times 10^{-4}$$

Take $\varepsilon_{cr} = 2.54 \times 10^{-4}$

$$S_{r,max} = 3.4 \times 50 + 0.425\left(\frac{0.8 \times 0.5 \times 16}{0.026}\right) = 274.62\ mm$$

$$w_k = 274.62 \times 2.54 \times 10^{-4} = 0.07\ mm$$

$w_k < 0.2\ mm \Longrightarrow$ **OK**

3.2.3. Base slab

The base of tank is subjected to earth pressure reaction due to the self-weight of the tank and the weight of the liquid stored. The needed reinforcement is given and all the necessary verifications carried out.

3.2.3.1. Calculation of actions on base slab

- **Total weight of empty tank (W_T)**

Area of base extension=$(7.6 \times 6.6) - (6.8 \times 5.8) = 10.72\ \mathrm{m^2}$

Weight of base=$(6.8 \times 5.8 \times 0.4) \times 25 = 394.4\ KN$

Weight of base extension =$(10.72 \times 0.4) \times 25 = 107.2\ KN$

Weight of soil on base extension =$(10.72 \times 4) \times 19.5 = 836.16\ \mathrm{KN}$

Weight of top slab =$(6.8 \times 5.8 \times 0.2) \times 25 = 197.2\ KN$

Weight of longitudinal wall = $2(6.8 \times 5 \times 0.4) \times 25 = 680\ KN$

Weight of lateral wall = $2(5 \times 4 \times 0.4) \times 25 = 400\ KN$

$$\boldsymbol{W_T = 2614.96\ KN}$$

- **Weight of water in the tank (W_w)**

$$\boldsymbol{W_w} = 6 \times 5 \times 4 \times 10 = \mathbf{1200\ KN}$$

3.2.3.2. Analysis of the base slab

It is considered as a two-way slab and interior panels are considered,

$l_x / l_z = 6.4/5.4 = 1.2$

- **Short span coefficients ($\boldsymbol{\beta_{sx}}$)**

Negative moment at continuous edge=0.042

Positive moment at mid span =0.032

- **Long span coefficient ($\boldsymbol{\beta_{sy}}$)**

Negative moment at continuous edge =0.032

Positive moment at mid span=0.024

3.2.3.3. Calculation of solicitations

This to obtain the load case that will give maximum solicitations

a. Case 1: Tank is empty, no ground water acting

Earth pressure intensity (q)=$\frac{W_T}{A} = \frac{2614.96}{7.6 \times 6.6} = 52.132\ \text{KN/m}^2$

At ULS:$P_{Ed1} = 1.35 \times 52.132 = 70.38\ \text{KN/m}^2$

i. Short span

Continuous edge moment = $0.042 \times 70.38 \times 5.4^2 = 86.20$ KNm

Mid-span moment = $0.032 \times 70.38 \times 5.4^2 = 65.67$ KNm

Cantilever moment =$\frac{70.38 \times 0.6^2}{2} = 12.67$ KNm

ii. Long span

Continuous edge moment = $0.032 \times 70.38 \times 5.4^2 = 65.67$ KNm

Mid-span moment = $0.024 \times 70.38 \times 5.4^2 = 49.25$ KNm

Cantilever moment $= 12.67$ KNm

b. case 2: Tank is empty, with ground water acting

Uplift pressure of water under the base due to 4.4 m head of water

Uplift pressure = $10 \times 4.4 = 44 \text{ KN/m}^2$

Net pressure at ULS = $P_{Ed3} = 70.38 - 1.2(44) = 17.58 \text{ KNm}$

This load case is not critical

c. case 3: Tank is full, with no ground water acting

At ULS,

$$P_{Ed3} = 1.35\left(\frac{2614.96}{7.6 \times 6.6}\right) + 1.2\left(\frac{1200}{7.6 \times 6.6}\right) = 99.1 \text{ KN/m}^2$$

i. Short span

Continuous edge moment = $0.042 \times 99.1 \times 5.4^2 = 121.37 \text{ KNm}$

Mid-span moment = $0.032 \times 99.1 \times 5.4^2 = 92.47 \text{ KNm}$

Cantilever moment =$\frac{99.1 \times 0.6^2}{2} = 17.84 \text{ KNm}$

ii. Long span

Continuous edge moment = $0.032 \times 99.1 \times 5.4^2 = 92.47 \text{ KNm}$

Mid-span moment = $0.024 \times 99.1 \times 5.4^2 = 69.35 \text{ KNm}$

Cantilever moment $= 17.84 \text{ KNm}$

d. case 4: Tank is full, with ground water acting

At ULS: $P_{Ed4} = 99.1 - 1.2(44) = 46.3 \text{ KNm}$

i. Short span

Continuous edge moment = $0.042 \times 46.3 \times 5.4^2 = 56.7 \text{ KNm}$

Mid-span moment = $0.032 \times 46.3 \times 5.4^2 = 43.2 \text{ KNm}$

Cantilever moment $=\frac{46.3\times0.6^2}{2} = 8.33$ KNm

ii. Long span

Continuous edge moment = $0.032 \times 46.3 \times 5.4^2 = 43.2$ KNm

Mid-span moment = $0.024 \times 46.3 \times 5.4^2 = 32.4$ KNm

Cantilever moment $= 8.33$ KNm

It results, the maximum bending moments are in load case 3. The bending moments in the short span and long span are almost equal. The maximum ones are taken and are used for design. The shear forces on the base slab results from the top slab, self-weight of the walls and the base projection. The maximum one is from the base projection.

$$V_{Ed} = 99.1 \times 0.6 = 59.46\ KN$$

Table 3. 9. Summary of solicitations at the base slab at ULS

Solicitation	M_{Ed}(KNm)	V_{Ed}(kN)
Short span	121.37	59.46
Long span	92.47	59.46

3.2.3.4. Reinforcement of the base slab

The reinforcement of the walls is done for the short span and long span.

a. Flexure design of short span

$M_{Ed} = 121.37$ KNm

$$K = \frac{M_{Ed}}{f_{ck}bd^2} = \frac{121.37 \times 10^6}{30 \times 1000 \times 344^2} = 0.034$$

Since $K < 0.167$ no compression moment reinforcement required

$Z=d\left[0.5 + \sqrt{(0.25 - 0.882K)}\right]$ =0.97d

$$A_{sreq} = \frac{M_{Ed}}{0.87 f_{yk} Z} = 836.16\ \text{mm}^2/\text{m}$$

Provide HA12 at 100 mm spacing, $A_{sprov} = 1130\ mm^2/m$

$A_{sprov} > A_{smin}$ so, the provided reinforcement is adequate.

Provide H12 at 100 mm spacing $A_{Sprov} = 1130\ mm^2/m$

b. Flexure design of long span

- $M_{Ed} = 92.47\ \text{Km}$
- Assuming ϕ 12 mm bars will be used and $b = 1000$ mm(designing per unit width)
- $d = 344$ mm
- $K = \frac{M_{Ed}}{f_{ck} b d^2} = \frac{92.47 \times 10^6}{30 \times 1000 \times 344^2} = 0.026$
- Since $K < 0.167$ no compression moment reinforcement required
- $Z = d\left[0.5 + \sqrt{(0.25 - 0.882K)}\right]$ =0.98d
- $A_{sreq} = \frac{M_{Ed}}{0.87 f_{yk} Z} = 642.83\ \text{mm}^2/\text{m}$

Provide H12 at 150 mm spacing, $A_{sprov} = 754\ mm^2/m$

$A_{sprov} > A_{smin}$ so, the provided reinforcement is adequate

Provide H12 at 150 mm spacing $A_{Sprov} = 754\ mm^2/m$

3.2.3.5. Verifications

a. Shear verification

$V_{Ed} = 59.46$ kN

$V_{Rdc} = \max(V_{Rdc1}, V_{Rdc2})$

$$V_{Rdc1} = \left[C_{Rd,c} k (100 \rho_1 f_{ck})^{1/3} + k_1 \sigma_{cp}\right] b_w d = 79.63\ \text{kN}$$

- $C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$
- $k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{164}} = 2.1 > 2$ so, $k = 2$

- $\rho_1 = \frac{A_{sprov}}{bd} = \frac{1130}{1000 \times 344} = 3.285 \times 10^{-3} < 0.2$
- $k_1 = 0.15$
- $\sigma_{cp} = \frac{N_{Ed}}{A_c} = 0$

$\mathrm{V_{Rdc2}} = (\mathrm{V_{min}} + \mathrm{K_1}\sigma_{cp})\mathrm{b_w d} = 186.45\ \mathrm{kN}$

- $\mathrm{V_{min}} = 0.035\mathrm{k}^{3/2}\mathrm{f_{ck}^{1/2}} = 0.0352^{3/2}30^{1/2} = 0.542\mathrm{N/mm^2}$

$\boldsymbol{V_{Rdc}} = \max(V_{Rdc1}, V_{Rdc2}) = \mathbf{186.45\ kN}$

Since $V_{Rdc} > V_{Ed}$ no shear reinforcement is required.

b. Thermal cracking

Design crack width $w_k = 0.2\ mm$

$w_k = S_{r,max}.\varepsilon_{cr}$

Maximum crack spacing,

$$S_{r,max} = 3.4C + 0.425\left(\frac{k_1 k_2}{\rho_{P,eff}}\right)$$

$\mathrm{k_1} = 0.8 \quad \mathrm{k_2} = 1 \quad \mathrm{C} = 50\ \mathrm{mm} \quad \mathrm{h} = 400\ \mathrm{mm} \quad \mathrm{b} = 1000\ \mathrm{mm} \quad \mathrm{A_{sprov}} = 754\ \mathrm{mm^2/m}$

$$\rho_{P,eff} = \frac{\mathrm{A_{sprov}}}{A_{c,eff}} = \frac{754}{140000} = 5.386 \times 10^3$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\phi}{2}\right)\right] = 1000 \times 140 = 140000\ \ \mathrm{mm^2/m}$

$\Longrightarrow\ S_{r,max} = 927.52\ \mathrm{mm}$

i. Early age cracking

$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}]R_j - 0.5\varepsilon_{ctu}$

$\mathrm{K} = 0.65 \qquad \alpha_c = 10 \times 10^{-6} \qquad T_1 = 25^{\circ}C \qquad \varepsilon_{ca} = 15 \times 10^{-6} \qquad \varepsilon_{ctu} = 76 \times 10^{-6}$

$$R_j = \frac{1}{1 + \frac{A_n}{A_o}\frac{E_n}{E_o}} = \frac{1}{1 + 0.8 \times 0.5} = 0.714$$

- $\frac{E_n}{E_0} = 0.8 \qquad \frac{A_n}{A_0} = \frac{h_n}{2\,h_0} = \frac{0.4}{2\times0.4} = 0.5$

$\varepsilon_{cr} = 0.65[10 \times 10^{-6} \times 35 + 15 \times 10^{-6}] \times 0.714 - 0.5 \times 76 \times 10^{-6} = 1.314 \times 10^{-4}$

$w_k = 927.52 \times\ 1.314 \times 10^{-4} = 0.122\ \text{mm}$

$w_k < 0.2\,mm\ \Longrightarrow$ **OK**

ii. long term restraint cracking

$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca})R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\,\varepsilon_{ctu}$

$\varepsilon_{cd} = 150 \times 10^{-6} \quad \varepsilon_{ca} = 50 \times 10^{-6} \quad \varepsilon_{ctu} = 104 \times 10^{-6}$

$R_1 = R_2 = R_3 = 0.67$

$$\varepsilon_{cr} = 0.65[(10 \times 10^{-6} \times 35 + 50 \times 10^{-6})][(10 \times 10^{-6} \times 35 + 50 \times 10^{-6}) + (10 \times 10^{-6} \times 25 \times 0.67) + (150 \times 10^{-6} \times 0.67)] - 0.5(104 \times 10^{-6})$$

$= 2.964 \times 10^{-6}$

$w_k = 927.52 \times\ 2.964 \times 10^{-4} = 0.275\ \text{mm}$

$w_k > 0.2\,mm$ **not ok**

Increase the area of steel to
H16 at 100 mm spacing
$A_s prov = 2010\,mm^2/m$

- $P_{p,eff} = \frac{2010}{1000\times40} = 0.014$

$S_{r,max} = 3.4 \times 50 + 0.425\left(\frac{0.8\times1\times16}{0.014}\right) = 558.57\,mm$

$w_k = 558.57 \times 2.964 \times 10^{-4}$

$w_k = 0.166\,mm$

$w_k < 0.2\,mm\ \Longrightarrow$ **OK**

c. Crack Width due to Loading

Load case 3 is the critical one, the bending moment at SLS is given by;

$$\boldsymbol{P_{Ed}} = \frac{1}{7.6 \times 6.6}(2714.96 + 1200) = 78.05\ KN/m$$

$$M_{SLS} = 0.042 \times 78.05 \times 5.4^2 = 95.59\ KN.m$$

The neutral axis is at:

$$MA_{Sprov}\alpha_e(d - x) - A_{sprov}\alpha_e(x - d')M = \frac{bx^2}{2}M\ ;\ \alpha_e = 7\ ;\ A_{sprov} = 1130\ mm^2/m$$

$$1130 \times 7 \times (344 - x) - 1130 \times 7 \times (x - 56) = 500x^2$$

$$500x^2 + 15820x - 3164000 = 0$$

$$x = \frac{-15820 \pm \sqrt{15820^2 + 4*500*3164000}}{1000} = 65.29\ mm$$

Calculate the compressive Strength

$$bx\frac{f_c}{2}\left(d - \frac{x}{3}\right) = M \Longrightarrow \frac{1000*65.29}{2} * \left(344 - \frac{65.29}{3}\right) f_c = 95.59 * 10^{-6}$$

$$\Longrightarrow 10.52 * 10^{-6} f_c = 95.59 * 10^{-6}$$

$$\Longrightarrow \boldsymbol{f_C = 9.09\ Mpa}$$

Calculate the tensile stress in steel

$$f_s = \alpha_e f_c \frac{d-x}{x} \Longrightarrow f_s = 7 * 9.09 * \frac{344-65.29}{65.29}$$

$$\Longrightarrow \boldsymbol{f_s = 271.62\ Mpa}$$

$$\Longrightarrow \rho_{p,eff} = \frac{1130}{111570} = 0.01$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\phi}{2}\right), \frac{(\mathrm{h-x})}{3}\right] = 1000 \times 111.57 = 111570\ \mathrm{mm^2/m}$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{271.62 - 0.4\left(\frac{2.89}{0.01}\right)(1 + 7 * 0.01)}{200000}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = 7.4 * 10^{-4} \geq \frac{0.6*271.62}{200000} \Longleftrightarrow 7.4 * 10^{-4} \geq 8.15 * 10^{-4}$$

$\Longrightarrow$ Take $\varepsilon_{cr} = \varepsilon_{sm} - \varepsilon_{cm} = 8.15 * 10^{-4}$

$$S_{r,max} = 3.4 * 50 + 0.425\left(\frac{0.8*0.5*12}{0.01}\right) = 374\,mm$$

$w_k = 374 \times 8.15 \times 10^{-4} = 0.3mm$

$w_k > 0.2\,mm$ **not ok**

Increase the area of steel to:

H20 at 150 mm spacing

$\boldsymbol{A_{sProv} = 2090\,mm^2/m}$

The new neutral axis x is at:

$$2090 * 7 * (344 - x) - 2090 * 7 * (x - 56) = 500x^2 \Longrightarrow x = 82.81\,mm$$

Calculate the new compressive Strength

$$bx\frac{f_c}{2}\left(d - \frac{x}{3}\right) = M \Longrightarrow \frac{1000*82.81}{2} * \left(344 - \frac{82.81}{3}\right) f_c = 95.59 * 10^{-6}$$

$$\Longrightarrow \boldsymbol{f_C = 7.3\,Mpa}$$

Calculate the new tensile stress in steel

$$f_s = \alpha_e f_c \frac{d-x}{x} \Longrightarrow f_s = 7 * 7.3 * \frac{344-82.81}{82.81}$$

$$\Longrightarrow \boldsymbol{f_s = 161.73\,Mpa}$$

$$\Longrightarrow \rho_{p,eff} = \frac{2090}{111570} = 0.02$$

- $A_{c,eff} = \mathrm{b} \times \min\left[\frac{h}{2}, 2.5\left(C + \frac{\Phi}{2}\right), \frac{(\mathrm{h-x})}{3}\right] = 1000 \times 105.73 = 105730\ \mathrm{mm^2/m}$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{161.17 - 0.4\left(\frac{2.89}{0.02}\right)(1 + 7*0.02)}{200000}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = 4.76 \times 10^{-4} \geq \frac{0.6 * 161.17}{200000} \Leftrightarrow 4.76 \times 10^{-4} \geq 4.84 \times 10^{-4}$$

$$\Longrightarrow \text{Take } \varepsilon_{cr} = \varepsilon_{sm} - \varepsilon_{cm} = 4.84 * 10^{-4}$$

$$S_{r,max} = 3.4 \times 50 + 0.425\left(\frac{0.8 \times 0.5 \times 20}{0.02}\right) = 340\ mm$$

$$w_K = 340 \times 4.84 \times 10^{-4} = 0.165\ mm$$

$$w_K < 0.2\ mm \Longrightarrow \boldsymbol{OK}$$

3.2.4. Reinforcement plan of the underground tank

A properly designed tank must be able to withstand the applied loads without cracks that would permit leakage. This is achieved through providing proper reinforcement and spacing. Table 3.10 shows the reinforcement per unit width of the structural elements of the tank. Figure 3.8 to 3.11 show the distribution of the reinforcement.

Table 3. 10. Reinforcement of the underground tank

Structural elements		A_{sprov} (mm^2/m)	Number of bars/ m	Spacing (mm)	Direction
Top slab	Short span	452	4 HA12	250	Vertical
	Long span	377	3 HA12	300	Horizontal
Walls	Short span	2680	14 HA16	75	Vertical
	Long span	2010	10 HA16	100	Horizontal
Base slab	Short span	2010	10 HA16	100	Vertical
	Long span	2090	7 HA20	150	Horizontal

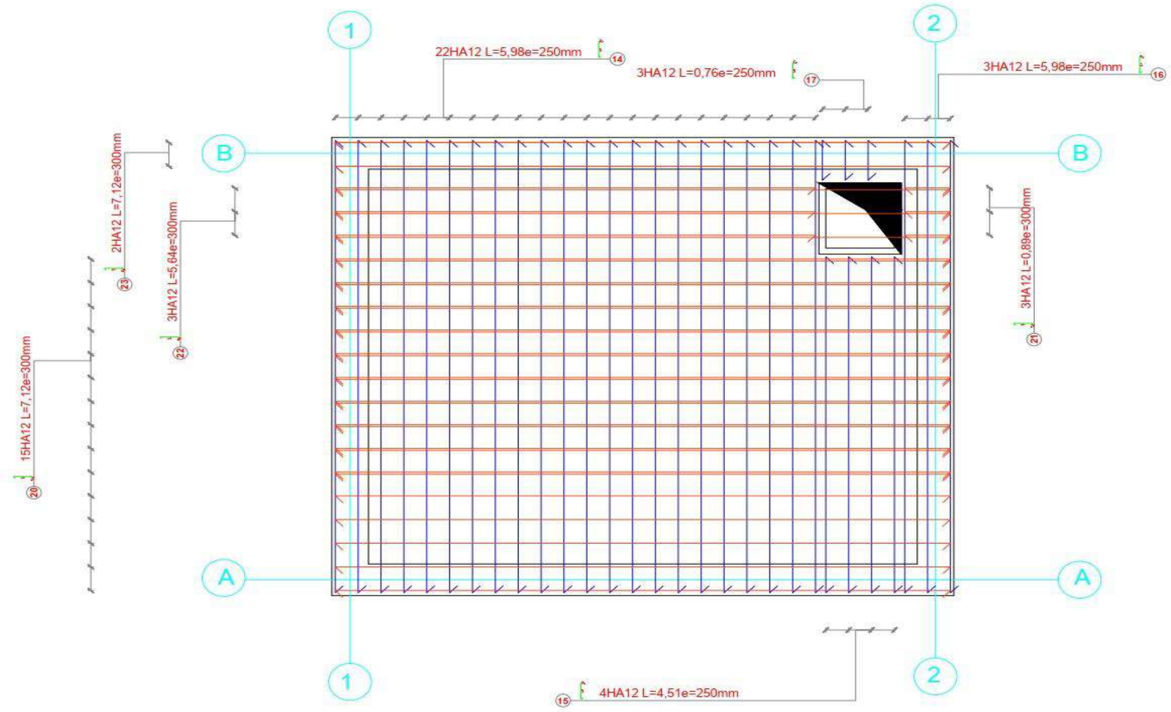

Figure 3. 8. Reinforcement plan of top slab

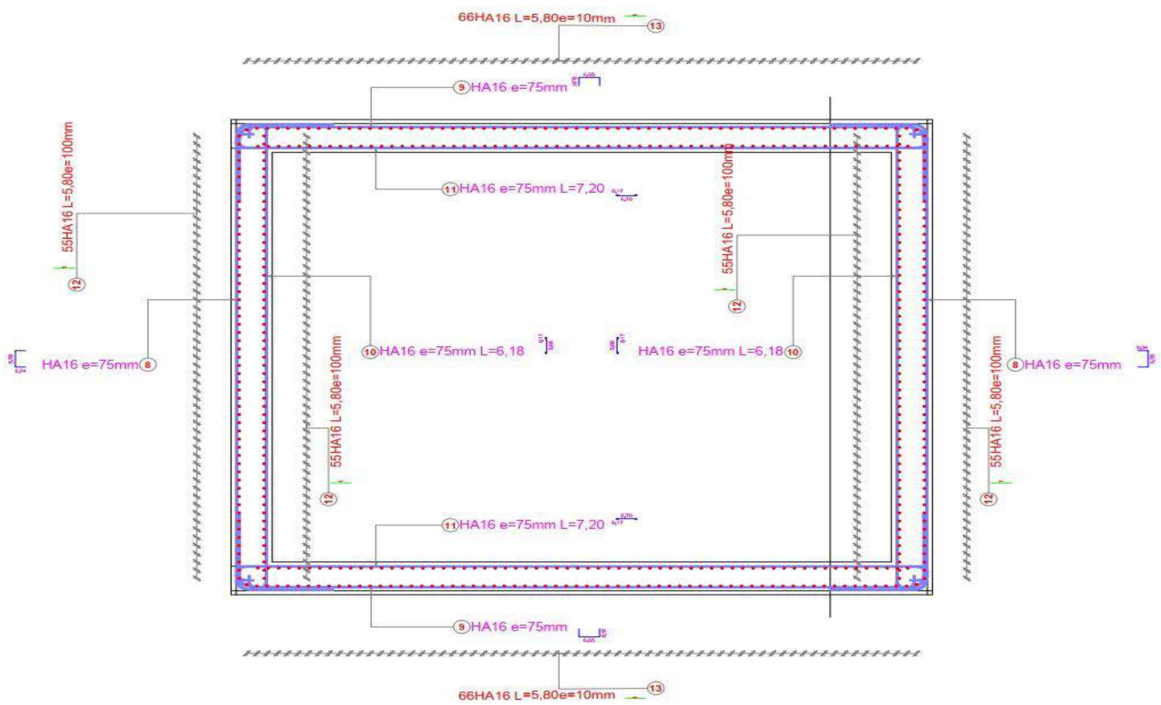

Figure 3. 9. Reinforcement plan of CUT C-C(Walls)

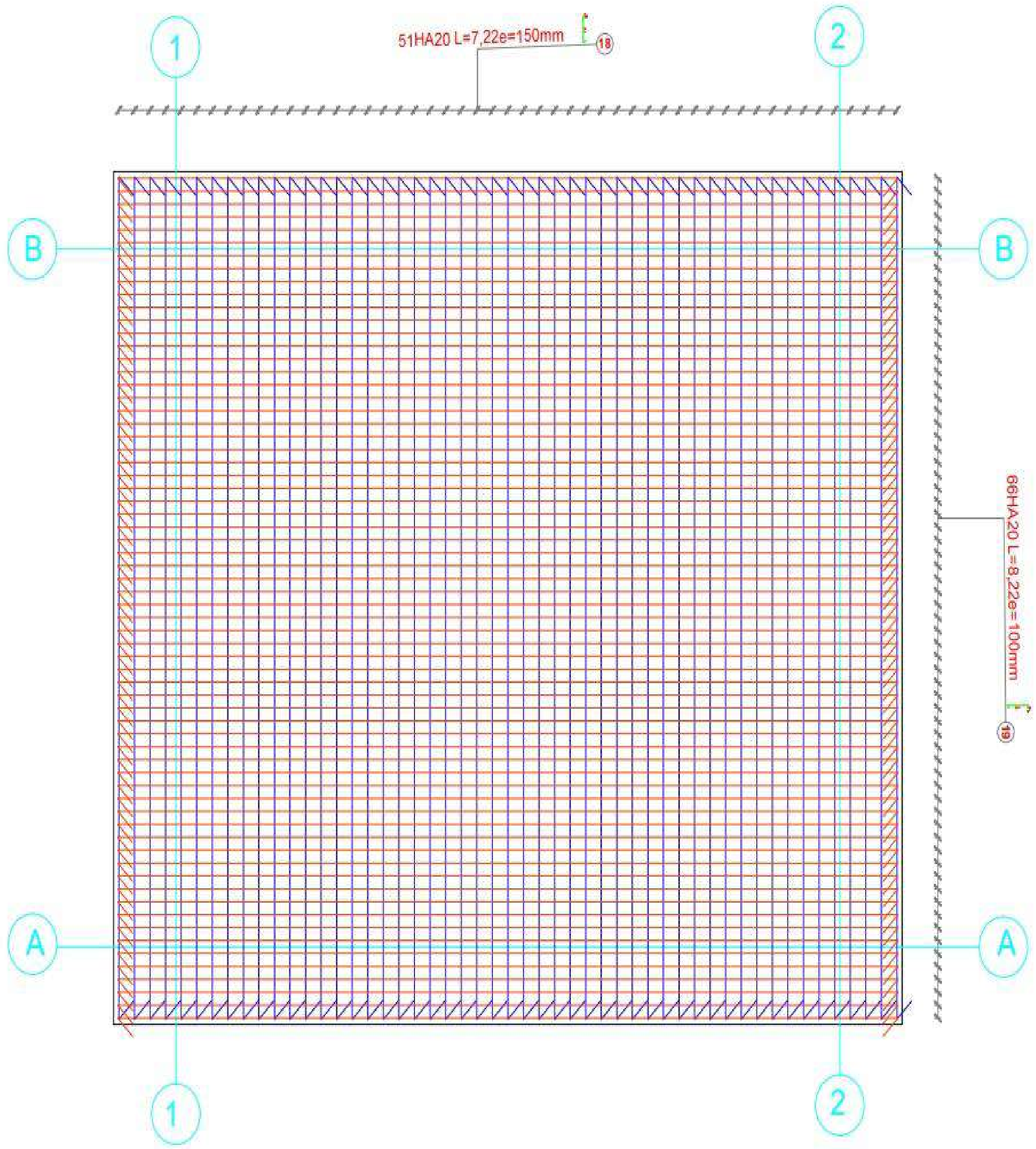

Figure 3. 10. Reinforcement of base slab

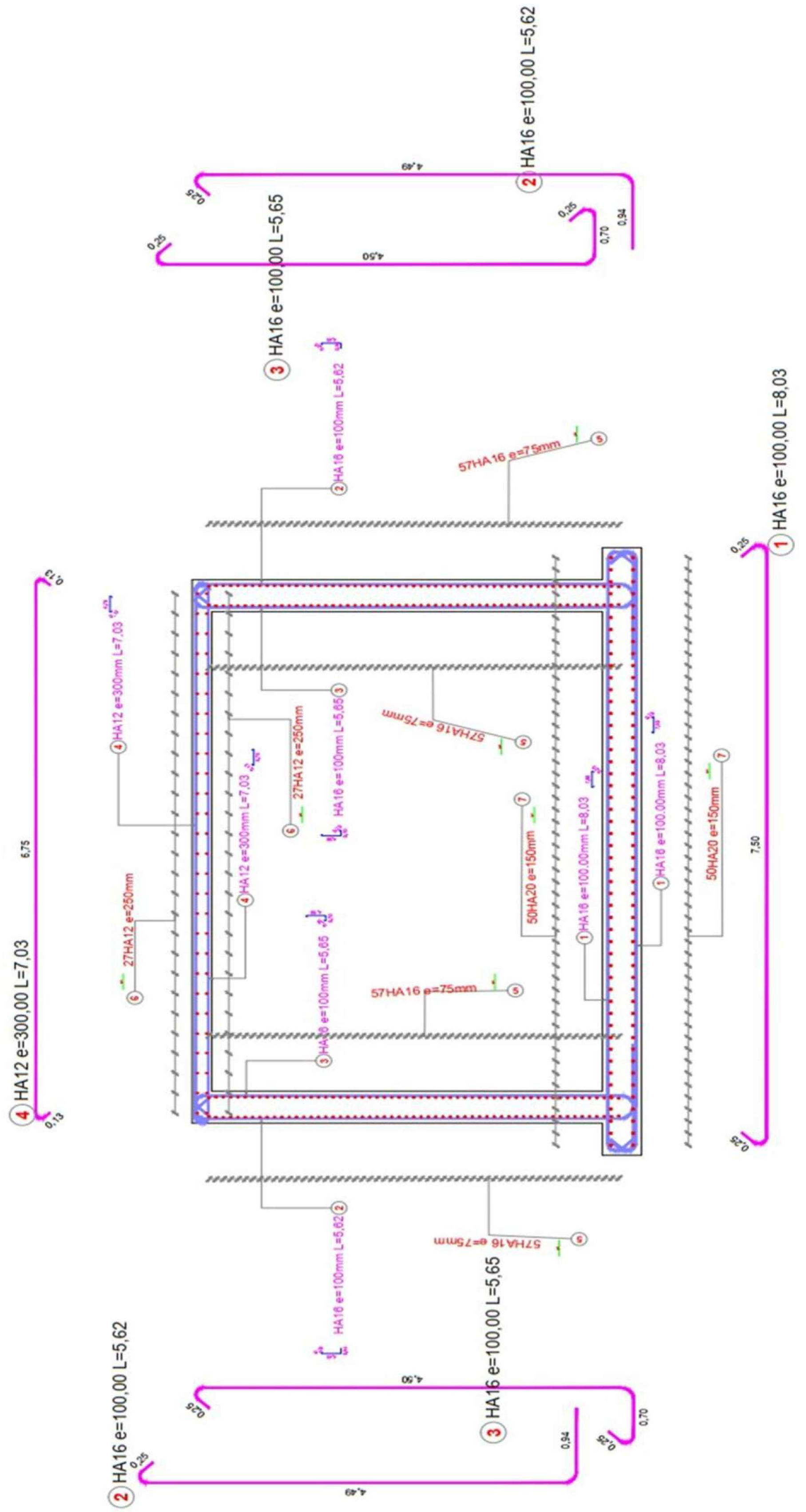

Figure 3. 11. Reinforcement plan of CUT B-B (Longitudinal view)

3.3. Parametric study in different types of soils

The parameters of soil that affect the solicitation on the underground tank are unit weight of soil (γ) and angle of internal friction (φ). Based on the given formulas throughout, an excel spreadsheet is generated so as to study the bending moments and their corresponding area of steel reinforcement required on the walls and base slab of the tank.

3.3.1. varying unit weight of soil

This done by fixing the value of angle of internal friction at 27°**,** then varying unit of weight of soil between values that ranges from 16-22. The results are shown in table 3.11.

Table 3. 11. Varying unit weight of soil and bending moments

	Wall/short span		Wall/long span		Base slab			
Unit weight of soil, γ (KN/m^3)	$M_{Ed,max}$ (KNm)	$A_{srequired}$ (mm^2/m)	$M_{Ed,max}$ (KNm)	$A_{srequired}$ (mm^2/m)	M_{Ed}^- (KNm)	$A_{srequired}$ (mm^2/m)	M_{Ed}^+ (KNm)	$A_{srequired}$ (mm^2/m)
16	64.26	436.49	47.08	318.42	116.41	801.81	88.69	606.36
17	66.17	449.69	48.38	327.32	117.82	811.86	89.77	613.90
18	68.08	462.89	49.68	336.22	119.23	821.92	90.84	621.44
19.5	70.94	482.73	51.63	349.58	121.35	837.02	92.46	632.77
21	73.80	502.62	53.58	362.96	123.47	852.13	94.07	644.10
22	75.71	515.86	54.88	371.89	124.90	862.22	95.15	651.66

Illustrative figures 3.12 to 3.15 are done so as to show the relationship between varied unit weight of soils and the reinforcement on the structural elements of the tank.

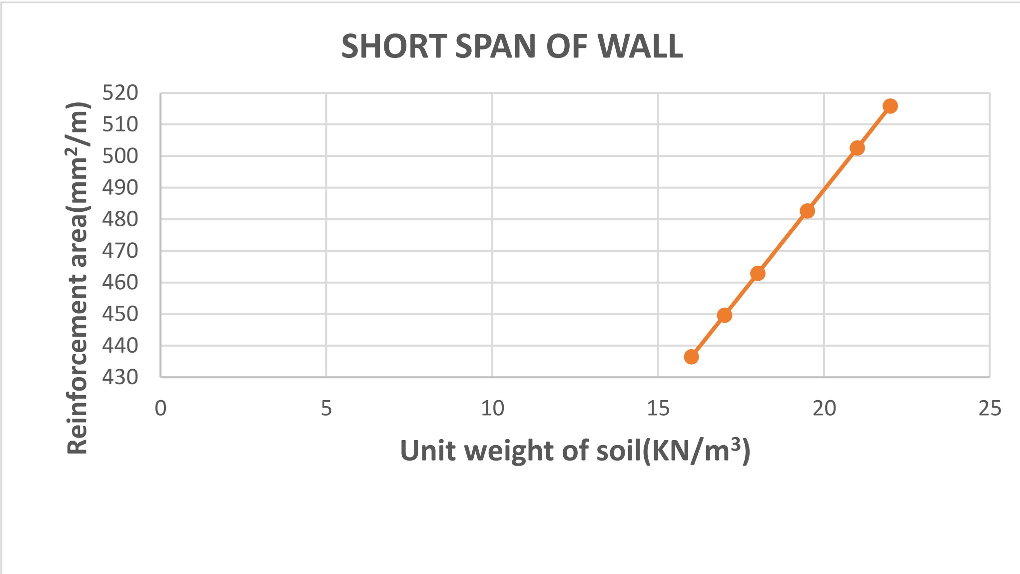

Figure 3. 12. Reinforcement on short span of wall as unit weight of soil increases

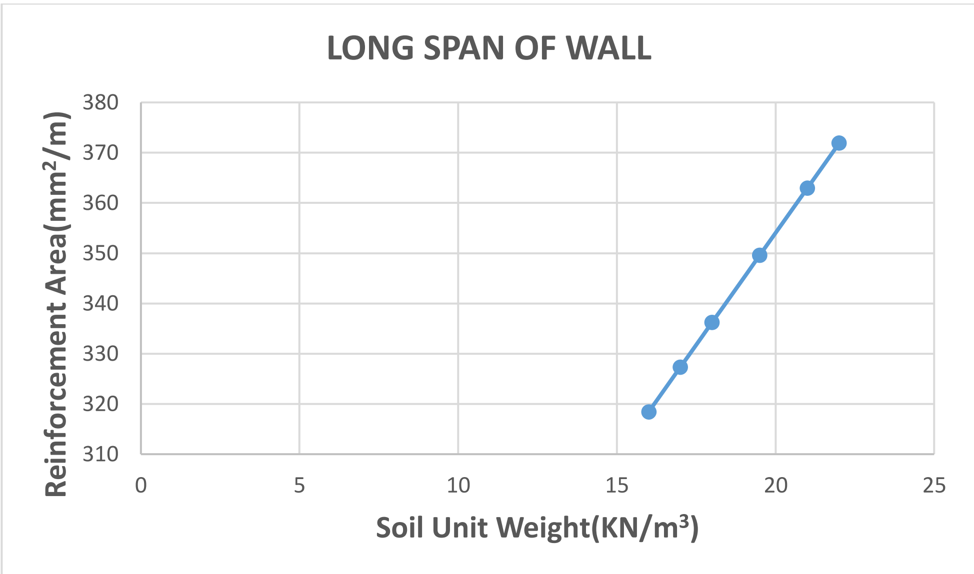

Figure 3. 13. Reinforcement on Long span of wall as unit weight of soil increases

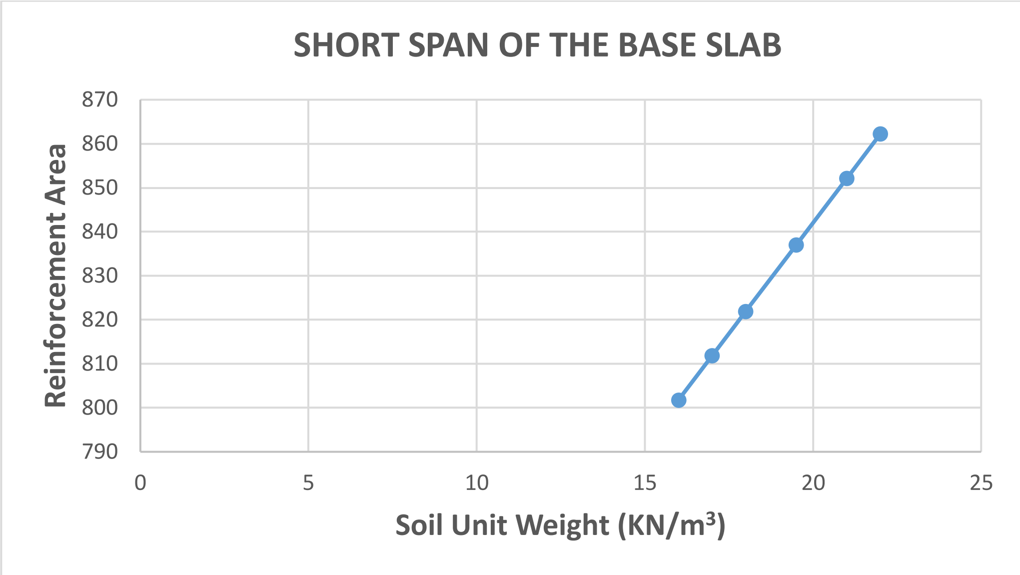

Figure 3. 14. Reinforcement on short span of base slab as unit weight of soil increases

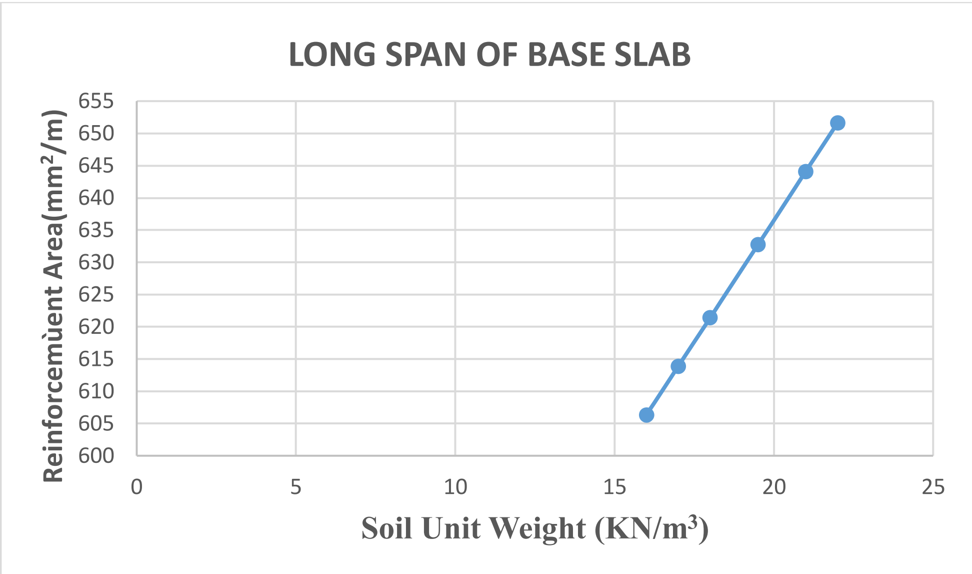

Figure 3. 15. Reinforcement on long span of base slab as unit weight of soil increases

It results that as the unit weight of soil increases, the bending moments on the tank increase thus increasing the reinforcement.

3.3.2. Varying angle of internal friction

This done by fixing the value of unit of weight of soil at 19.5 KNm, then varying the angle of internal friction between values that ranges from 0-44. The results are shown table 3.12.

Table 3. 12. Varying angle of internal friction and bending moments

	Wall/short span		Wall/long span	
Friction angle (°)	$M_{Ed,max}$ (KNm)	$A_{srequired}$ (mm^2/m)	$M_{Ed,max}$ (KNm)	$A_{srequired}$ (mm^2/m)
0	113.75	782.93	86.35	590.01
10	93.46	639.80	69.90	475.53
20	78.81	537.38	58.01	393.44
27	70.94	482.73	51.63	349.58
40	60.10	407.83	42.84	289.42
44	57.55	390.24	40.77	275.27

Illustrative figures 3.16 and 3.17 are done so as to show the relationship between varied angles of internal friction and the reinforcement on the structural elements of the tank.

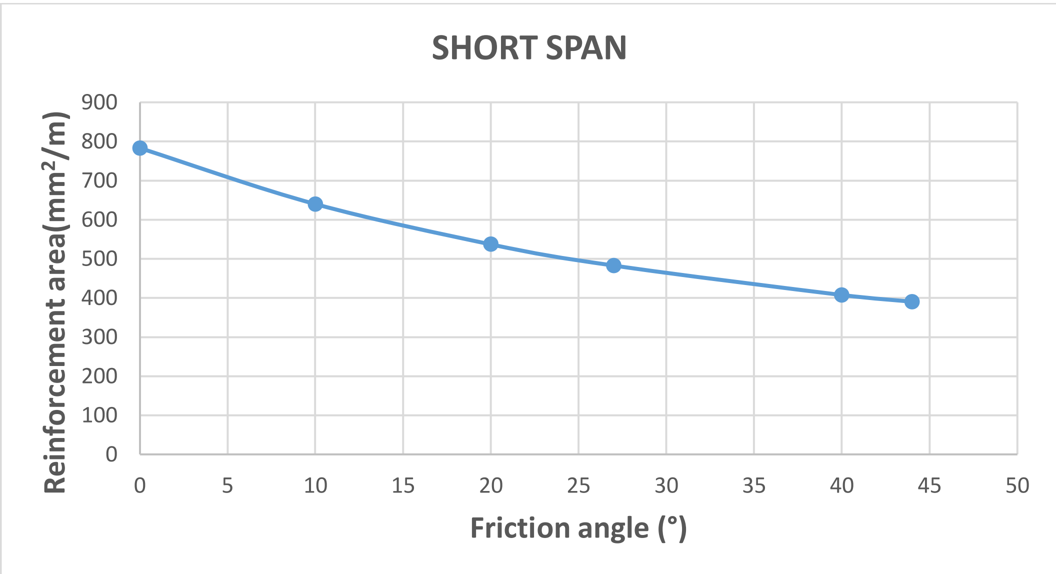

Figure 3. 16. Reinforcement on short span of wall as angle of internal friction of soil increases

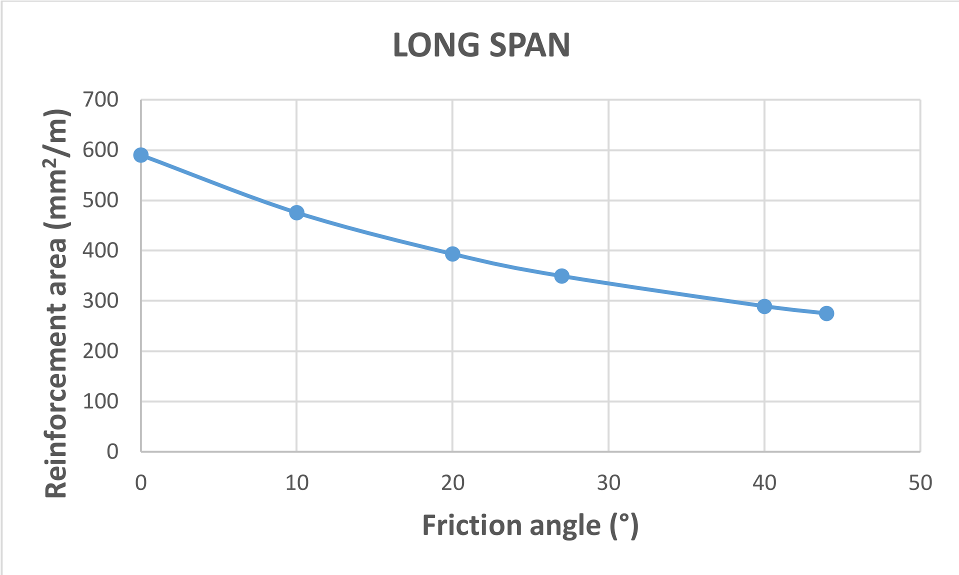

Figure 3. 17. Reinforcement on short span of wall as angle of internal friction of soil increases

It results that as the angle of internal friction of soil increase, the bending moments on the tank decrease thus decreasing the reinforcement.

Conclusion

The aim of this chapter was to present and interpret results of the design and calculation of underground tank. A presentation of the conceived case study was made based on the materials used and actions considered. According to the design process, detailed analysis of the structural elements of the tank have been carried out under different load cases and detailed design calculations. The goal of providing a structurally sound tank that will not leak was achieved by providing the correct amount and distribution of reinforcement, the correct spacing and crack width calculation that did not exceed 0.2 mm. Finally, it was shown that the different type of soils does have an effect on the design of the underground tank. It results that as the unit weight of soil increased, the bending moments on the tank increased thus increasing the reinforcement. Whereas, as the angle of internal friction increased heading to a well compacted soil, the bending moments decreased thus less reinforcement required.

GENERAL CONCLUSION

The main objective of this thesis was firstly to provide a properly designed reinforced concrete underground tank to store water for firefighting purposes and, later, to perform a parametric study to observe the effect of different types of soils on the designed tank. The tank was rectangular in shape with three components, namely the top slab, walls, and base slab, and was able to withstand the applied loads without cracks that would permit leakage. In order to achieve this objective, the work was structured around three chapters. The first chapter consisted of the current state of knowledge on the classification of various water tank types, a general overview of different types of soils, and essential design methods for a liquid-retaining structure that is built below the ground level. The second chapter, entitled methodology, first described the steps to be followed for the conception of the case study, then the detailed design assumptions based on existing standards of the rectangular reinforced concrete underground water tank. In addition, the presentation of analytical analysis and limit state design method of the case study were explained. Finally, presentation and interpretation of the different analyses announced in the methodology were made in chapter 3. As a result, according to the design process, detailed analysis of the structural elements of the tank were carried out under different load cases and detailed design calculations. The goal of providing a structurally sound tank that will not leak was achieved by providing the correct amount and distribution of reinforcement, the correct spacing and crack width calculation that did not exceed 0.2 mm. For instance, with sand soil, on the top slab, 4 HA12 per unit width at 250 mm spacing in the short span and 3 HA16 per unit width at 300 mm in the long span were needed. 14 HA16 per unit width at 75 mm spacing in the short span and 10 HA16 per unit width at 100 mm in the long span on walls were needed. On the base slab, 10 HA16 per unit width at 100 mm spacing in the short span and 7 HA20 per unit width at 150 mm in the long span were needed. In addition, the parametric study results indicated as the unit weight of soil increased, the maximum solicitations acting on the tank increased thus increasing the reinforcement. Whereas, as the angle of internal friction increased heading to a well compacted soil, the maximum solicitations decreased thus less reinforcement required. The interpretation of the results obtained during the parametric study gave us a better understanding of the need to incorporate soil parameters, notably unit weight of soil and angle of internal friction, when designing underground tanks.

However, certain points are seen limits to this work, notably the lack of considering safe bearing values for different types of soils Moreover, geotechnical field data would have been better than data from the literature, which would certainly deepen this work in the choice of the appropriate design parameters. In order to improve this work, it is proposing as a perspective a finite element analysis of the seismic response for underground tanks and effect of different types of soils on the dynamic behavior of underground tanks.

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ANNEX

ANNEX 1: Tables for methodology

Table A1.1. Table of partial factors for materials (EC2)

Limit State	Material	Partial Factor for Material γ_m
Ultimate limit state	Concrete	1.5
	Reinforcing Steel	1.15
Serviceability limit state	Concrete	1.0
	Reinforcing Steel	1.0

Table A1.2. Environmental requirements with regard to durability for reinforcement steel $C_{min,dur}$ (EC2)

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Table A1.3. Design value for coefficient of thermal expansion (Narayanan and Goodchild ,2012)

Coarse aggregate/rock group	Design value for coefficient of thermal expansion (microstrain/°C)
Chert or flint	12
Quartzite	14
Sandstone	12.5
Marble	7
Siliceous limestone	10.5
Granite	10
Dolerite	9.5
Basalt	10
Limestone	9
Glacial gravel	13
Lytag (coarse and fine)	7

Table A1.4. T_1 for walls assuming C30/37 strength concrete class (Narayanan and Goodchild ,2012)

Formwork system and wall thickness		T_1 for assumed cementitious binder and binder content (°C)									
		CEM I	20% fly ash	30% fly ash	40% fly ash	50% fly ash	not specified (= CEM I)	20% ggbs	40% ggbs	60% ggbs	80% ggbs
Assumed cement Class+		R	R	N	S	S	R	R	N	N	S
Indicative binder content* (kg/m³)		340	360	365	380	400	340	340#	340	375	450
Using steel formwork	**250 mm th.**	16	13	11	10	9	16	13	12	10	9
	500 mm th.	28	24	22	20	18	28	24	21	20	17
	750 mm th.	37	32	29	27	24	37	33	30	27	23
	1000 mm th.	43	38	35	32	29	43	40	36	34	29
Using 18 mm ply formwork	**250 mm th.**	24	20	18	16	15	24	21	18	16	14
	500 mm th.	36	32	29	26	24	36	33	29	27	23
	750 mm th.	43	38	35	31	29	43	39	36	34	29
	1000 mm th.	47	42	39	34	33	47	44	41	39	34
Using 37 mm ply formwork	**250 mm th.**	27	23	20	18	16	27	24	20	18	16
	500 mm th.	40	35	32	29	27	40	36	33	31	27
	750 mm th.	45	40	37	34	31	45	42	39	37	32
	1000 mm th.	49	44	40	37	34	49	47	43	41	36

Notes
1 Derived from CIRIA C660[18]: Temperature als – Prediction of the early-age temperature rise in concrete
2 For slabs see text in Section A5.3 above

Key
* Indicative only and intentionally at high end of the range
\# Assumed
\+ See note to Table A5

Table A1.5. Autogenous shrinkage strain, ε_{ca} (Narayanan and Goodchild ,2012)

Strength Class (f_{ck28}/f_{cu28}) MPa	$\varepsilon_{ca}(t)$ microstrain				
	3 days	7 days	14 days	28 days	∞
C20/25	8	11	13	17	25
C30/37	15	21	27	33	50
C40/50	23	32	40	50	75

Note
Autogenous shrinkage is normally considered not to increase beyond 28 days and is deemed to be within drying shrinkage.
ref: BS EN 1992-1-1 Exp (3.12), Exp (3.13)

Table A1.6. Tensile strain capacity for sustained loading, ε_{ctu} (Narayanan and Goodchild ,2012)

Strength Class (f_{ck28}/f_{cu28}) MPa	$\varepsilon_{ctu(t)}$ (microstrain)			
	ε_{ctu3S} *#	ε_{ctu3N} *#	ε_{ctu3R} *#	ε_{ctu28} *
C20/25	53	63	68	91
C25/30	58	70	75	100
C30/37	63	**76**	81	**108**
C35/45	67	81	87	116
C40/50	71	86	92	123

Key
* Values determined from fctmt/Ecmt according to BS EN 1992-1-1. They include a factor of 1.23 for to allow for the effects of creep and sustained loading (CIRIA C660 Cl. 4.8).
* Values are for quartzite aggregates. For limestone aggregates, add 10%, for sandstone aggregates, add 40% and for basalt aggregates, deduct 20%
\# At 3 days, values depend on Class of cement used. See note to Table A5
REF: BS EN 1992-1-1, 3.1.2(6), 3.1.2 (9), Table 3.1

Table A1.7. Restraint factor R_j near base wall (Narayanan and Goodchild ,2012)

A_n/A_o	0.25	0.50	0.75	1.00	1.25	1.43
$R_{early\ age}$	0.85	0.74	0.66	0.59	0.53	0.50
$R_{long\ term}$	0.80	0.67	0.57	0.50	0.44	0.41

Notes
1 Based on CIRIA C660[38]
2 Assumes wall cast at edge of slab. For walls cast remote from an edge, restraint factors are higher: the calculated value of A_n/A_o should be divided by a factor of 2.0 to obtain an estimate of R.
3 For a wall cast at the edge of a slab CIRIA C660 recommends assuming that $A_n/A_o = h_n/h_o$

Table A1.8. Drying shrinkage at 10 000 days, $\varepsilon_{cd,t=10000}$ for C30/37 concretes (Narayanan and Goodchild ,2012)

$\varepsilon_{cd,t=10000\ Ext'}$ (microstrain)				
Cement Class	***Notional size, h_0, mm***			
	200	**300**	**400**	**500**
	External RH = 85%,			
R	248	217	207	198
N	179	156	150	143
S	144	125	120	114
	Internal RH = 45%,			
R	584	510	488	465
N	421	368	352	336
S	338	296	283	270

Notes
1 Table assumes time at moment considered less time to end of curing $(t - t_s) = 10\ 000$ days and two sides exposed. If one side exposed $h_0 = 2 \times$ thickness where h_0 = area of concrete/perimeter.
2 Drying shrinkage is generally taken to be zero at early age.
3 Notional thickness, $h_0 = 2Ac/u$. Ac = area of concrete section. u =perimeter exposed to drying.
Ref: BS EN 1992-1-1 Exps (3.9), (B.11) and (B.12), Table 3.3

Table A1.9. Sectional areas of groups of bars, mm^2(Mosley, J.H, & Bungey, J. H.)

Bar size (mm)	*Number of bars*									
	1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*
6	28.3	56.6	84.9	113	142	170	198	226	255	283
8	50.3	101	151	201	252	302	352	402	453	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	566	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	943	1260	1570	1890	2200	2510	2830	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	6430	7240	8040
40	1260	2510	3770	5030	6280	7540	8800	10100	11300	12600

Table A1.10. Sectional areas per metre width for various bar spacings, mm^2(Mosley, J.H, & Bungey, J. H.)

Bar size (mm)	*Spacing of bars*								
	50	*75*	*100*	*125*	*150*	*175*	*200*	*250*	*300*
6	566	377	283	226	189	162	142	113	94
8	1010	671	503	402	335	287	252	201	168
10	1570	1050	785	628	523	449	393	314	262
12	2260	1510	1130	905	754	646	566	452	377
16	4020	2680	2010	1610	1340	1150	1010	804	670
20	6280	4190	3140	2510	2090	1800	1570	1260	1050
25	9820	6550	4910	3930	3270	2810	2450	1960	1640
32	16100	10700	8040	6430	5360	4600	4020	3220	2680
40	25100	16800	12600	10100	8380	7180	6280	5030	4190