FAILURE CRITERIA AND ANALYSIS OF AN UNDER GROUND RESERVOIR ON SUBMERGED SOIL USING PRINCIPLE OF BUOYANCY

A DISSERTAION AS MAJOR PROJECT

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE
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Submitted by:

MANISH GOYAL

(ROLL No 2K18/STE/504)

Under the guidance of

Dr. PRADEEP KUMAR GOYAL
Associate Professor, Department. Of Civil Engineering
Delhi Technological University

DEPARTMENT OF CIVIL ENGINEERING
DELHI TECHNOLOGICAL UNIVERSITY
(Formerly Delhi College of Engineering)
Bawana Road, Delhi-110042

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DELHI TECHNOLOGICAL UNIVERSITY

(Formerly Delhi College of Engineering)

Bawana Road, Delhi-110042

CANDIDATE'S DECLARATION

I, Manish Goyal, Roll No. 2K18/STE/504, student of M.Tech. (Structural Engineering), hereby declare that the Dissertation as Major Project titled "DETERMINATION OF FAILURE CRITERIA AND ANALYSIS OF AN UNDER GROUND RESERVOIR ON SUBMERGED SOIL USING PRINCIPLE OF BUOYANCY" which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associateship, Fellowship or other similar title or recognition.

Manish Goyal.

Place: Delhi

Date: 25.08.2021 (MANISH GOYAL)

DELHI TECHNOLOGICAL UNIVERSITY

(Formerly Delhi College of Engineering)

Bawana Road, Delhi-110042

CERTIFICATE

I hereby certify that the Dissertation as Major Project titled "DETERMINATION OF FAILURE CRITERIA AND ANALYSIS OF AN UNDER GROUND RESERVOIR ON SUBMERGED SOIL USING PRINCIPLE OF BUOYANCY" which is submitted by Manish Goyal, Roll No 2K18/STE/504, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the students under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

Pradeep Kumar

Place: Delhi

Date: 25.08.2021

(Dr. Pradeep Kumar Goyal)

Associate Professor

Deptt. Of Civil Engineering

ABSTRACT

The basic requirement in the design of reinforced water tank is to ensure it must be crack free. This study is about the analysis of an underground reinforced concrete water reservoir under fully submerged soil prone to exert buoyant force. A Microsoft Excel Spreadsheet is generated for quick assessment of behavior in various conditions of reservoir, geometrical features, critical condition for upliftment for both full and empty conditions of the tank and the probable feasible solution. It has been observed that the empty tank on fully submerged soil becomes the most critical condition in terms of factor of safety and to sustain in this condition geometrical modification is not sufficient and also uneconomical. In all the examined probable solutions, the base extension with considering the weight of soil wedge due to angle of internal friction of soil and provision of anti floatation slab gives satisfactory stability to the water reservoir on fully submerged soil. Considering both the above mentioned solution, a comparison study is made on STAAD Pro for determination of node displacement, shear stresses & moment at base slab and side walls.

Keywords: Water reservoir, uplift pressure, stability, buoyant force, Water Table.

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LIST OF SYMBOLS

BH	*Bore Hole*
W_T	*Total downward (gravity) force*
W_B	*Buoyant Force*
W_W	*Weight of walls*
W_S	*Weight of slabs*
FS	*Factor of Safety*
h	*Internal height of tank*
l	*Internal length of UGR*
w	*Internal width of tank*
H	*Outer height of UGR (including top and bottom slab)*
L	*Outer length of UGR (Including walls)*
W	*Outer width of tank*
t	*thickness of rectangular tank*
V	*Volume displaced by water due to structure*
w	*width of earthfill*
W_1, W_2, W_3	*Individual weight of UGR components*
W_T	*Total downward force including all components*
Wshelf	*Additional weight of soil acting on the shelves*
Ws	*Weight of earthfill above top slab*
γc	*Unit weight of concrete*
γr	*Unit weight of RCC*
γw	*Unit weight of water*
γ_s	*Unit Gravity of soil*
γ_w	*Specific gravity of water*
$\Sigma\, Fv$	*Summation of all vertical forces*
Φ	*Angle of internal friction of soil*

LIST OF EQUATIONS

CHAPTER- 1

INTRODUCTION

1.1 General

Water tank is for providing storage of water for drinking, food preparation, irrigation, fire suppression, chemical manufacturing, construction of civil engineering works and many other applications. The main emphasis in the structural analysis and design of a reinforced water tank should be to ensure it must be crack free both as a consequence of the loading and as results of temperature and shrinkage effect.

Reinforced water tanks can be constructed on ground, above ground or below ground depending on the requirement. The ground water tanks are resting on the ground and their walls are subjected to water pressure from sides and the base is subjected to weight of water and pressure of soil from ground. The water tanks may or may not be covered at the top. The elevated tanks are supported on staging by columns or frames. Their walls are subjected to water pressure and their base has to carry the water load, the top and walls load while the staging has to carry water and the entire tank load. For the design of elevated structures wind forces are also considered.

All around of Delhi, the authorities permanently banned supply of water through Over Head Tank. Major part of the city water supply is being done by Under Ground Reservoirs through Booster Pumping Stations. Being the Capital city many people migrate to Delhi in search of their livelihood and accordingly the population of the city is increasing. With the increasing population, authorities need to upgrade its existing infrastructure by making additional source of accumulation of treated water through Under Ground Reservoirs.

1.2 Underground reservoir

An Under Ground Reservoir has no foundation as it rest on the wide portion of natural, firm and different types of soil. The walls of underground tanks subjected to water pressure from inside and the soil pressure from outside. The base of underground water tank is subjected to weight of water acting downward and the soil pressure acting upward. The soil pressure depends on the soil condition whether it is wet or dry. These tanks must always be covered at the top to avoid any mishappening and to ensure the quality of water. These tanks could be constructed on fully or partially submerged soil as per the site conditions. These tanks are constructed fully underground and should not be uplifted due to ground water pressure surrounding the tank. As they are underground structure and due to presence of ground water, they are subjected to uplift and severe corrosion.

1.3 Site selection

For the analysis, a site near the Yamuna River (not in the flood plain of river) for construction of UGR is selected. Since the lots of people are migrated and living in this area, they have no proper water supply network system rather they are depend on tanker water supply. At that site, the matter of concern is the ground water level which may vary between 2 m to 4m being in the vicinity of river. The basic problem at that site is existence of submerged soil over which the UGR has to be constructed. Before designing the components of UGR it is essential to study the behavior of submerged soil and its buoyancy over the UGR structure. The water pressure of submerged soil acts upwards, which may uplift the structure due to buoyancy.

CHAPTER-2

LITERATURE REVIEW

Some research papers have been studied for the analysis and design of underground water tank, which provides the basic idea to initiate the work and urges for incorporating the principle of buoyancy due to high ground water level in analysis of underground reservoirs. Following papers have been studied and revealed the concept for the analysis as well as various aspects in this field

1. **W. O. Ajagbe, E. O. Ilugbo, J. O. Labiran and A. A. Ganiyu**

   Analysis and Design of a Fully Submerged Underground Water Tank using the Principle of Beam on Elastic Foundations

In that study, the underground water tank's walls and base slab were analyzed as a completely submerged UG reservoir. The analysis was done based on the theory of beam on elastic foundations (Biot, 1937) that works on the assumption that the reaction forces of the foundation are proportional at every point to the deflection of the beam at that time. This theory is adopted when the flexural rigidity (EI) of a beam is taken into account, which is considered as Winkler foundation regarding the soil acting as a bed of springs.

To avoid the tedious and repetitive calculations required in the design and analysis of an underground reservoir, a Microsoft Excel Spreadsheet Design and Analysis Program, named MESDAPro, was generated for the research for fast assessment of various tanks as per their capacities, soil conditions and other relative parameters.

Factor of safety of 1.4 and 1.6 were applied to the combined self-load and superimposed load respectively and their sum was made to realize the specified ultimate design load for the top slab. It was observed that the moments of wall, wall base and base slab decreases with increase in soil sub-grade modulus at constant capacity, height and breadth of the tank while they increase with increase in height of the tank at constant value of sub-grade modulus, tank capacity and breadth. In all the examined cases, the moments obtained is higher when the tank is considered empty than when considered full.

2. **Surinder Kumar**

CADD package for analysis and design of underground tanks

In early times when the use of computer is not prevailed in the design and analysis, a FORTRAN 77 software is used to avoid the tedious calculations, interactive use of charts and their interpolations. IS 3370- PartIV-1969 has been used to design the walls of tanks by using wall co-efficient. Based on the existing ground conditions and water table it was concluded that when L/H ratio of tank exceeds 3, the wall is designed as cantilever. As this study used the old codes in designing the tanks and use of FORTRAN 77 based software, it becomes difficult to understand the actual procedure adopted to get a final conclusion and the effect of changed codal provisions on the study.

3. **Thalapathy. M., Vijaisarthi R,P, Sudhaker.P, Sridharan.V, Sateesh.V.S**

Analysis and economical design of water tanks

This study deals with the design philosophy for the safe and economical design of tanks based on working stress method. The design has been made on excel sheet and concluded that in circular tanks when h/d ratio is 0.45, it becomes the safe and economical design based on IS codes. It was also explained that the Limit state design is most economical as compared to working stress in term of quantity of steel and concrete used.

4. **Suraj P. Shinde**

Computer aided analysis and design of underground water tank

A comparative study was done with the results of SAP and STAAD Pro and manual results in which it was found that the result remain almost same in all the three cases. While designing or analysis of tank stability, analysis has been made for different combination of conditions which may come during the functional life of the structure. Some remedial measure also suggested if the stability of the structure is not achieved. Out of those remedial measures some methods have been used in this thesis for the analysis and checked the effectiveness of these measures. Analysis of those measures also important, as per the requirement of volume of water or fluid storage.

5. Mr. Manoj Nallanathel, Mr. B. Ramesh[l], L. Jagadeesh

In this design it was mentioned that the corner stresses and maximum shear and bending stresses found less for circular tanks than remaining other designs. The shapes of water tanks, plays an important role in the stress distribution and economy. By using STAAD pro, the results obtained were very useful and precise than the conventional method and its results. In Underground tank, Uplift pressure is predominant which is caused by surrounding soil on outside walls of tank

6. Issar Kapadia, Purav Patel, Nilesh Dholiya, Nikunj Patel, January 2017 IJSDR

Design, Analysis and comparison of Underground Rectangular Water Tank by using STAAD Provi8 Software

In this paper displacement behavior of the tank due to dead load, due to water in the tank and due to external soil pressure on walls have been studied by using STAAD Pro software. They compare their analysis by taking dimensions of two different tanks without changing the storage volume of the tanks. By varying the size of two major components e.i; wall thickness and floor thickness, analysis has been made and results have been compared.

In this study several features are not available to understand the various inputs used for the analysis like presence of ground water table, soil pressure and angle of internal friction etc.

7. Anshuman Nimade, Niraj Soni, Goutam Verma, Vikas Joshi, Sharad Chaurasia, March 2018, IJSTE

Parametric Study of Underground Water Tank using FEM

The authors have developed a finite element model to study the behavior like node displacement and stress pattern of underground tank for different L/B ratio. They also studied the base pressure, plate moments by considering the tank empty and full water level condition. They have considered the hydrostatic and soil pressure on external walls and neglected the uplift pressure due to deep ground water table. They have also considered the base extension of 500 mm on all sides and safe bearing capacity of 120 kN/m^2. The findings of their study are related to effect of stress, node displacement and base pressure varies with L/B ratio when the ratio is greater than 2.

The effect of ground water table to check the stability of the structure in different conditions might not be incorporated. Load of top slab may also be considered to add the effect of dead load on the walls and floor slab.

8. NPCA White Paper – precast.org (2018)

This paper gives an idea about the ideology of buoyancy that how it is useful for the analysis of ground water tanks when the ground water level may cause upliftment of the structure. This paper is completely based on the Principle of buoyancy *(Archimedes' Principle)*. An example has been given in that study to calculate the various forces acting on the underground tank when the ground water level is maximum and measures to counter the effects of buoyancy primarily the upliftment. The structure shall remain safe against the upliftment. Infact, this paper helps to calculate various forces and their use in the analysis of the underground reservoir.

9. Komal K Wagh, Akshay K Ghuge, Deepak N Gaidhane, Gajendra R Gandhem April 2021, IRJET

Design and analysis of underground tank by using STAAD Pro

In this study, by considering various parameters like unit weight of soil, angle of internal friction, bearing capacity of soil etc. they design the tank with pre-determined conditions and compare their results with manual calculation and found that there is saving of 15% to 20% of total steel in the whole structure.

Likewise in second literature, the effect of ground water table to check the stability of the structure in different conditions is not available. Load of top slab may also be considered to add the effect of dead load on the walls and floor slab.

10. IS 3370 (Part 1) – 2009: Code of Practice for Concrete Structures for storage of liquid.

The structure shall be checked and designed for both full and empty condition. For critical condition water load is considered as dead load. Crack width shall not be more than 0.2 mm both in direct tension and flexure. The stability of the structure should be checked for safety against overturning and sliding. Cracking in concrete can be avoided by filling the tank at slower rate for first time. The rate of filling first time shall not be more than 1 m per 24 hours.

Outcome of the literature review

The study of various literatures, encourage us to incorporate the ground water table effect and check the stability of the underground reservoir. Apart from other studies the weight of top slab and overburden soil (earthfill) over it, also incorporated with variable depth under different conditions. After finding the critical condition, suitable remedial measure has to be adopted with the data of ground water, soil subgrade modulus and angle of internal friction as per the site conditions. Their analysis through STAAD Pro provides a comparative statement of different remedial measures which finally gives us an idea that which remedial measure shall be the best suited by analyzing the node displacements, stresses effect and moment at various locations.

CHAPTER-3

ANALYSIS METHODOLOGY

To analyze the behavior of structure on submerged soil, a systematic approach is adopted for computation of various forces acting upward due to amount of displaced ground water and to resist that total downward force coming from the structure. Different parameters are required for the analysis of underground water tank in submerged soil as under.

- Determination of water table
- Computation of Downward (Gravity) Forces
- Computation of Upward Buoyant Force
- Safety Factor for upliftment

3.1 Determination of water table

Ground water table can be determined using the following approaches as mentioned below:

- Soil Investigation Reports and Boring Data
- Regional and Seasonal Variations
- Conservative Approach

These are described below.

3.1.1 Soil Investigation Reports and Boring Data

When designing an underground concrete structure, it is necessary to understand the aim of the structure that it is to be used. Site conditions and underground conditions are vital piece of data required for the planning and calculations for effective performance of the structure within the prevailed condition and to stop uplift / floatation.

The primary factor that has got to be determined for the analysis is that, in which conditions the concrete structure is going to be placed below the ground level i.e., the information of water table. This information gives an idea to the designers to spot the potential areas of flotation where it could be an issue.

The designer must check the soil investigation reports to collect the required information about the area. The soils report is presumably the foremost reliable source of data since it supports a study of the jobsite's conditions. In absence of soil investigation report, core drilling of soil shall be done within the vicinity of the project. With the help of core drilling, we can determine the depth of the water level from natural ground level. The information of ground water levels got from the boring reports can be used as an idea as the effect of seasonal variation is not accounted in that report. If core drilling option is not feasible or available at that site, information of ground water level may be collected from the local authorities as well as from the well drillers. If no such information is available with the authorities as well as from the local drillers, the designer shall design the structure considering the ground water level at natural ground level, even if flooding therein is not common.

3.1.2 Regional and Seasonal Variations

The ground water level or water table is the level of an underground soil surface below which the soil remains saturated with water. The water level deviates both with the prevailing seasons and from year to year because it's suffered by climatic variations and by the quantum of precipitation employed by the vegetation. Excess withdrawal of water from the wells and implementation of rain water harvesting to recharge them artificially also affected the ground water level considerably. Considering these factors, on safer side, an engineer should adopt appropriate value of water table in the effective design of an underground RCC water tank. It must be ensured that the underground structure won't float or shift upwards from a miscalculation of water level.

3.1.3 Conservative Approach

If the soils investigation reports or other old information is not available on the water table including variation (regional and regional), the design engineer should adopt the value of ground water level for designing the structure on the conservative side means by considering the water table on ground level. This may make sure that the structure is going to be ready to withstand regional and seasonal fluctuations. If we go for a conservative approach, the structure must be design with the water level at natural ground level. A conservative approach contributes to offset unforeseen and supererogatory value once enough data concerning the location conditions is unprocurable.

3.2 Downward forces (gravity forces)

After the water level is decided, the design engineer must check out computing all the downward forces which will be working on the structure. It requires to be calculated within the design of an underground structure so as to work out if the entire gravitational forces (downward, W_T) are in excess of the buoyant force (upward W_B). All the downward force (W_T) is calculated by the summation of all gravitational vertical forces as shown in Eq. (1)

$$W_T = W_1 + W_2 + W_3 + W_4 + W_5 \ldots \quad (1)$$

Where, W_1, W_2, W_3, W_4….. are the weights of different components of the concrete structure.

If all the vertical downward forces (**W_T**) are more than the upward buoyant force (**W_B**), then the structure will not float. For this condition (**$W_T > W_B$**), an underground structure shall be designed for most critical condition. Following are the vertical downward forces (**W_T**) which shall be considered for analysis of underground water tank.

- Weight of all components of UGR and additional concrete if applicable.
- Weight of water in tank (for tank full condition).
- Weight of soil on top slabs (Earth fill).
- Weight of soil on extended base.
- Weight of soil wedge due to internal angle of friction between soil particles.

3.2.1 Weight of all components of UGR and additional concrete (W_1)

This includes the weight of walls, base and top slab, beams, columns and additional filled concrete to counter the buoyant force. If opening is provided, weight of concrete replaced by opening should be subtracted from total weight of all components.

3.2.2 Weight of water in tank

In the tank full condition, weight of water acting on the bottom slab is computed which is proportional to the depth of water in the tank.

3.2.3 Weight of soil on top slabs (Earth fill) (W_2)

The weight of the soil resting on the slab may be determined by multiplying the surface area of the top slab by the depth of earth fill alongwith the density of the soil.

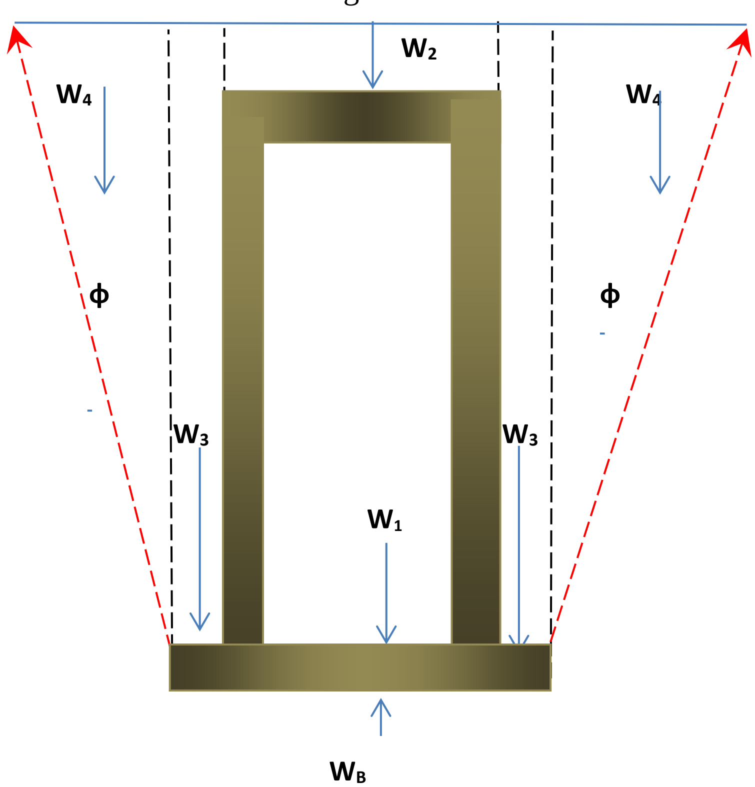
3.2.4 Weight of Soil on Extended Base (W_3)

The weight of the soils on the extended portion is determined by multiplying the area of the extended portion of bottom slab beyond the walls by the total depth of the structure alongwith the density of the soil.

3.2.5 Weight of soil wedge due to internal angle of friction between soil particles. (W_4)

By extending the size of bottom slab of the underground structure beyond the side walls, it incorporates some friction from the soil due to internal frictional resistance of soil particles in addition of vertical downward force. It can be obtained by adding the weight of the soil wedge because of base extension and frictional resistance of soil. Soil friction angle depends on the type of soil, and its cohesiveness.

The various downward forces acting on the structure are shown in figure 3.1.



W_1	Weight of concrete structure
W_2	Weight of filled up soil
W_3	Weight of soil on base extension
W_4	Weight of soil wedge due to soil internal friction
Φ	Angle of internal friction
W_B	Buoyant Force

Figure 3.1*: Gravitational (downward) forces on an underground structure*

3.3 Upward buoyant force

It works on principle of buoyancy in which the object is buoyed up by the fluid equal to the weight of displaced fluid. It can be expressed by Eq.(2)

$$W_B = \gamma_w \times V \tag{2}$$

Where,

W_B = buoyant force, γ_w = density of the water, V = displaced volume of the fluid

For the static equilibrium, the algebraic sum of all vertical forces (Σ Fv = 0) must be equal to zero to analyze the underground structures.

3.4 Safety factor for upliftment

The factor of safety is expressed in Eq. (3) as a ratio of summation of resisting downward forces and a disturbing upward force due to buoyancy.

$$Factor\ of\ Safety\ (FS) = \frac{Downward\ Force(W_T)}{Upward\ Force(W_B)} \tag{3}$$

As per IS 3370- Part-1 (2009), Code of practice for Concrete Structures for Storage of Liquids in clause no. 7.2.b.2, the stability of the structure should be checked against uplift using a factor of safety of 1.2.

3.4.1 Selection of an appropriate factor of safety

In general, the factor of safety (**FS**) is proportional to the overall impact to the project / structure. The design engineer should choose a suitable FS after obtaining required necessary information about the location of jobsite. In UGRs the factor of safety (**FS**) indicates the risk associated with hydrostatic loading conditions. In the areas, where flooding is predominant to the top of the structure and using resistance to dead weight only, generally a FS of 1.10 is adopted Areas, where high ground water conditions persist because of high flood plains, a FS of 1.25 can be taken for the analysis. In the areas where data of maximum ground water or high flood levels are not available or where soil friction is included in the flotation resistance, higher FS values should be considered *(ref: NPCA – Buoyancy White paper, 2018)*

In this study factor of safety for determining the stability is considered as 1.2 which as per the codal provisions of IS 3370- Part-1 (2009).

CHAPTER- 4
PREVENTIVE MEASURES FOR UPLIFTMENT

Various methods may be adopted to overcome the effect of upliftment due to buoyancy for the stability of structure. In case, during the analysis of the underground reservoir, the required safety factor is not achieved, the problem can be fixed by adopting the measures mentioned below in detail. Some of the most prominent methods explained to reduce the effect of buoyancy.

- *Increasing thickness of the components of UGR*
- *Filling of concrete in the additional depth*
- *Extension of bottom slab all around the structure*
- *Anti-floatation slab*

4.1 Increasing thickness of the components of UGR

In this method, the objective is to increase the downward gravity weight. So, the dead weight of the structure can be increased by increasing the thickness of components of UGR like top slab, walls, bottom slab etc. This will increase the concrete mass and hence, the downward weight can be increased. It could not be able to add considerable downward weight by increasing the thickness of members only. Infact, it is an expansive method which may affect the economy of the project.

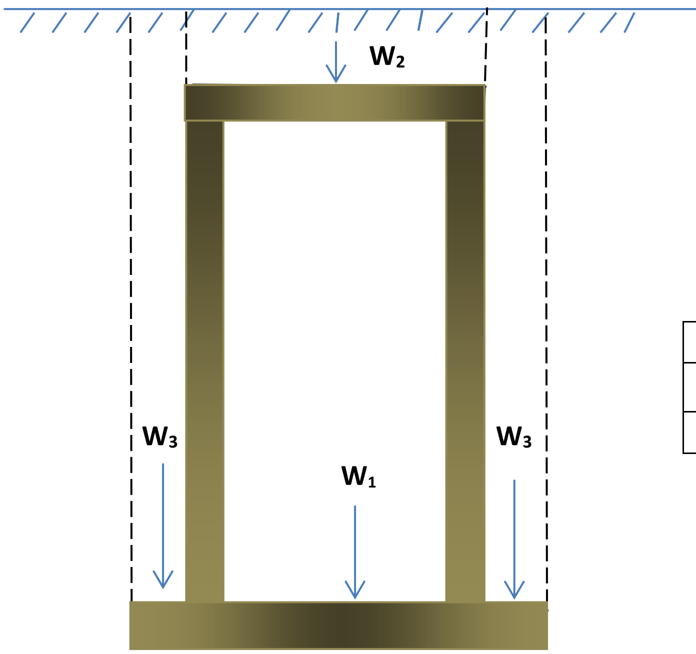
4.2 Filling of concrete in the additional depth

In this method, required depth to get the desired factor of safety is worked out, Then the additional depth other than the required depth for functional use of UGR is filled with the concrete to increase the gross dead weight of the structure without increasing the member thickness. This method may be suitable for the tanks, which do not require huge accumulation of water as the structure will become deeper. When large amount of water is to be stored, the depth will go beyond the imagination and this may increase the fabrication problems as well as make a sharp impact on the economy of the project.

4.3 Extension of bottom slab all around the structure

This is another method, which can be used to avoid the effect of buoyancy is to increase the size of bottom slab than the top slab. It means that the sides of bottom slab will come outside of the walls. This required more excavation than the required. This may help in increasing the total downward force because the soil above the extended part of bottom slab also incorporates in adding the gravitational forces. This additional weight can be calculated as the surface area of the extended part of bottom slab multiplying by the depth of soil over the extended part and the density of the soil.

The size of the extended base may be large and wide as per the requirement to resist the buoyant force. It may be considered as the cost-effective method over the other methods explained above to overcome the effect of buoyancy. Efforts shall be made to cast the bottom slab monolithically with the other components of tank to avoid the design of connections. In Figure 4.1, W_3 is the additional weight of soil which shall be considered while computing the downward gravity forces.

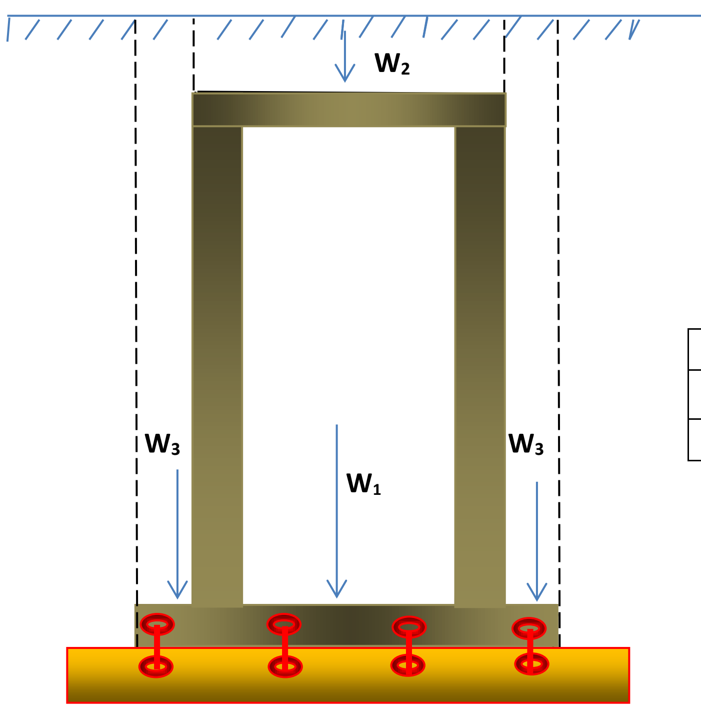


W_1	Weight of concrete in structure
W_2	Weight of top soil
W_3	Weight of soil on extended base

Figure 4.1*: Base Extension of Bottom Slab of UGR*

4.4 Anti-floatation slab

It has already been mentioned that the extended bottom slab shall be cast monolithically with the other components of the structure. Sometimes, it is not possible to construct monolithically slab due to large extended base. To overcome this problem, it is better to use another bottom slab over which the entire tank will rest which may be precast or cast-in-situ. When the main structure is rest directly on the anchored slab, the problem of misalignment in the existing & bottom slab may be possible. This misalignment may cause differential stress and cracking due to point load. Both the slab must be sit flush to each other and it is also a concern during fabrication. This method is most suitable for the precast structure as already they have gained the required strength. A bed of rich cement mortar shall be recommended between the two slabs or surfaces. Antifloatation slab and anchorage to main structure are shown in Fig.4.2.



W_1	Weight of concrete structure
W_2	Weight of overburden soil
W_3	Weight of soil on extended base

Figure 4.2: Anti Floatation Slab and Anchorage to Main Structure

The mechanical connections must be designed between both the slab i.e., anchored slab and the bottom slab of the structure. The net force required for connection is determined by Eq. (4).

$$\textit{Mechanical Connection Force} = FS \times W_B - W_T \tag{4}$$

CHAPTER-5

ANALYSIS OF FORCES ON UNDERGROUND WATER TANK

LOAD CALCULATION

Primarily two types of loads are need to be considered i.e.; downward forces and upward forces as mentioned in chapter 3 above to calculate the factor of safety. Several components are evaluated to determine total downward load coming of and from the superstructure over the submerged soil. The various load calculations from the pre-determined geometry of the structure has been done by adopting the following procedure.

5.1 Downward gravity forces

Downward forces include forces due to concrete members or components of the reservoir W_1, weight of earthfill W_2, weight of overburden soil W_3, and frictional resistance due to extended base W_4. The procedure for the calculation of above forces is illustrated one by one in the following articles.

5.1.1 Weight of concrete (W_1)

Weight of concrete is the main downward force which can be determined weight of the components of UGR like walls and slabs by getting the volume of the structure and multiplying it by the unit weight of concrete. The method for the calculation the weight of walls and slabs for rectangular tank is mentioned as in Eq. (5)

$$W_1 = W_w + W_s \tag{5}$$

Where, W_w and W_s are the weight of walls & slabs of UGR respectively.

$$\text{Weight of walls } (W_w) = L_w \times H_w \times T_w \times \gamma_c \tag{6}$$

$$\text{Weight of top and bottom slab } (W_s) = L_s \times B_{bt} \times T_{bt} \times \gamma_c \tag{7}$$

Where,

Lw, Ls – Length of wall and slab *Hw – Height of wall* *Bs – Width of top & bottom slab*

Tw – Wall thickness *T_{bt} – Bottom & top slab thickness* *γc- Concrete Density*

5.1.2 Weight of earthfill (W_2)

For Rectangular tanks weight of earthfill is taken as mentioned in Eq. (8)

$$W_2 = L_s \times B_s \times H_s \times \gamma_s \tag{8}$$

Ls, Bs, Hs – Length, width & height of earthfill soil on top slab

5.1.3 Weight of Soil on Base Extension (W_3)

The weight of the soils on the extended base is calculated by multiplying the depth of the structure by the surface area of extended base of bottom slab and the unit weight of the soil. The method for getting the weight of the soil is as under in Eq. (9)

$$W_3 = L_e \times W_e \times H \times \gamma_s \tag{9}$$

L_e - Length and width of extended base

W_e - Width and width of extended base

H - Total depth of structure including earthfill

γ_s - Density of soil

5.1.4 Weight of soil wedge due to Frictional Resistance on Extended Base

Due to the internal angle of soil friction a soil wedge develops which also incorporate to increase the total downward force. It can be determined by the buoyant weight of the soil wedge due to the base extension (see Figure 3.1 above). Soil internal angle of friction vary by the type of soil, and its cohesive property as shown in Table 5.1.

Table-5.1: *Type of soil and soil friction angles*

Soil type	Angle of internal friction ϕ (in degree)
Silt (Non- Plastic)	26-30
Silty Sand	27-32
Fine to Medium Sand	29-32
Well- Graded Sand	32-35
Sand and gravel mixture	33-36

In this study, being on conservative side, value of internal friction ϕ is taken as 20 degree.

5.2 Upward buoyant force (W_B)

The buoyant force can be determined by the volume of displaced fluid is multiplied by the unit weight of water. It can be expressed as Eq.10

$$W_b = \gamma_w \times V \tag{10}$$

Where,

γ_w – Density of water

V – Displaced Volume of water

5.3 Factor of safety (FS)

Once we get the total downward gravity forces and upward buoyant force, the factor of safety can be determined by the following expression:

$$Factor\ of\ Safety\ (FS) = \frac{Downward\ Force(W_T)}{Upward\ Force(W_B)} \tag{3}$$

If,

W_T > **W_B**, structure will not uplift and becomes stationary

W_T < **W_B**, structure will not remain stationery and may shift upward or float.

When **FS** is less than 1, the upward force is higher than the downward forces. In this condition the structure will not remain stationery and may float. On the other hand, if the **FS** is more than 1, the upward force is less than the downward forces, which means that the structure is stable.

CHAPTER-6

CRITICAL CONDITION FOR UPLIFTMENT IN UNDERGROUND RESERVOIR

To get the critical condition for failure of the underground water tank, it is necessary to find the ground water table at the site of study. To get the input of ground water table, 5 nos. bore holes were bored and obtained the water table of the site. The water table in meters is mentioned in Table 6.1 for reference. For calculation of water demand population of the area is required. The population of the area is assumed as 7500 and water demand is calculated at the rate of 135 lpcd. Using the Microsoft excel spreadsheet for calculation of various loads, the critical conditions for both full and empty conditions for the tank, failure is obtained and probable countermeasures developed in terms of stability to achieve desired factor of safety. The analysis is based on the methodology discussed in chapter-3 and is explained in further articles in detail.

Actual Ground water level data is collected from the site near Yamuna River as shown in Figure 6.1. Bore holes data were obtained from site by drilling.

Table-6.1*: Ground Water Level Data*

Bore Hole No.	Water Table (m)
BH-1	3.80
BH-2	3.60
BH-3	2.00
BH-4	3.30
BH-5	4.10

Figure 6.1: *Site near Yamuna River*

The average depth of water table is obtained by averaging the depth of ground water table of individual bores as 3.58 m. The most critical depth from the above data is 2 m. Due to seasonal variation; ground water level may be on grade. For safer side, ground water level is considered on grade itself in this study.

6.1 Geometry of the underground reservoir

Considering the population of around 7500 and water demand as 135 lpcd, the required size of UGR is estimated as 12m × 10m × 4.50m as shown in Table 6.2

***Table 6.2** Geometry of the UGR based on population*

Projected population of the area	7,500 Nos.
Water Demand	135 LPCD
Water required per capita per day	1012500 Litre
	1.0125 MLD
taken as	1.05 MLD
	223017.6 Gallon
Required Capacity of underground reservoir	0.223018 MGD
taken as	0.25 MGD
Volume required to accumulate required quantity of water	1052.5 m^3
Considering the rate of inflow is equal to rate of outflow, the effective storage volume required	526.25 m^3
taken as	526.00 m^3
Assuming the inside depth of the tank	4.50 m
Required area	116.89 m^2
Assuming width of tank	10 m
Required Length of tank	11.69
taken as	12 m
Final Dimensions of UGR (in meter)	12 × 10 × 4.50

6.2 Critical conditions

The UGR shall be analyzed for the most critical condition in which UGR is considered to uplift due to the buoyant force exerts by submerged soil. The possible conditions of failure are:

1. When reservoir is empty and ground water level is maximum.
2. When reservoir is full and ground water level is maximum.
3. When reservoir is empty and ground water level is minimum.

For the three conditions factor of safety is checked as per the method described above. The plan & section view of the UGR is shown in Figure 6.2, in which grid roofing at top slab by 3 nos. of beams of size 0.23m x 0.23m on each side are taken. Depth of earthfill above the top slab is taken as 0.40 m.

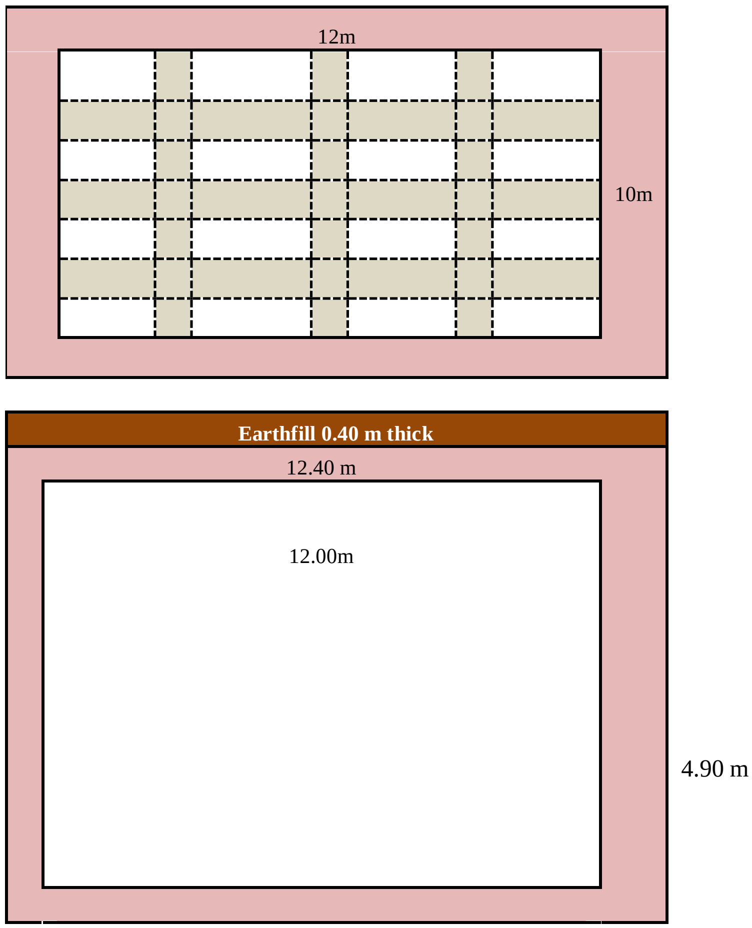


Figure 6.2 *Plan & section view of UGR*

Dimension of the UGR is shown in Table 6,4

Table 6.3*: Geometry of UGR taken to obtain critical condition*

l	w	h	Bottom Slab Thickness	Top slab Thickness	Wall thickness	L	W	H
12m	10m	4.50m	0.2m	0.2m	0.2m	12.40m	10.40m	4.90m

l, w, h are the internal length, breadth & height of the UGR
L,W,H are the outer length, breadth & height of the UGR

Factor of safety should be 1.2 as per As per IS 3370 part-1 clause 7.2.b.2

Table 6.4: *Other parameters required for analysis*

Depth of Earthfill over the Top slab	0.4 m
Ground Water Table Level	At the grade
Opening in the top slab of tank	0.6 m
Unit Weight of Concrete (γ_c)	24 kN/m^3
Unit Weight of RCC (γ_r)	25 kN/m^3
Unit weight of Soil (γ_s)	19 kN/m^3
Unit weight of Water (γ_w)	10 kN/m^3
Factor of Safety (FS)	1.2
Soil internal friction angle (ϕ)	20 degree

6.2.1 When Reservoir is empty and Ground Water Level is Maximum.

Table 6.5 shows result for the condition when reservoir is empty and Ground Water Level is Maximum.

Table 6.5: *When Reservoir is empty and Ground Water Level is Maximum.*

1. Total Weight of Concrete or RCC (Walls & Slabs)		
[(L × W × H) - (l × w × h] X γ_r		2297.60 kN
2. Total Weight of Concrete / RCC (Beams)		
No. of Beams of each side × length × size × γ_r		87.29 kN
3. Weight of overburden Soil		
L × W × Depth of earthfill × γ_s		980.10 kN
4. Weight of opening		
(π/4 × d^2 × Top slab thickness × γr) + (π/4× d^2× depth of earthfill × γ_s)		3.56 kN
TOTAL DOWNWARD WEIGHT = 1 + 2 + 3 – 4	W_T =	3361.42 kN
TOTAL UPWARD BUOYANT FORCE= L × W × (H+Depth of earthfill- depth of water table) × γ_w	W_B =	6834.88 kN
FACTOR OF SAFETY = W_T / W_B	**FS**	**0.49 < 1.2**
	Result	**Unstable**

When the reservoir is empty and the ground water level is maximum, the factor of safety is less than 1.2. It means that the reservoir will not stable in this condition.

6.2.2 When Reservoir is full and Ground Water Level is Maximum.

With the same input data, the result for above condition is shown below in Table 6.6.

__Table 6.6:__ When Reservoir is full and Ground Water Level is Maximum

1. Total Weight of Concrete or RCC (Walls & Slabs)		
$[(L \times W \times H) - (l \times w \times h] \times \gamma_r$		2297.60 kN
2. Total Weight of Concrete / RCC (Beams)		
No. of Beams of each side × length × size × γ_r		87.29 kN
3. Weight of overburden Soil		
L × W × Depth of earthfill × γ_s		980.10 kN
4. Weight of Water filled in UGR		
$l \times w \times h \times \gamma_w$		5400.00 kN
5. Weight of opening		
($\pi/4 \times d^2$ × Top slab thickness × γr)+ ($\pi/4 \times d^2$ × depth of earthfill × γ_s)		3.56 kN
TOTAL DOWNWARD WEIGHT = 1 + 2 + 3 + 4 – 5	W_T =	8761.42 kN
TOTAL UPWARD BUOYANT FORCE= L × W × (H + Depth of earthfill - depth of water table) × γ_w	W_B =	6834.88 kN
FACTOR OF SAFETY = W_T / W_B	**FS**	**1.28 >1.2**
Result of above Criteria		**Stable**

When the reservoir is full and the ground water level is maximum, the factor of safety is more than 1.2. It means that the reservoir will remain stable in this condition.

6.3.3 When Reservoir is empty and Ground Water Level is Minimum

The result for the above condition is shown in Table 6.7 for the same input data

__Table 6.7:__ When reservoir is empty and ground water level is maximum

1. Total Weight of Concrete or RCC (Walls & Slabs)		
$[(L \times W \times H) - (l \times w \times h)] \times \gamma_r$		2297.60 kN
2. Total Weight of Concrete / RCC (Beams)		
No. of Beams of each side × length × size × γ_r		87.29 kN
3. Weight of overburden Soil		
L × W × Depth of earthfill × γ_s		980.10 kN
4. Weight of opening		
($\pi/4 \times d^2$ × Top slab thickness × γ_r) + ($\pi/4 \times d^2$ × depth of earthfill × γ_s)		3.56 kN
TOTAL DOWNWARD WEIGHT = 1 + 2 + 3 – 4	W_T =	3361.42 kN
TOTAL UPWARD BUOYANT FORCE= L × W × (H+Depth of earthfill- depth of water table) × γ_w	W_B =	1676.48 kN
FACTOR OF SAFETY = W_T / W_B	**FS**	**2.01 >1.2**
Result of above Criteria		**Stable**

When the reservoir is empty and the ground water level is minimum, the factor of safety is more than 1.2. It means that the reservoir will remain stable in this condition.

After evaluating the results of all the three conditions, the UGR will remain stable in following two conditions

- *When Reservoir is full and Ground Water Level is Maximum.*
- *When Reservoir is empty and Ground Water Level is Minimum.*

It was observed that the critical condition or we can say that chance of failure is more when reservoir is empty and ground water level is maximum (6.2.1).

Considering the critical condition, appropriate and favorable solution needs to be worked out in order to make the structure stable. Detail discussion on those solutions is done in following chapter.

CHAPTER 7

STABILITY ANALYSIS OF THE UNDERGROUND RESERVOIR FOR CRITICAL CONDITION

In previous chapter, the calculation for computing downward gravity forces and upward buoyant force and checked the stability of structure in term of factor of safety. We found that the instability in the structure when the reservoir is empty and ground water level is maximum. Now we should ensure that the structure shall remain safe in the critical condition and for that some solutions or countermeasures needs to be worked out. In this chapter we will discuss one by one the most favorable solutions and check the stability in that particular solution. The solutions are:

1. Increase the thickness of components to increase the dead weight of the structure.
2. Increase the depth of the structure which may be more than the required functional depth of the structure and to fill concrete in the additional depth
3. Increase the size of bottom slab of the structure and the base is extended beyond the walls of the structure to incorporate the weight of soil over the extended base.
4. Provision of an anti - floatation slab below the structure on which the structure rests.

7.1 <u>Solution-1</u>: Add weight by increasing member thickness.

In this method the geometry of the structure shall be modified to increase the dead weight of the concrete without affecting the capacity of the reservoir. The calculation for the same is shown in Table 7.1 & 7.2

Let us assume

Bottom slab thickness	0.90 m
Top slab thickness	0.60 m
Walls thickness	0.45 m

Table 7.1: Modified geometry – Increasing member thickness

Modified Geometry of the UGR

L	w	h	Bottom Slab Thickness	Top slab Thickness	Wall thickness	L	W	H
12 m	10 m	4.50 m	0.90 m	0.60 m	0.45 m	12.90 m	10.90 m	6.00 m
Depth of overburden soil also increased from 0.40 m to 0.45 m								

Other parameters are same as in Table 6.4

***Table 7.2**: Stability Analysis – Increasing member thickness*

1. Total Weight of Concrete or RCC (Walls & Slabs) $[(L \times W \times H) - (l \times w \times h)] \times \gamma_r$		7591.50 kN
2. Total Weight of Concrete / RCC (Beams) No. of Beams of each side x length × size × γ_r		87.29 kN
3. Weight of overburden Soil L × W × Depth of earthfill × γ_s		1202.22 kN
4. Weight of opening $(\pi/4 \times d2 \times \text{Top slab thickness} \times \gamma_r) + (\pi/4 \times d2 \times \text{depth of earthfill} \times \gamma_s)$		6.66 kN
TOTAL DOWNWARD WEIGHT 1 + 2 + 3 – 4	**W_T =**	**8874.35 kN**
TOTAL UPWARD BUOYANT FORCE L × W × (H+Depth of earthfill- depth of water table) × γ_w	**W_B =**	**9069.35 kN**
FACTOR OF SAFETY W_T / W_B	**FS**	**0.98 < 1.2**
	Result	**Unstable**

The result shows that increasing of member thickness even by considerable amount also not sufficient to withstand the buoyant force although it increases the cost of the construction.

7.2 Solution-2: Add concrete fill to the inside of tank

To fill the concrete in the tank the geometry specially the depth is to be increased its dimensions so that the reservoir capacity could not be affected by pouring concrete. For that height of the tank calculation is shown in Table 7.3 & 7.4.

Table 7.3: Additional depth required - Add concrete fill to the inside of tank

Factor of Safety taken		1.2
Additional weight reqired to resist boyant force FS = (W_T/W_B) × Or W_T = FS × W_B W_T =		8201.86 kN
Total downward weight calculated for most critical condition, when reservoir is empty and ground water level is maximum (Table 6.5)		3361.42 kN
Additional weight required to resist buoyant force (W_{add})		4840.44 kN
Calculation of additional Depth Required		
1	Effective density of concrete $\gamma_{c,eff}$ = (γc-γw)	14 kN/m^3
2	Volume of additional concrete required = $V_{add} = W_{add} / \gamma_{c,eff}$	345.75 m^3
3	Present outer area (12.40 x 10.40)	128.96 m^2
4	Additional depth required ($h_{add} = V_{add}$ / External area of UGR (L×B)	2.68 m
	New Depth / Height = H + h_{add}	7.58 m

After calculating the new depth of reservoir, the modified geometry is shown in Table 7.4

Table 7.4: Modified geometry - Add concrete fill to the inside of tank

L	W	H	Bottom Slab Thickness	Top slab Thickness	Wall thickness	L	W	H
12 m	**10 m**	**7.18 m**	**0.2 m**	**0.2 m**	**0.2 m**	**12.40 m**	**10.40 m**	**7.58 m**
Depth of overburden soil also increased from 4.50 m to 0.60 m								

Now the depth of reservoir has been modified from 4.90 m to 7.58 m due to additional 2.68 m depth. In this additional depth, 321 cubic meter of concrete is required which is not an economical solution. But in this case we assume that we have no option left out against this and we can afford so much additional concrete, now we have to check in that condition whether the structure remains stable or not because in that case overall depth is increase, so accordingly the buoyant force also increases due to increased displaced volume of ground water as explained in Table 7.5.

Table 7.5: Stability check - Add concrete fill to the inside of tank

1. Total Weight of Concrete or RCC (Walls & Slabs) $[(L \times W \times H) - (l \times w \times h)] \times \gamma_r$	2898.15 kN
2. Total Weight of Concrete / RCC (Beams) No. of Beams of each side × length × size × γ_r	87.29 kN
3. Weight of overburden Soil L × W × Depth of earthfill × γ_s	1470.14 kN
4. Weight of opening (π/4 × d^2 × Top slab thickness × γ_r)+ (π/4 × d^2 × depth of earthfill × γ_s)	4.63 kN
5. Weight of concrete infill inside l × w × Additional Depth × γ_c	7721.36 kN
TOTAL DOWNWARD WEIGHT = 1 + 2 + 3 - 4 + 5 **W_T=**	**12172.31 kN**
TOTAL UPWARD BUOYANT FORCE L × W × (H + Depth of earthfill - depth of water table) × γ_w **W_B=**	**10550.25 kN**
FACTOR OF SAFETY W_T / W_B **FS**	**1.15 < 1.20**
Result	**Unstable**

From the above results required FS is not achieved even after increasing the depth and filling of additional concrete, so this solution is not feasible and another solution should be worked out.

7.3 Solution-3: Base slab extension

The base is extended to incorporate the weight of overburden soil over the base slab to add an additional weight to counter the buoyant force. This method has two advantages

(1) Overburden soil over the extended base of bottom slab take part in increasing downward force.

(2) Wedge action of the soil adds weight due to soil internal angle of soil friction.

Figure 7.1 shows the effect of internal angle of friction of soil by soil wedge action and weight soil on extended base.

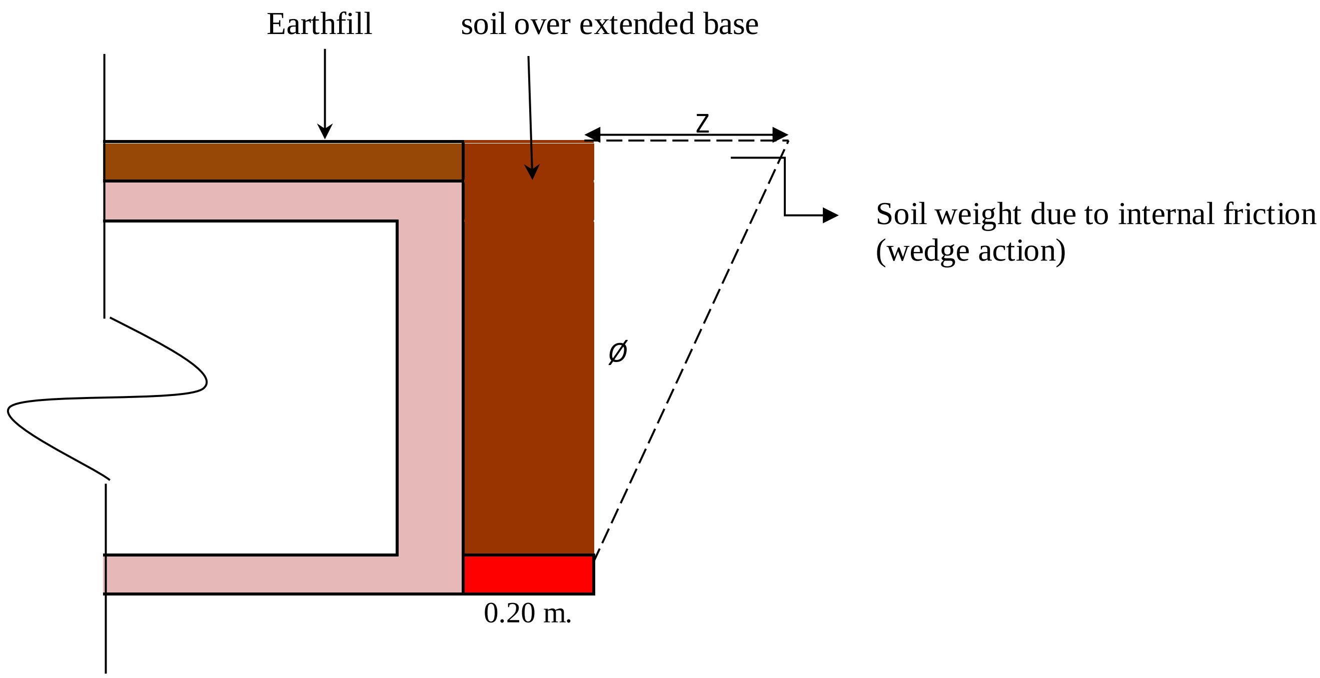


Figure 7.1: *Effect of soil wedge action due to angle of internal friction*

The analysis can be made by two ways:

(1) Considering the weight of overburden soil over the extended base of bottom slab only

(2) Weight of soil wedge due to soil internal angle of soil friction.

7.3.1 Considering the weight of overburden soil over the extended base of bottom slab only

Let us take our first case that we consider weight of soil on extended base and its effect to achieve the stability. Calculation of modified geometry and stability check is shown in Table 7.6 & 7.7

Let us assume

Thickness of bottom slab	0.60 m
Thickness of top slab	0.40 m
Thickness of walls	0.45 m
Base Extension	0.20 m both side

Table 7.6: *Modified Geometry – Base Extension*

l	w	h	Bottom Slab Thickness	Top slab Thickness	Wall thickness	L	W	H
12 m	**10 m**	**4.50 m**	**0.60 m**	**0.40 m**	**0.45 m**	**12.90 m**	**10.90 m**	**5.50 m**

Depth of overburden soil is 0.45 m

Other parameters are same as in Table 6.4

Table 7.7: *Stability Check – Base extension without soil wedge*

1. Total Weight of Concrete or RCC (Walls & Slabs) $[(L \times W \times H) - (l \times w \times h)] \times \gamma_r$	5833.88 kN
2. Total Weight of Concrete / RCC (Beams) No. of Beams of each side × length × size × γ_r	87.29 kN
3. Weight of overburden Soil L × W × Depth of earthfill × γ_s	1202.22 kN
4. Weight of opening $(\pi/4 \times d^2 \times$ Top slab thickness $\times \gamma_r) + (\pi/4 \times d^2 \times$ depth of earthfill $\times \gamma_s)$	5.24 kN
5. Buoyant Weight of soil engaged by extension Density of buoyant soil $= \gamma_s - \gamma_w = 19-10$ Perimeter of Bottom slab = 13.30+10.90+13.30+10.90 = 48.40m W_5 = Perimeter of Bottom slab × extended base width × (h+ top slab thickness + Earthfill) × density of buoyant soil = W_5	9 kN/m^3 466.09 kN
TOTAL DOWNWARD WEIGHT = 1 + 2 + 3 - 4 + 5 **W_T =**	**7584.23 kN**
TOTAL UPWARD BUOYANT FORCE L × W × (H+Depth of earthfill- depth of water table) × γ_w **W_B =**	**8366.30 kN**
FACTOR OF SAFETY $= W_T / W_B$ FS	**0.91 < 1.20**
Result	**Unstable**

It can be seen that only weight of overburden soil over extended base does not affect the result. Since Factor of safety without considering the soil wedge is not sufficient to make the structure stable. Now we will take another case of the weight due to wedge action because of soil friction.

7.3.2 Weight of soil wedge due to soil internal angle of soil friction.

Due to soil internal angle of friction, there is wedge develop on the extended base, the effect of soil wedge helps to increase the dead eight of the structure

Table 7.8: *Stability Check – Base extension considering the soil wedge action*

Assume angle of internal friction (On conservative side as mentioned above)	**20** degree 0.34906585 radians
From figure 7.1 z = tan $20^0 \times$ (h + Top slab thickness + Earthfill)	1.95 m
Outer perimeter with extension = **2 x (L+ Z + W +Z)** Volume of wedge = 1/2 × height × z × perimeter Density of buoyant soil = $\gamma_s - \gamma_w$ = 19-10 =	55.39 m 288.51 m^3 9.00 kN/m^3
Weight of soil wedge due to base extension W_e =Volume × Density of buoyant soil	2596.62 kN
Add this weight to total Downward Force **$W_T + W_e$**	**10180.85 kN**
Factor of Safety due to soil wedge **FS**	**1.22 >1.20**
Result	**Stable**

In Table 7.8, it is observed that the weight of soil wedge increase the downward gravity weight. This weight helps keeping the buoyant weight under control and the structure remain stable.

7.4 Solution-4: Anti-Floatation Slab

In this method as already mentioned earlier, the structure is attached to a bigger concrete slab than the bottom slab of the structure which may be precast or cast-in-situ. The input parameters of modified geometry and its stability check are shown in Table 7.9 & 7.10.

Let us assume

Thickness of bottom slab	0.60 m
Thickness of top slab	0.60 m
Thickness of walls	0.50 m
Base Extension	0.30 m both side
Thickness of additional slab	0.60 m

Table 7.9: *Modified geometry – Anti- floatation slab*

l	w	h	Bottom Slab Thickness	Top slab Thickness	Wall thickness	L	W	H
12 m	10 m	4.50 m	0.60 m	0.60 m	0.50 m	13.00 m	11.00 m	5.70 m

Depth of overburden soil increased from 0.45 m to 0.60 m

Other parameters are same as in Table 6.4

Table 7.10: *Stability check – Anti- floatation slab*

1. Total Weight of Concrete or RCC (Walls & Slabs) $[(L \times W \times H) - (l \times w \times h)] \times \gamma_r$	6877.50 kN
2. Total Weight of Concrete / RCC (Beams) No. of Beams of each side × length × size × γ_r	87.29 kN
3. Weight of overburden Soil L x W x Depth of earthfill x γs	1630.20 kN
4. Weight of opening $(\pi/4 \times d^2 \times$ Top slab thickness $\times \gamma_r) + (\pi/4 \times d^2 \times$ depth of earthfill $\times \gamma_s)$	7.46 kN
5. Boyant Weight of soil engaged by extension Density of buoyant soil $= \gamma_s - \gamma_w$ =19-10 Perimeter of Bottom slab = 13.30+10.90+13.30+10.90 = 48.60m W_5 = Perimeter of Bottom slab × extended base width × (h+ top slab thickness +Earthfill) × density of buoyant soil = W_5 W_5 =	9.00 kN 826.69 kN
TOTAL DOWNWARD WEIGHT = 1 + 2 + 3 - 4 + 5 **W_T =**	**9414.21 kN**
TOTAL UPWARD BUOYANT FORCE L × W × (H + Depth of earthfill- depth of water table) × γ_w **W_B =**	**9009.00 kN**
FACTOR OF SAFETY = W_T / W_B **FS**	**1.04 < 1.20**
Result	**Unstable**

It is seen that mere changing of dimensions not sufficient to achieve the targeted safely factor. As explained in the previous chapter, make an anti-floatation slab slightly larger than the bottom slab of UGR, results can be modified as shown in table 7.11.

Table 7.11: *Stability Check – Anti-floatation slab with extended base*

Adding an anti-floatation slab larger than the structure by 0.30 m all around

Additional Downward Force due to anti-floatation slab	
(L+2e) × (W+2e) ×Thickness of additional slab × buoyant density of slab material **We**	**1419.84 kN**
Add this weight to total Downward Force (W_{T1}) **W_T+We**	**10834.05 kN**
Factor of Safety due to anti floatation slab **FS**	**1.20**
Result	**Stable**

Once the analysis is safe then we have to design connections for connecting anti floatation slab to the main structure. To design the tie down connections it is require estimating that for how much force connection has to be design. Figure 7.2 explain the connection force and how the anti-floatation slab will anchor to the main structure. The connection force for mechanical connection between anchored slab and the bottom slab of the structure is given by

$$\textbf{Connection force} = (W_B \times FS) - W_T = 1396.59 \text{ KN} \qquad \text{(from Eq.4)}$$

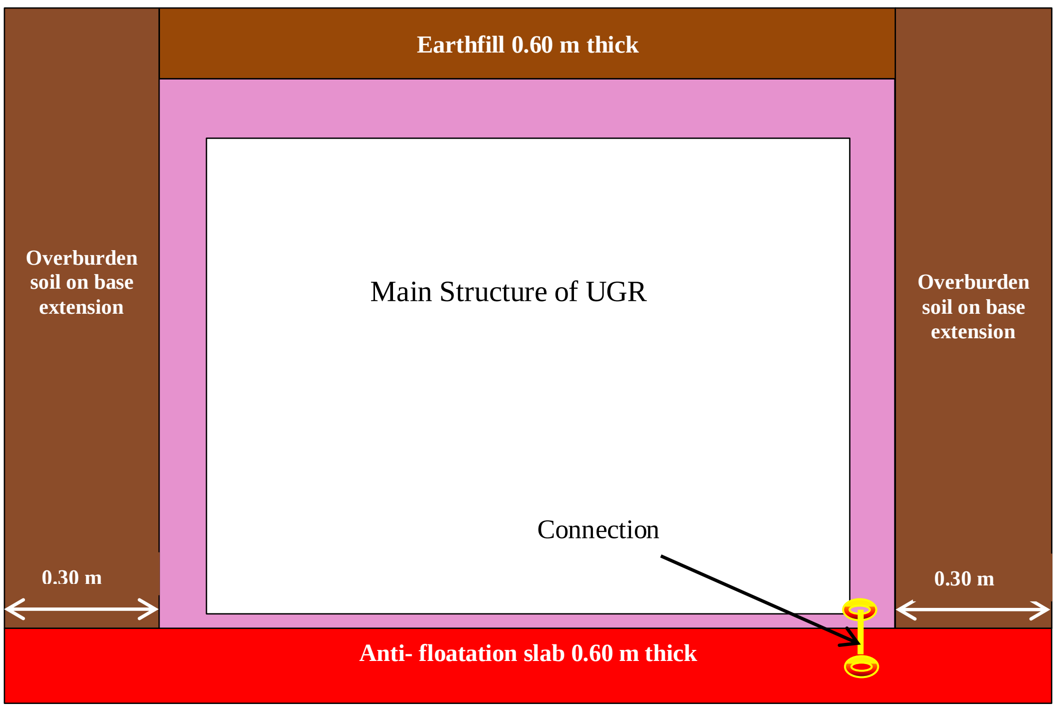


Figure 7.2: *Connection for anchoring anti floatation slab with main structure*

CHAPTER-8

MODELING AND ANALYSIS OF UGR USING STAAD Pro

In previous chapters, it has been observed that the stability of the structure is achieved for the most critical condition i.e.; when tank is empty and ground water table is maximum, by applying two different countermeasures as mentioned below:

Case-1: Add extension to the outside of base slab to engage the weight of soil wedge due to angle of internal friction of soil.

Case-2: Add a separate anti floatation slab below the base slab.

Now, by using STAAD Pro software the strength analysis has been done by analyzing the structure for both the case and find different values of node displacement, shear stress and bending moment in X & Y directions.

The structure has been modeled considering it a plate model. Top and bottom slabs are modeled by generating plate mesh of 12 nos. & 10 nos. on 12 m and 10 m side respectively. The walls have been modeled by generating plate mesh of 12 nos. × 5 nos. on side 12 m and 4.50 m and 10 nos. × 5 nos. on side 10 m × 4.50 m respectively. Extended base has been modeled separately due to unsymmetrical dimension to generate mesh.

8.1 Case-1 Add extension to the outside of base slab to engage the weight of soil wedge due to angle of internal friction of soil.

8.1.1 Modeling

Dimension of the tank is given in Table 8.1. By using these dimensions, the UGR is modeled in STAAD Pro as shown in Figure 8.1 and 3D view of the UGR is shown in Figure 8.2.

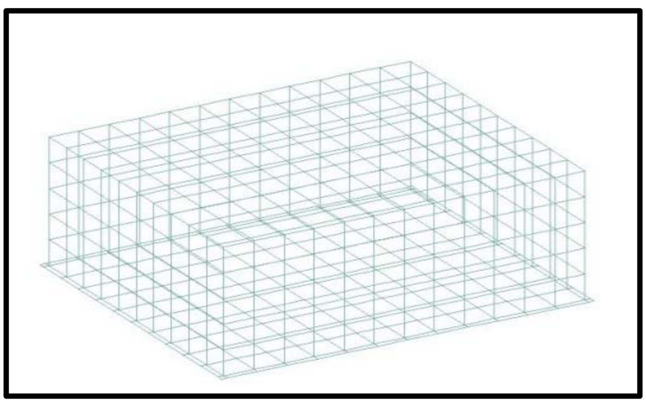


Figure8.1: *Modeling of UGR for case - 1*

Table 8.1: *Geometrical inputs for Case-1*

Tank Dimension	12 m × 10 m × 4.50 m
Thickness of bottom slab	0.60 m
Thickness of top slab	0.40 m
Thickness of walls	0.45 m
Base Extension	0.20 m all around
Angle of internal friction of soil Ø	20 degree
Earthfill on top slab	0.45 m

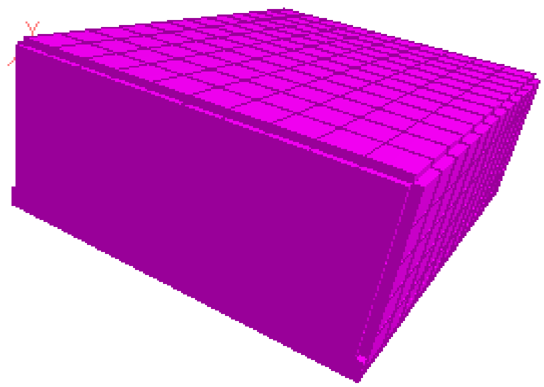


Figure 8.2: 3-D view of UGR for case-1

8.1.2 Defining properties

After modeling of the structure for case-1, the geometrical attributes and properties for the components has been given to the structure in STAAD Pro separately for top and bottom slab and for walls. Assigning thickness to the top slab, bottom slab and walls are shown in Figure 8.3, 8.4 & 8.5.

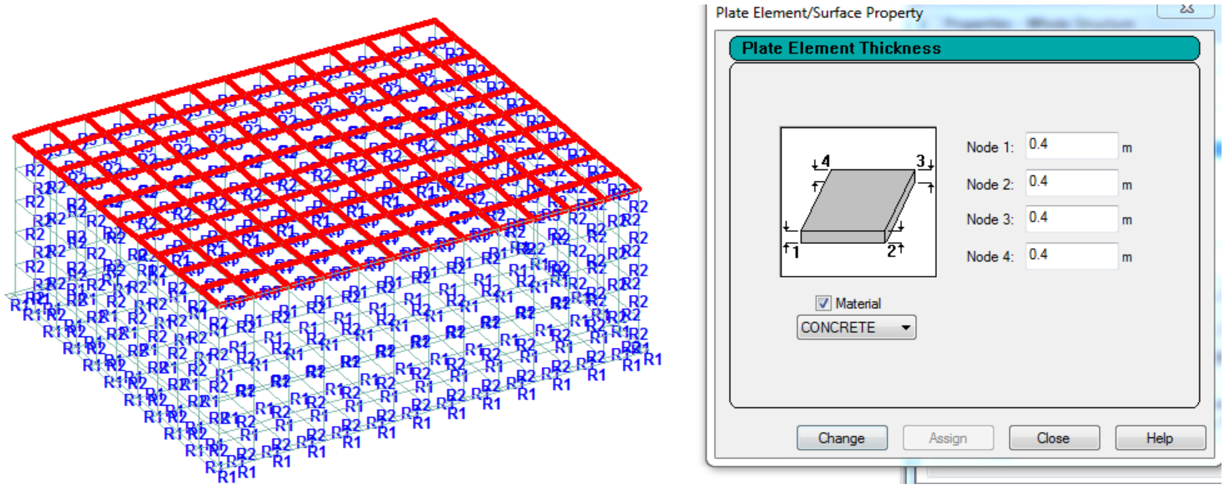


Figure 8.3: *Assigning plate element properties for the top slab - Case-1*

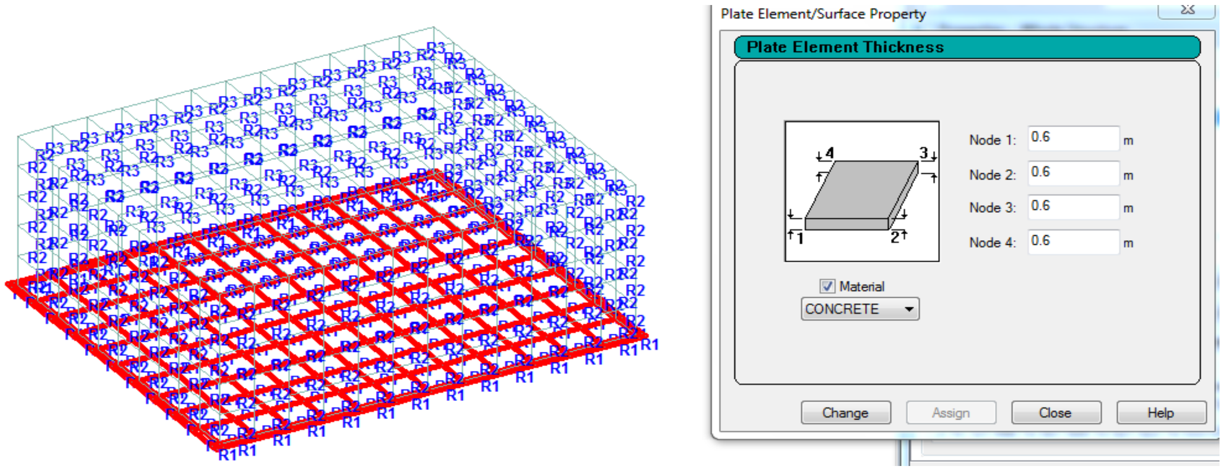


Figure 8.4: *Assigning plate element properties for the bottom slab - Case-1*

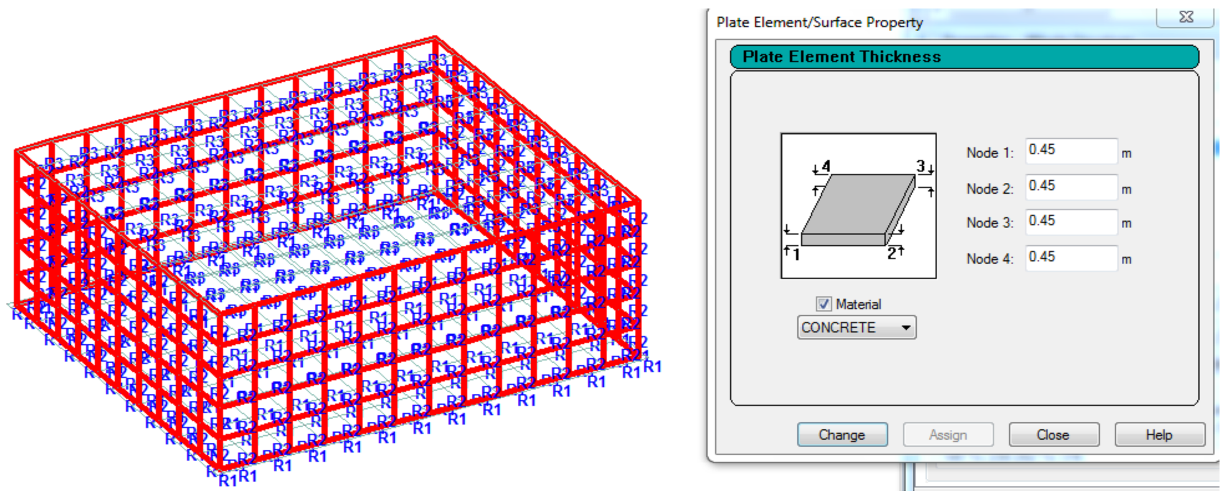


Figure 8.5: *Assigning plate element properties for the walls - Case-1*

8.1.3 Assigning of support

As we know that an underground reservoir does not need any foundation and they rest on the ground directly. The bearing capacity data of all the bores for 50 mm permissible settlement has been investigated. To assign the support we have to find out the soil subgrade modulus from the given data.

$$Soil\ Subgrade\ Modulus\ (K) = \frac{Net\ safe\ bearing\ capacity\ of\ soil}{Permissible\ settlement} \quad (11)$$

It is measured in kN/m^2 per meter depth of settlement. In this study, the permissible settlement is assumed 50 mm. The net safe bearing capacity of soil for shear & permissible settlement of 50 mm for different bore holes are obtained from the field study and shown in Table 8.2.

Table 8.2: Net safe bearing capacity for permissible settlement of 50 mm

Bore Hole	Depth (m)	Net Safe Bearing Capacity of soil (kN/m^2)		
		Shear	Settlement	Recommended
1 and 2	5.00	197	135	130
3,4 and 5	5.00	215	145	130

It is clear from the table that, the recommended net safe bearing capacity of soil for permissible settlement of 50 mm for all bore holes is 130 kN/m^2.

Therefore, soil subgrade modulus $(K) = \frac{130}{0.05}$ = 2600 kN/m^2 per meter depth of settlement

Value of soil subgrade modulus is entered in STAAD Pro for the support on the bottom slab assigned is shown in Figure 8.6

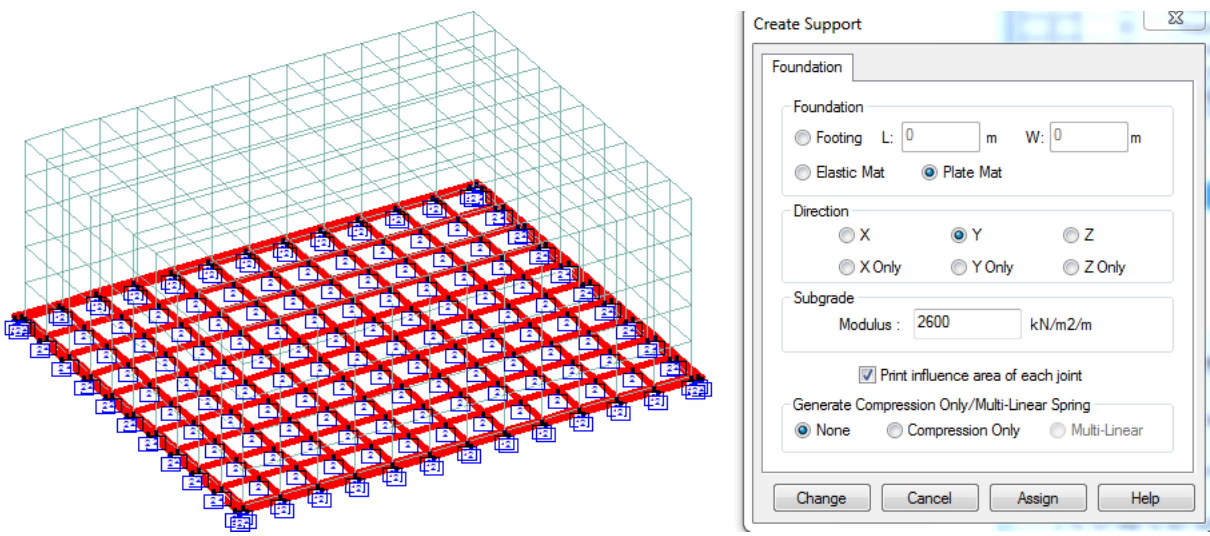


Figure 8.6: *Assigning support (plate mat- soil subgrade modulus)on bottom plate – Case – 1*

8.1.4 Generation & Assigning of Loads

The nature of load and where it is acted in the structure are given in the Table 8.3 Following loads as shown in table have been generated and assigned to the structure. Different types of load acting on the respective components of the structure are shown in Figure 8.7 (a) to (f)

Table 8.3: *Nature of load acting on different components of UGR for case-1*

Sr. No.	Nature of load	Acting on Structure Component
1	Self-weight	All components of structure
2	Weight of submerged soil	On extended base
3	Weight of earthfill	On top slab
4	Water pressure outside of structure	On side walls
5	Earth Pressure at Rest (KaγH) , Ka = 1-Sin Ø	On Side Walls
6	Weight due to soil internal friction	On Extended Base
7	Buoyant Force	On Bottom Slab (upward)

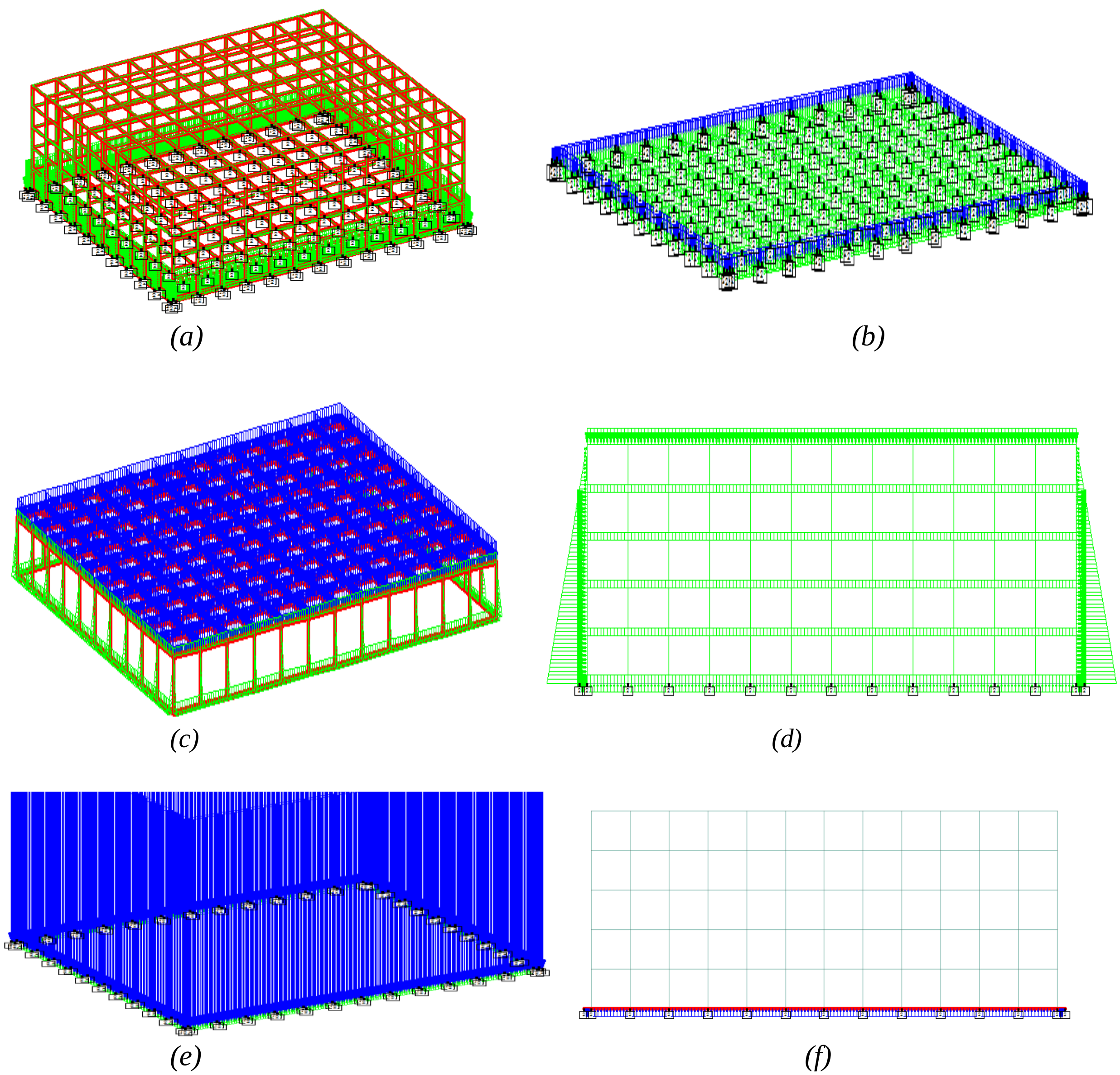


Figure 8.7: *Loads– case– 1 (a) Self weight, (b) wt. of soil on extended base, (c) wt. of overburden soil (d) water pressure and earth pressure at rest, (e) Wt. of soil wedge due to soil internal friction, (f) Buoyant pressure due to water table acting upward*

8.1.5 Post-processing - Analysis of Result

After complete modeling, the structure has been analyzed and the maximum node displacement, shear stress & bending moment both in X & Y directions is obtained

8.1.5.1 Node Displacement

The node displacements for front and back walls and side wall are shown in Figure 8.8 & 8.9.

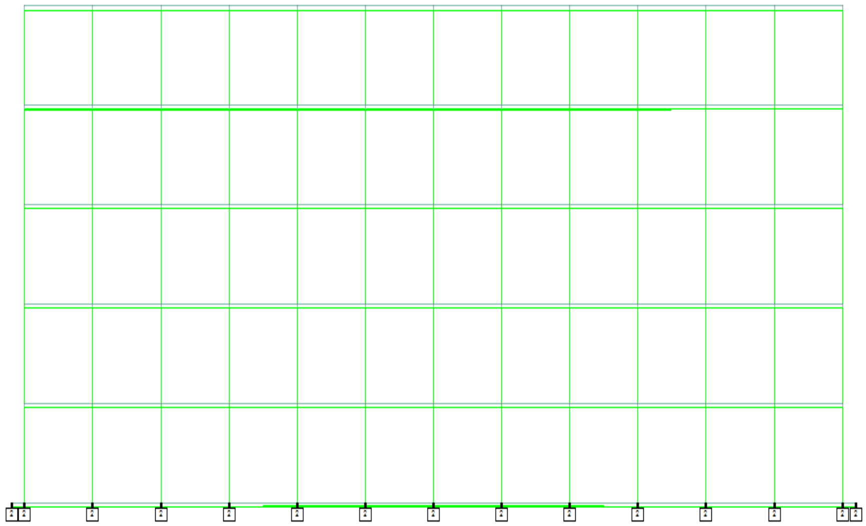


Figure 8.8: *Displacement of front and back wall of UGR for Case - 1*

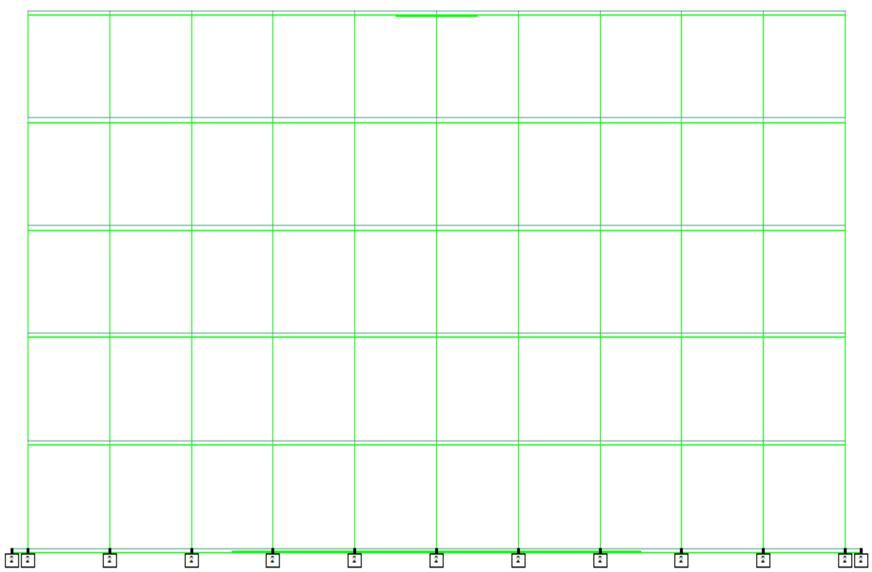


Figure 8.9: *Displacement of side wall plates of UGR for Case - 1*

Different values of nodes displacement for case-1 are shown in Table 8.4. This table shows that the maximum displacement in the whole structure is occur at node 439 which is at the center of top slab. This displacement is below the average settlement of 50 mm

Table 8.4: *Node Displacement for Case-1*

		Horizontal	**Vertical**	**Horizontal**	**Resultant**	**Rotational**		
	Node	**X mm**	**Y mm**	**Z mm**	**mm**	**rX rad**	**rY rad**	**rZ rad**
Max X	253	1.027	-41.218	0.065	41.231	0	0	0
Min X	213	-1.14	-41.567	0.065	41.582	0	0	0
Max Y	74	0	-33.552	0	33.552	0	0	0
Min Y	439	-0.139	-48.003	0.162	48.003	0	0	0
Max Z	308	-0.055	-41.521	1.51	41.549	0	0	0
Min Z	357	-0.056	-41.179	-1.378	41.202	0	0	0
Max rX	69	0	-39.282	0	39.282	0.002	0	0
Min rX	78	0	-39.013	0	39.013	-0.002	0	0
Max rY	324	-0.019	-41.46	0.6	41.464	0	0	0
Min rY	292	-0.091	-41.692	0.601	41.696	0	0	0
Max rZ	129	0	-39.656	0	39.656	0	0	0.002
Min rZ	15	0	-39.371	0	39.371	0	0	-0.002
Max Rst	439	-0.139	-48.003	0.162	48.003	0	0	0

8.1.5.2 Shear Stress & Bending Moments

The maximum and minimum values of shear stress and bending moment are shown in the form of contour in X and Y directions are shown in figure 8.10, 8.11, 8.12 & 8.13. The maximum value of shear stress in X & Y direction is 0.290 N/mm^2& 0.454 N/mm^2 from figure 8.10 & 8.11 respectively. The maximum value of bending moment in X & Y direction is 386 kNm per meter & 295 kNm per meter from figure 8.12 & 8.13 respectively.

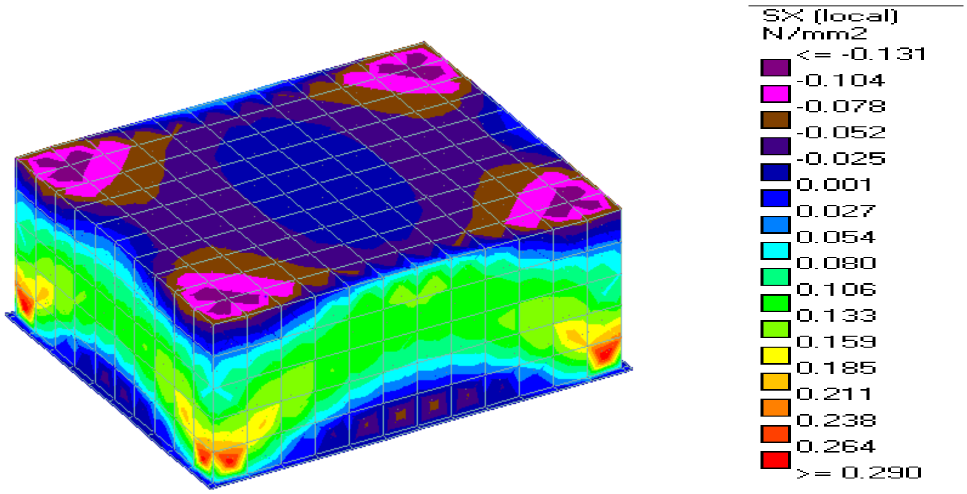


Figure 8.10: *Stress contour in X direction for Case-1*

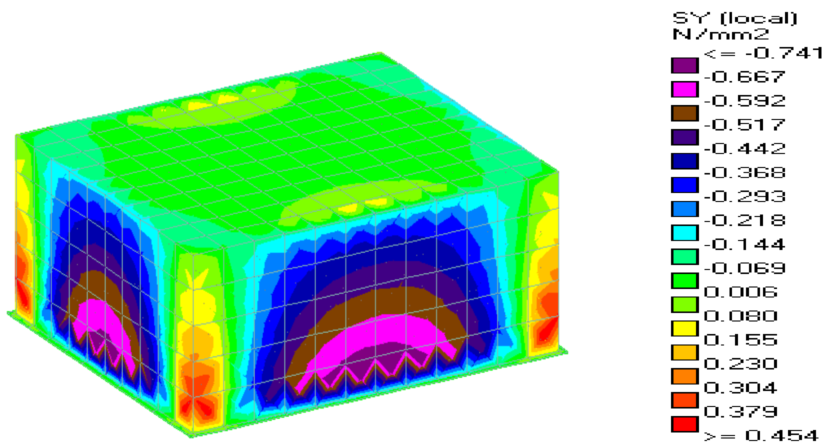


Figure 8.11: *Stress contour in Y direction for Case-1*

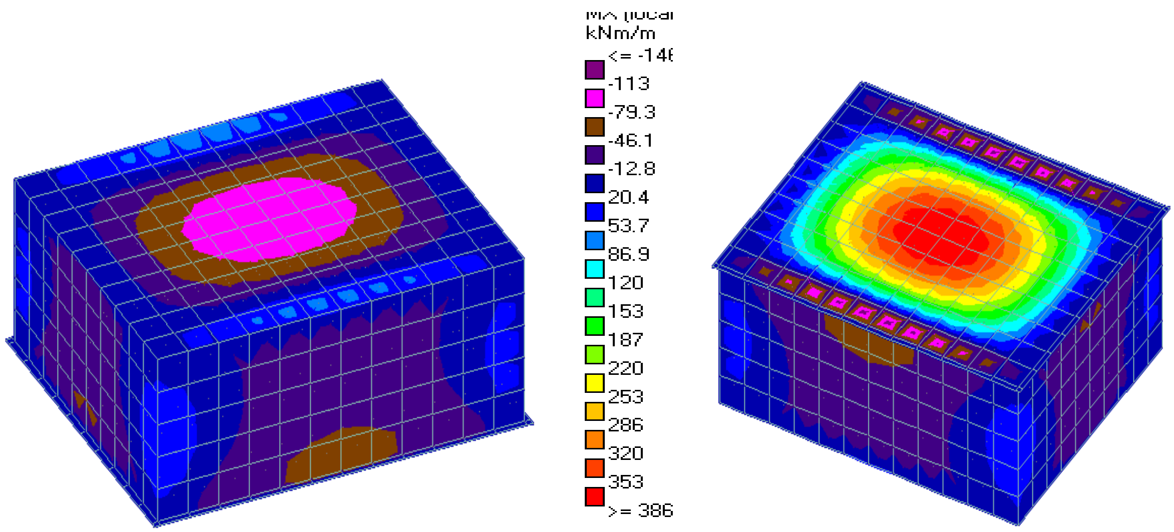


Figure 8.12: *Moment contour in X direction for Case-1*

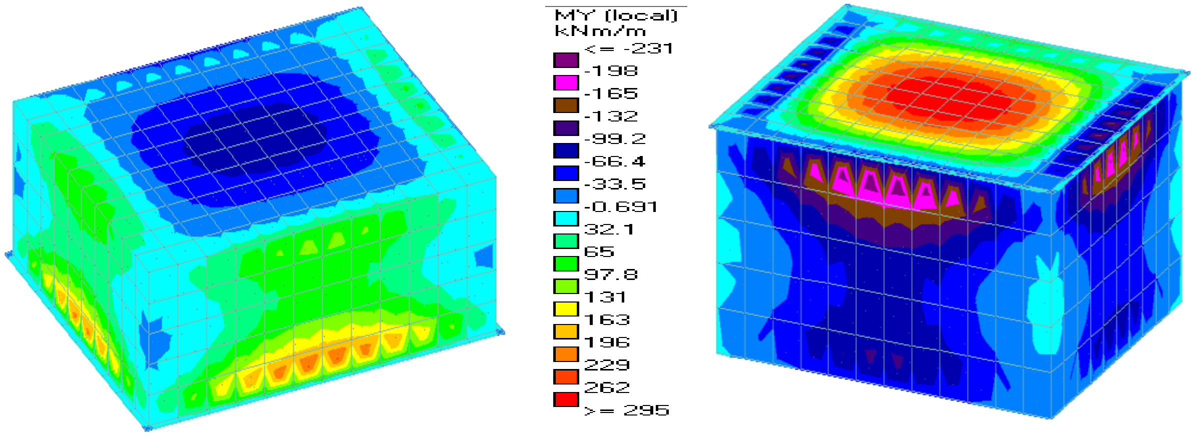


Figure 8.13: *Moment contour in Y direction for Case-1*

The values of different shear stress and bending moments on different plates are given in tabular form in Table 8.5.

Table 8.5: *Shear stresses and bending moments for Case – 1*

		Shear		Membrane			Bending Moment		
	Plate	SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm^2	SY (local) N/mm^2	SXY (local) N/mm^2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Qx	532	0.849	-0.087	0	0	0	8.311	3.105	-62.586
Min Qx	525	-0.85	-0.087	0	0	0	8.39	3.09	62.662
Max Qy	545	0.091	0.886	0	0	0	3.452	7.308	64.868
Min Qy	536	0.092	-0.884	0	0	0	3.468	7.223	-64.737
Max Sx	252	0.195	-0.268	0.290	0.453	0.154	-12.77	42.79	-22.711
Min Sx	377	0.061	0.061	-0.131	-0.115	0.126	13.154	13.837	24.079
Max Sy	312	-0.195	0.268	0.29	0.454	0.152	12.735	-42.737	22.654
Min Sy	277	0.014	-0.297	-0.071	-0.741	0.015	54.469	230.846	-7.272
Max Sxy	258	0.11	-0.111	0.196	-0.149	0.211	-5.432	28.747	-10.429
Min Sxy	303	-0.11	-0.11	0.195	-0.147	-0.212	-5.474	28.656	10.416
Max Mx	66	0.046	-0.024	0	0	0	386.144	294.799	4.871
Min Mx	71	-0.567	0.019	0	0	0	-145.794	7.079	-23.133
Max My	66	0.046	-0.024	0	0	0	386.144	294.799	4.871
Min My	337	-0.014	0.297	-0.071	-0.738	0.017	-54.394	-230.518	7.254
Max Mxy	103	0.15	-0.157	0	0	0	40.452	29.088	185.304
Min Mxy	110	-0.15	-0.156	0	0	0	39.989	28.855	-185.117

8.2 Case-2: Add a separate anti floatation slab below the base slab.

The same procedure has been followed for case-2 as mentioned for case-1 above with different inputs of geometry and loads acting on different components of UGR.

8.2.1 Modeling

There is no provision of modeling a plate on another plate in STAAD Pro. An alternative approach is used to join the anti- floatation slab with the bottom slab of tank by means of dummy columns having 0 densities, 0.30 m high and with no property other than shape. These columns act as connectors and have no role in sharing & transferring of loads. Dimension of the tank is given in Table 8.6. By using these dimensions, the UGR is modeled in STAAD Pro as shown in Figure 8.14 and 3D view of the UGR is shown in Figure 8.15.

Table 8.6: *Geometrical inputs for Case-2*

Tank Dimension	12 m x 10 m x 4.50 m
Thickness of bottom slab	0.60 m
Thickness of top slab	0.60 m
Thickness of walls	0.50 m
Base Extension	0.30 m all around
Thickness of additional slab	0.60 m
Earthfill on top slab	0.60 m

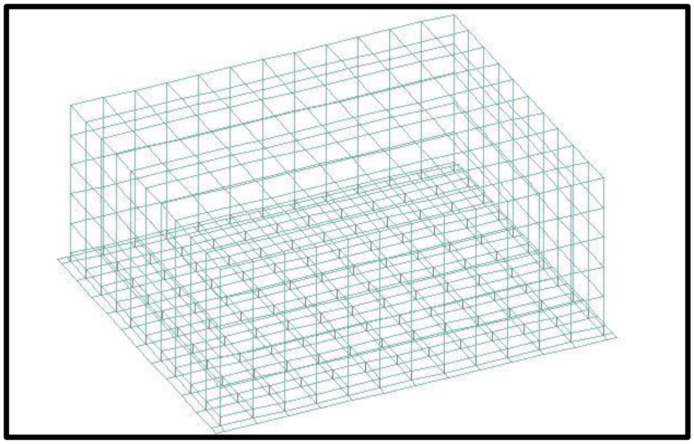


Figure 8.14: *Modeling of UGR for Case – 2*

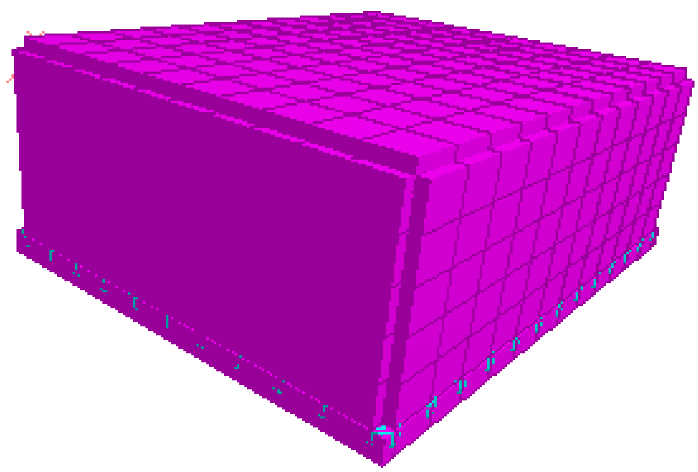


Figure 8.15: *3-D View of UGR for Case-2*

8.2.2 Defining properties

After modeling of the structure for case-2, the geometrical attributes and properties for the components has been given to the structure in STAAD Pro simultaneously for top and bottom slab being the same thickness and individually for walls. Assigning thickness to the top slab, bottom slab and walls are shown in Figure 8.16 & 8.17.

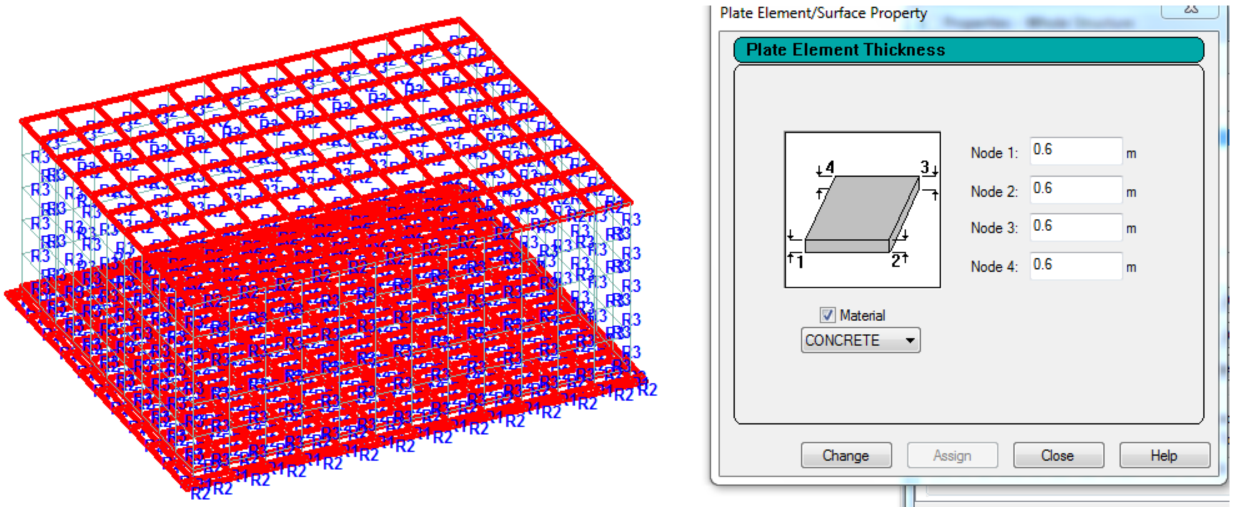


Figure 8.16: *Assigning plate element properties for the top & bottom slab - Case-2*

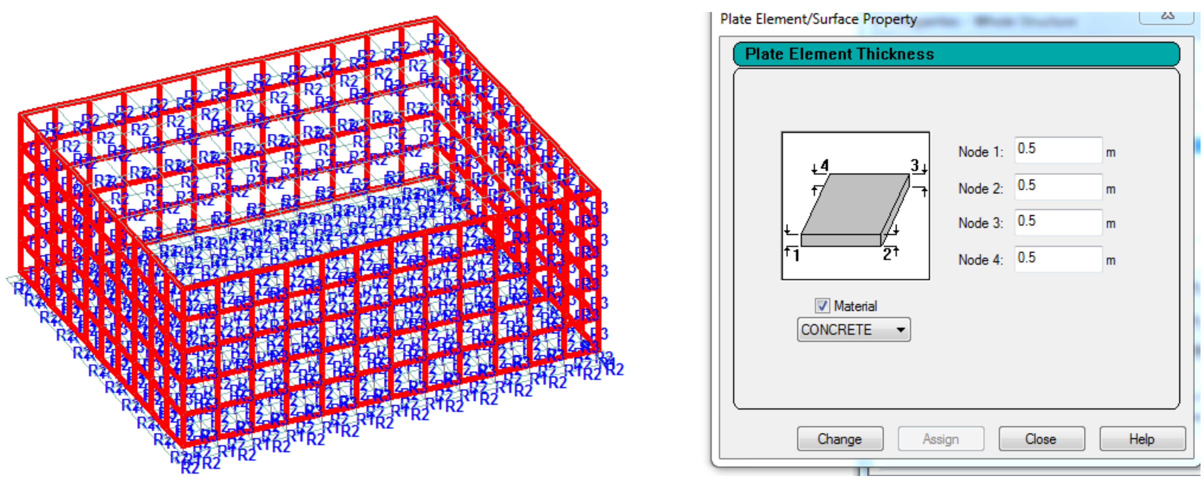


Figure 8.17: *Assigning plate element properties for the walls - Case-2*

8.2.3 Assigning of support

The method for assigning support is exactly same as mentioned in 8.1.3. In this case also, the value of soil subgrade modulus is same i.e.; 2600 kN/m2 per meter depth of settlement. Value of soil subgrade modulus entered in STAAD Pro for the support on the bottom slab assigned is shown in Figure 8.6

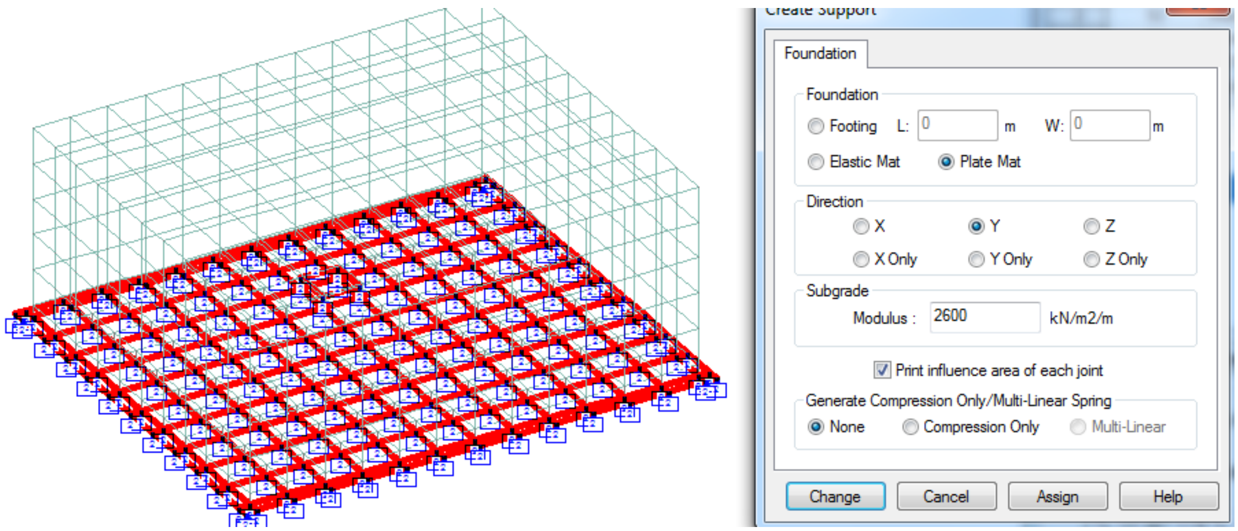


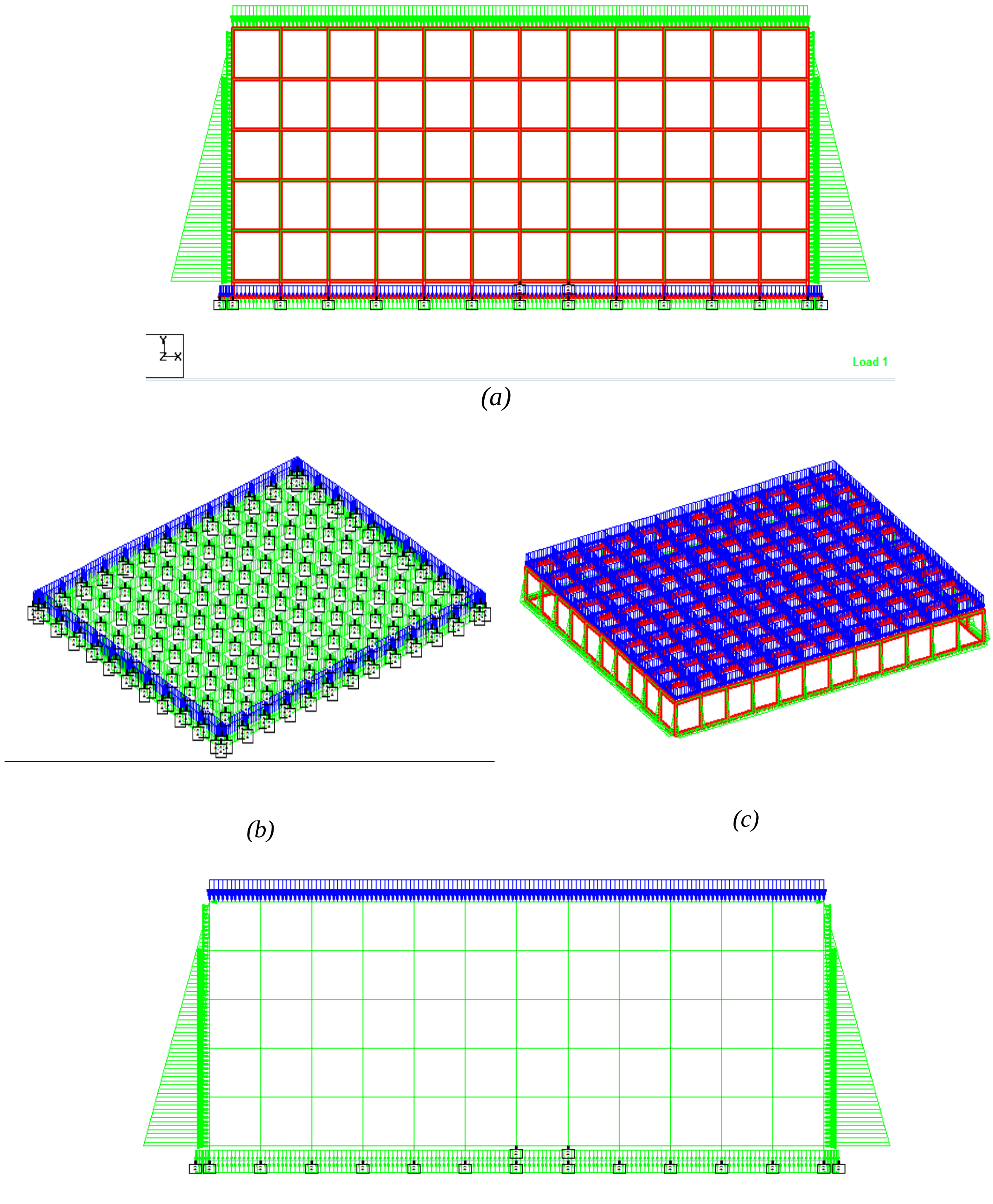
Figure 8.18: *Assigning support (plate mat- soil subgrade modulus)on bottom plate – Case – 2*

8.2.4 Generation & Assigning of Loads

The nature of load and where it is acted in the structure are given in the Table 8.7 Following loads as shown in table have been generated and assigned to the structure. Different types of load acting on the respective components of the structure are shown in Figure 8.19 (a) to (d)

Table 8.7: *Nature of load acting on different components of UGR for case-2*

Sr. No.	Nature of load	Acting on Structure Component
1	Self-weight	All components of structure
2	Weight of submerged soil	On extended base
3	Weight of earthfill	On top slab
4	Water pressure outside of structure	On side walls
5	Earth Pressure at Rest (KaγH) , Ka = 1-Sin Ø	On Side Walls
6	Buoyant Force	On Bottom Slab (upward)



(d)

Figure 8.19: *Loads – case– 2(a) self-weight, (b) wt. of soil on extended base, (c) wt. of overburden soil (d) Water pressure and earth pressure at rest.*

8.2.5 Post-processing - Analysis of Result

After complete modeling, the structure has been analyzed and the maximum node displacement, shear stress & bending moment both in X & Y directions is obtained

8.2.5.1 Node Displacement

The node displacements for front and back walls and side wall are shown in Figure 8.20 & 8.21.

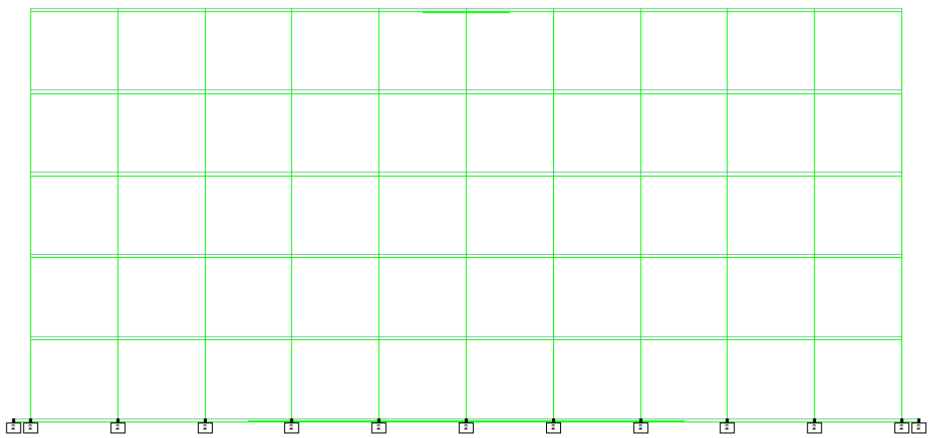


Figure 8.20: *Displacement of front and back wall of UGR for Case – 2*

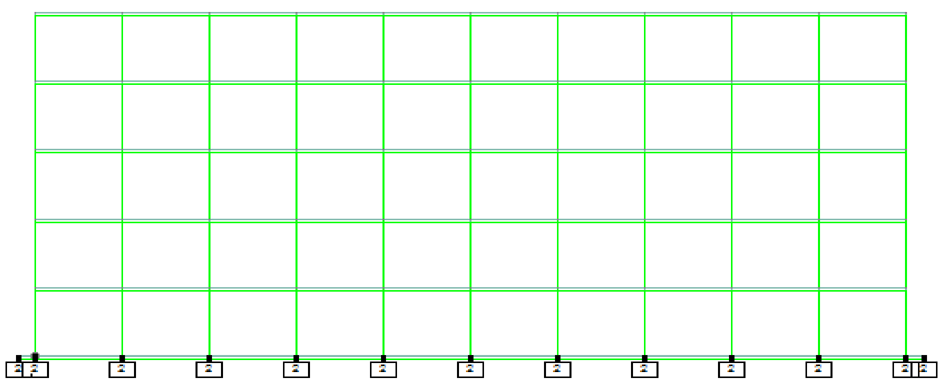


Figure 8.21: *Displacement of side wall plates of UGR for Case - 2*

Different values of nodes displacement for case-1 are shown in Table 8.8. This table shows that the maximum displacement in the whole structure is occur at node 914 which is at the center of top slab. This displacement is below the average settlement of 50 mm.

Table 8.8: *Node Displacement for Case-2*

		Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	**X mm**	**Y mm**	**Z mm**	**mm**	**rX rad**	**rY rad**	**rZ rad**
Max X	1048	0.119	-26.77	0.399	26.773	0	0	0
Min X	1192	-0.233	-26.951	0.519	26.957	0	0	0
Max Y	75	0	-25.62	0	25.62	0	0	0
Min Y	914	-0.071	-29.37	0.639	29.377	0	0	0
Max Z	1107	-0.057	-27.505	0.764	27.515	0	0	0
Min Z	756	-0.008	-25.992	-0.022	25.992	0	0	0
Max rX	918	-0.071	-27.002	0.653	27.01	0.001	0	0
Min rX	1009	-0.07	-28.029	0.625	28.036	-0.001	0	0
Max rY	1098	-0.035	-27.444	0.528	27.449	0	0	0
Min rY	1092	-0.053	-27.528	0.528	27.533	0	0	0
Max rZ	882	-0.082	-27.966	0.639	27.974	0	0	0.001
Min rZ	946	-0.059	-28.078	0.639	28.086	0	0	-0.001
Max Rst	914	-0.071	-29.37	0.639	29.377	0	0	0

8.2.5.2 Shear Stress & Bending Moments

The maximum and minimum values of shear stress and bending moment are shown in the form of contour in X and Y directions are shown in figure 8.22, 8.23, 8.24 & 8.25. The maximum value of shear stress in X & Y direction is 0.628 N/mm^2& 0.507 N/mm^2 from figure 8.22 & 8.23 respectively. The maximum value of bending moment in X & Y direction is 78.7 kNm per meter & 104 kNm per meter from figure 8.24 & 8.25 respectively.

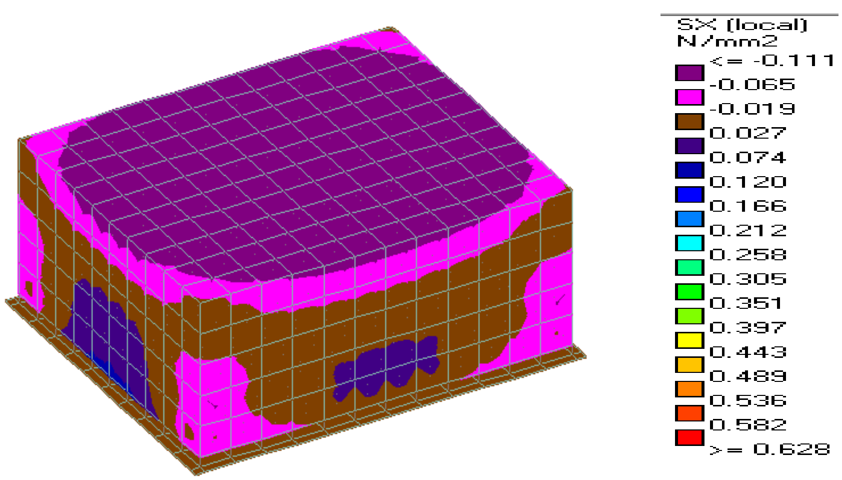


Figure 8.22: *Stress contour in X direction for Case-2*

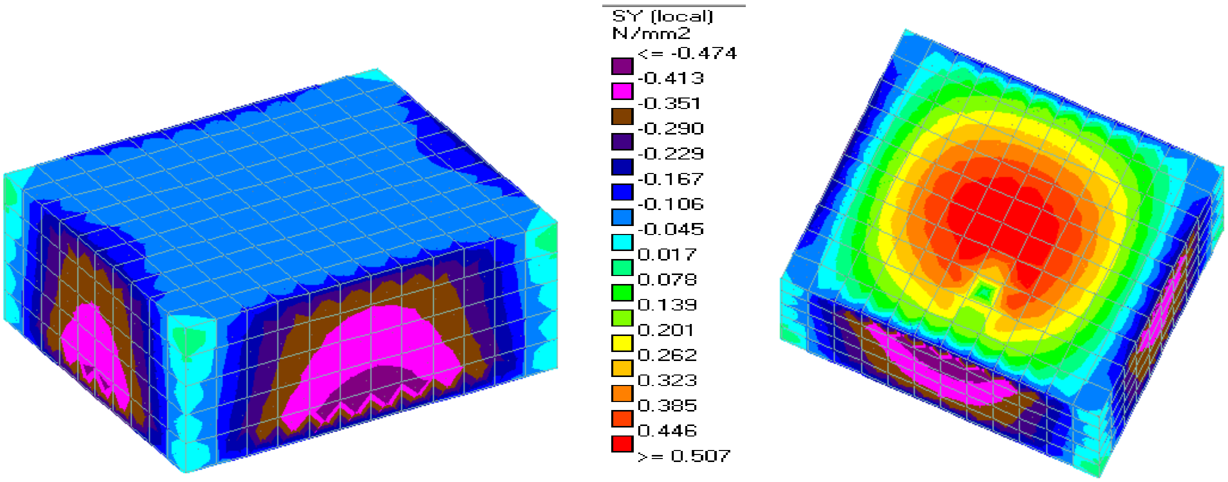


Figure 8.23: *Stress contour in Y direction – Case – 2*

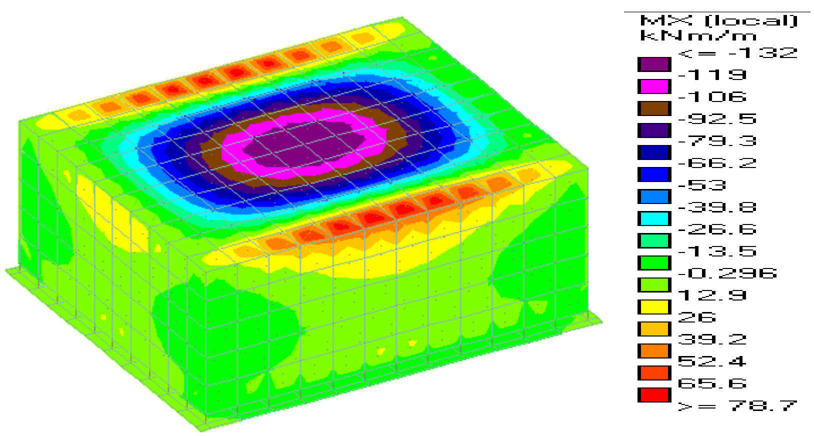


Figure 8.24: *Moment contour in X direction for Case-2*

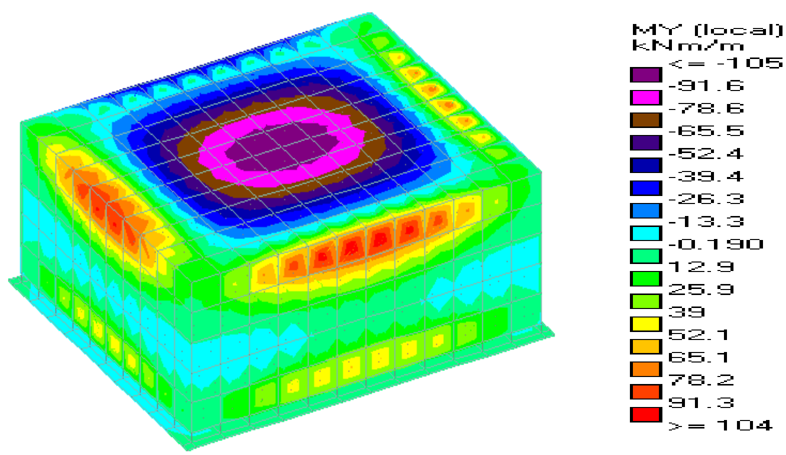


Figure 8.25: *Moment contour in Y direction for Case-2*

The values of different shear stress and bending moments on different plates are given in tabular form in Table 8.9.

Table 8.9: *Shear stresses and bending moments for Case – 2*

	Plate	Load Case	Shear		Membrane			Bending Moment		
			SQX (local) N/mm^2	SQY (local) N/mm^2	SX (local) N/mm2	SY (local) N/mm^2	SXY (local) N/mm^2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Qx	791	(DL) DEAD WEIGHT	0.214	0.015	-0.111	-0.059	-0.003	78.739	3.463	-6.782
Min Qx	782	(DL) DEAD WEIGHT	-0.214	0.015	-0.111	-0.067	0.004	78.305	3.203	6.797
Max Qy	846	(DL) DEAD WEIGHT	0.018	0.198	-0.093	-0.112	-0.003	-1.105	73.26	-8.522
Min Qy	736	(DL) DEAD WEIGHT	0.018	-0.198	-0.094	-0.112	0.004	-1.12	73.251	8.523
Max Sx	674	(DL) DEAD WEIGHT	0.01	-0.007	0.628	0.503	0.012	51.534	39.491	0.627
Min Sx	801	(DL) DEAD WEIGHT	0.214	-0.015	-0.111	-0.06	0.005	78.716	3.434	6.809
Max Sy	665	(DL) DEAD WEIGHT	-0.017	0.007	0.61	0.507	0.008	50.862	39.516	0.698
Min Sy	971	(DL) DEAD WEIGHT	0.002	0.146	0.02	-0.474	-0.004	-12.781	-51.166	-1.659
Max Sxy	711	(DL) DEAD WEIGHT	0.042	-0.043	0.055	0.03	0.24	7.926	6.204	26.426
Min Sxy	621	(DL) DEAD WEIGHT	0.042	0.043	0.054	0.03	-0.24	7.854	6.126	-26.397
Max Mx	791	(DL) DEAD WEIGHT	0.214	0.015	-0.111	-0.059	-0.003	78.739	3.463	-6.782
Min Mx	786	(DL) DEAD WEIGHT	-0.019	-0.01	-0.09	-0.081	0.001	-132.021	-100.452	1.781
Max My	958	(DL) DEAD WEIGHT	0.001	0.14	-0.059	-0.334	-0.005	24.705	104.319	-2.256
Min My	1019	(DL) DEAD WEIGHT	-0.001	-0.141	-0.051	-0.336	-0.004	-24.772	-104.7	2.244
Max Mxy	840	(DL) DEAD WEIGHT	0.057	0.058	-0.08	-0.074	0.033	-5.003	-1.429	59.612
Min Mxy	750	(DL) DEAD WEIGHT	0.057	-0.058	-0.08	-0.073	-0.032	-4.994	-1.403	-59.613

The summary of both the case-1 & case-2 showing all the parameters related to geometry of the structure, node displacement, shear stress and bending moment is shown in Table 8.10

Table 8.10: *Summary sheet - Analysis result for both the cases*

Sr. No.	Particulars	Case-1	Case-2
1	Size of Tank	12 m × 10 m × 4.50 m	12 m × 10 m × 4.50 m
2	Top Slab Thickness	400 mm	600 mm
3	Bottom Slab Thickness	600 mm	600 mm
4	Anti- floatation Slab thickness	NA	600 mm
5	Wall thickness	450	450 mm
6	Thickness of extended base slab	200 mm all sides	300 mm all sides
7	Thickness of anti- floatation slab	NA	600 mm
8	Earth fill	450 mm	600 mm
9	Factor of Safety achieved	1.22	1.2
10	Modeling complications	No complications	Plate on plate is a challenge
11	No. of loads	7	6
12	Maximum Displacement	41.86 mm	29.37 mm
13	Maximum Shear stress along X direction	0.245 N/mm^2	0.628 N/mm^2
14	Minimum Shear Stress along X direction	0.119 N/mm^2	0.111 N/mm^2
15	Maximum Shear Stress along Y direction	0.417 N/mm^2	0.50 N/mm^2
16	Minimum Shear stress along Y direction	0.60 N/mm^2	0.47 N/mm^2
17	Maximum Moment along X direction	327 kNm/m	80 kNm/m
18	Minimum moment along X direction	125.45 kNm/m	132 kNm/m
19	Maximum Moment along Y direction	250 kNm/m	105 kNm/m
20	Minimum moment along Y direction	195 kNm/m	105 kNm/m
21	Fabrication complications	No major complications	Connection design require between bottom slab & additional slab

CHAPTER-9

PARAMETRIC STUDY OF THE UGR BY STAAD Pro

On the basis of stability analysis for 50 mm permissible settlement, the node displacement, shear stresses and bending moment for two different cases has been obtained for which the desired stability achieved in terms of Factor of Safety as prescribed by BIS. Now, a study has been done for the stability and strength analysis considering the following parameters.

- ***Stability analysis*** by variable value of permissible settlement or soil subgrade modulus
- ***Strength analysis*** by variable geometry (variable L/B ratio & L/H ratio) of the UGR.

On the basis of that study various graphical relations has been obtained between

For Stability Analysis

- Different depth of UGR and FS for both case-1 & case-2 as explained in previous chapter.
- Different L/B ratio and factor of safety for both case-1 & case-2

For strength analysis (settlement analysis)

- Permissible settlement and node displacement for both case-1 & case-2
- Permissible settlement and shear stress for both case-1 & case-2
- Permissible settlement and bending moment for both case-1 & case-2

All the above mentioned graphical representations have been explained in this chapter in the following articles in detail.

9.1 Stability Analysis

The stability analysis has been done in two different ways based on different geometrical inputs mentioned in Table 9.1 & 9.2.

- Stability analysis at different depth from 3.0 m to 6.0 m at an interval of 0.50 m for L/B ratio 1.20
- Stability analysis for different L/B ratio from 1 to 2 at an interval of 0.1 at a depth of 4.50 m.

In both the cases, variable depth and L/B ratio, the analysis has been made for four different conditions for different geometrical parameters of case-1 & case-2 as mentioned in previous chapter is shown in Table 9.1 & 9.2.

Case-1(a): Base Extension only

Case-1(b): Base Extension with weight of soil wedge

Case-2(a): Without anti floatation slab (Approx. similar to case1 (a): Base extension only)

Case-2(b): Anti floatation slab with extended base

Table 9.1: *Geometrical Inputs –case-1*

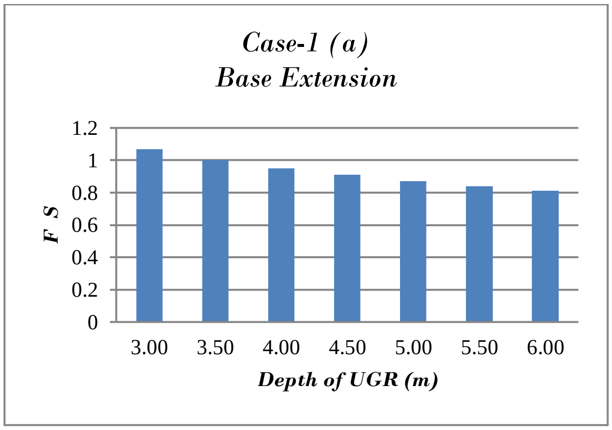
Ground water level	0 m
Thickness of bottom slab	0.60 m
Thickness of top slab	0.40 m
Thickness of walls	0.45 m
Base Extension	0.20 m both side
Thickness of additional slab	0.60 m
Depth of overburden soil	0.45 m

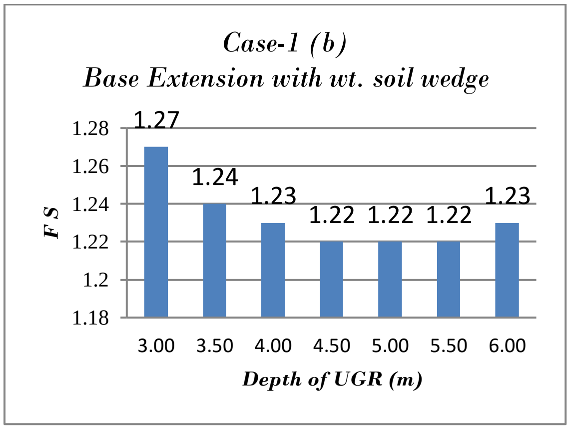
Table 9.2: *Geometrical inputs – case-2*

Ground water level	0 m
Thickness of bottom slab	0.60 m
Thickness of top slab	0.60 m
Thickness of walls	0.50 m
Base Extension	0.30 m both side
Thickness of additional slab	0.60 m
Depth of overburden soil	0.60 m

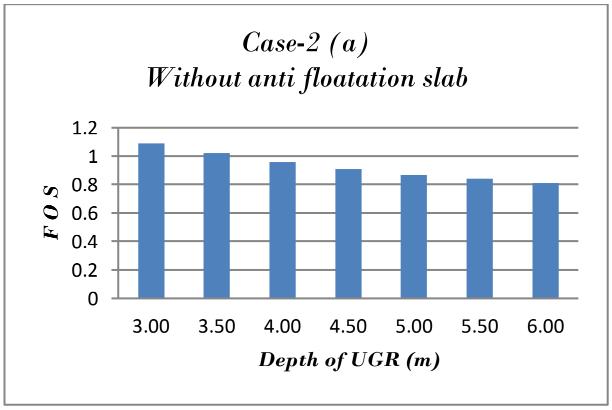
9.1.1 Stability analysis at different depth from 3.0 m to 6.0 m at an interval of 0.50 m for L/B ratio 1.20 **with** *geometrical inputs in Table 9.1*

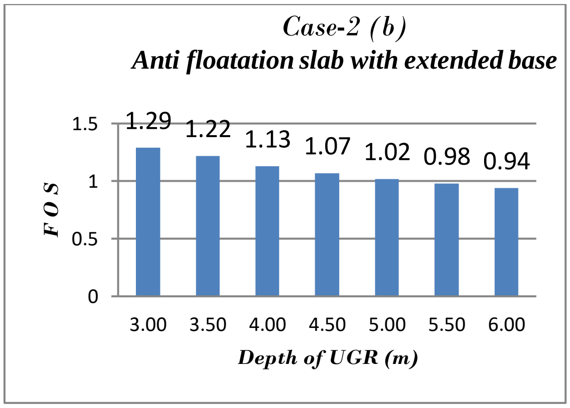
In this case, a graphical representation of stability of UGR in terms of FS developed from depth of 3.0 m to 6.0 m at an interval of 0.50 m below the ground level with the geometrical inputs in Table 9.1 is shown in Figure 9.1. From figure 9.1 (a), it can be seen that in case-1(a), when only base extension is considered to counter the effect of buoyancy, the desired stability cannot be achieved. Whereas, when the effect of soil internal friction is considered, the desired factor of safety is achieved even upto a depth of 6.00 m as shown in Figure 9.1 (b). Similarly, in Figure 9.1 (c), when a measure of buoyancy effect is considered without providing anti floatation slab, the result is same as in case-1 (a). On the other hand, when anti- floatation slab is taken into account, the structure is stable upto a depth of 3.50 m only and beyond that depth the geometry of UGR is not capable to make the structure stable as shown in Figure 9.1 (d). It means, when the geometrical data consider in Table 9.1, the structure may be designed economically by considering the base extension alongwith the weight of soil wedge due to internal friction of soil particles Figure 9.1 (b) only. *The minimum depth which can be adopted in this case is 4.50* m as the FS is at margin with the required FS of 1.20. The stability could not be obtained in case 1(a) & 2(a) as shown in Figure 9.1 (a) & (b) for any depth with L/B ratio of 1.20.





(a) **(b)**



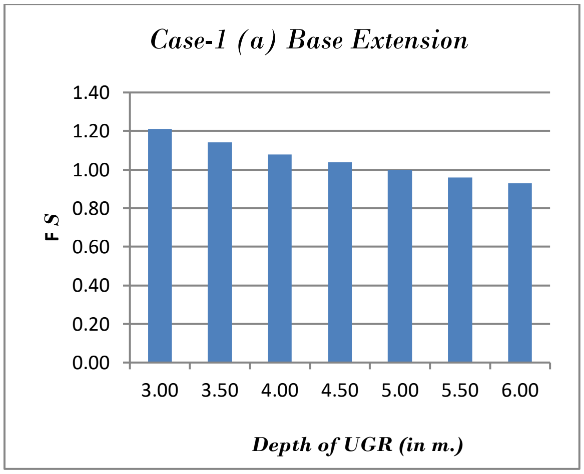


(c) **(d)**

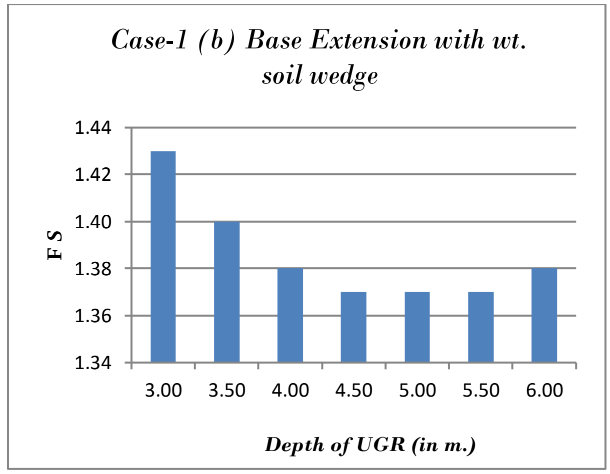
Figure 9.1 (a) to (d): *Graphical representation of stability analysis for variable depth with the geometrical inputs shown in table 9.1*

9.1.2 Stability analysis at different depth from 3.0 m to 6.0 m at an interval of 0.50 m for L/B ratio 1.20 with geometrical inputs in Table 9.2

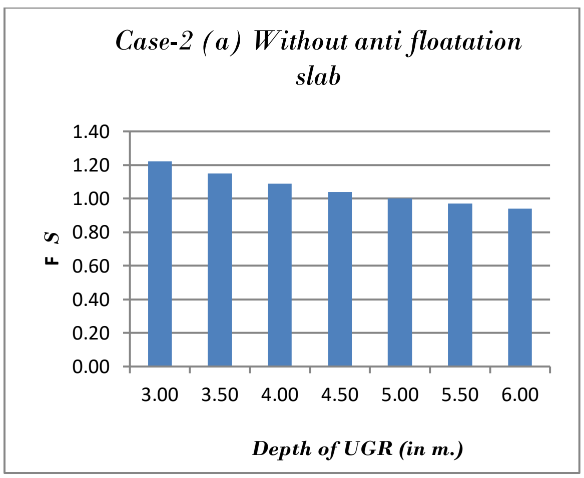
In this case, a graphical representation of stability of UGR in terms of FS has been made from depth of 3.0 m to 6.0 m at an interval of 0.50 m below the ground level with the geometrical in Table 9.2 is shown in Figure 9.2. With these inputs, in case-1(a), when only base extension is considered, the permissible depth is only of 3.0 m which is very small to accumulate required quantity of water needs for the area but when the effect of soil internal friction taken into account in case 1 (b), the structure is stable for all depth ranging from 3.0 m to 6.0 m. Similarly, in case-2 (a), when anti floatation slab is not taken; the suitable depth is only upto 3.0 m which is exactly same as in case 1 (a). On the other hand, when anti-floatation slab with extended base is considered the stability of structure is achieved upto a depth of 4.50 m which is equal to the assumed depth of UGR to collect required quantity of water.



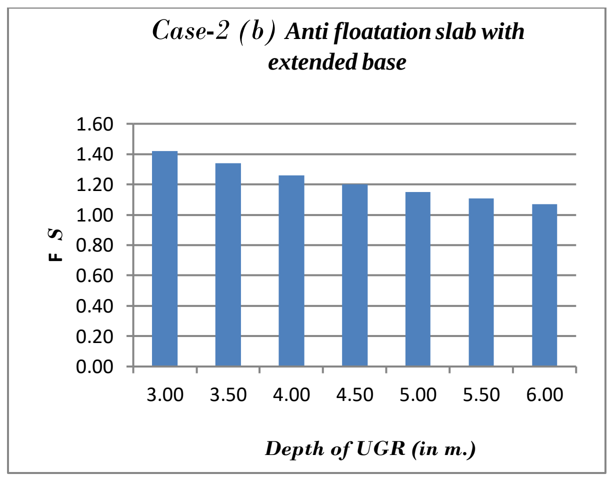
(a)



(b)



(c)



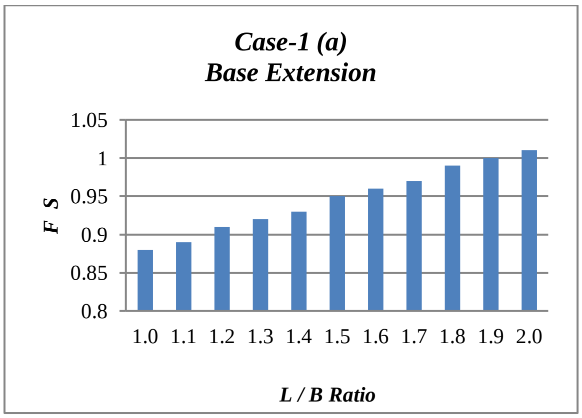
(d)

Figure 9.2 (a) to (d): *Graphical representation of stability analysis for variable depth with the design inputs shown in table 9.2*

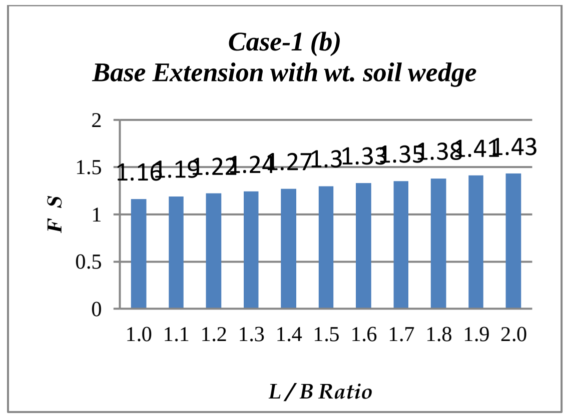
It can be concluded from Figure 9.2 (b), when the geometrical data consider in Table 9.2, the structure may be designed for any depth from 3.0 m to 6.0 m for L/B ratio of 1.20. From Figure 9.2 (d), the design will be economical for the depth of 4.50 m with L/B ratio of 1.20 because the FS is exactly 1.20 at this depth. The structure remain stable for a depth of 3.0 m only in case 1(a) & 2(a) as shown in Figure 9.2 (a) & (b) for L/B ratio of 1.20 which is not sufficient.

9.1.3 Stability analysis for different L/B ratio from 1 to 2 at an interval of 0.1 at a depth of 4.50 m. with geometrical inputs in Table 9.1

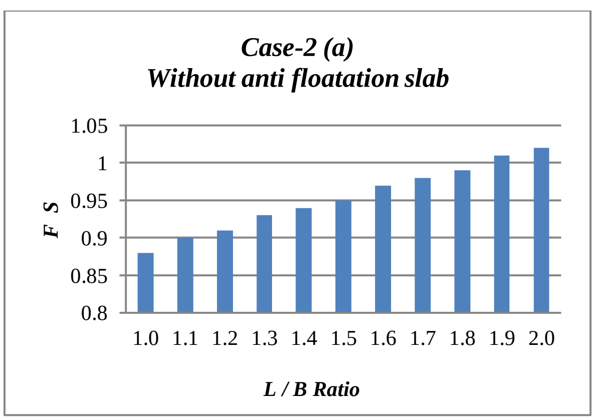
Another study has been done to check the stability of the structure by varying the L/B ratio for a constant height of 4.50m. Likewise in 9.1.1 & 9.1.2 the stability may be checked with two different geometrical inputs mentioned in Table 9.1 & 9.2. The graphical representation of stability in terms of FS for different L/B ratio at a constant depth of 4.50 m with the geometrical inputs of Table 9.1 is shown in Figure 9.3 (a) to (d). In figure 9.3, it can be seen that the required FS is obtained in case1 (b) only for the L/B ratio 1.2 & above, when the effect of soil wedge due to soil internal friction is considered.



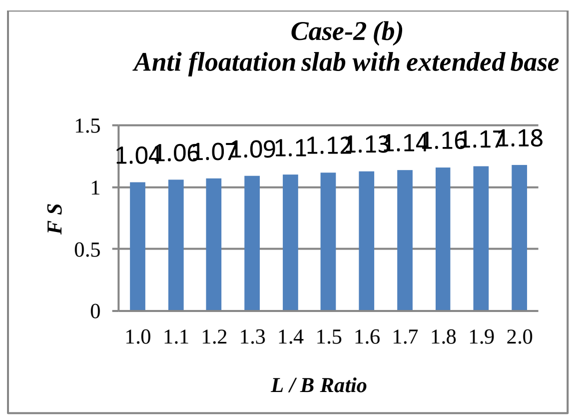
(a)



(b)



(c)



(d)

Figure 9.3 (a) to (d): *Graphical representation of different L/B ratio for constant depth of 4.50 m with geometrical inputs as in table 9.1*

9.1.4 Stability analysis for different L/B ratio from 1 to 2 at an interval of 0.1 at a depth of 4.50 m. with geometrical inputs in Table 9.2

The graphical representation of stability in terms of FS for different L/B ratio at a constant depth of 4.50 m with the geometrical inputs of Table 9.2 is shown in Figure 9.4 (a) to (d). In figure 9.4 (b), it can be seen that the required FS is obtained in case 1(b) for all L/B ratio from 1.0 to 2.0, when the effect of soil wedge due to soil internal friction is considered whereas, the stability in case 2(b) is obtained from L/B ratio 1.20 to 2.0 as shown in Figure 9.4 (d). The stability could not be obtained in case 1(a) & 2(a) as shown in Figure 9.4 (a) & (b) for any L/B ratio.

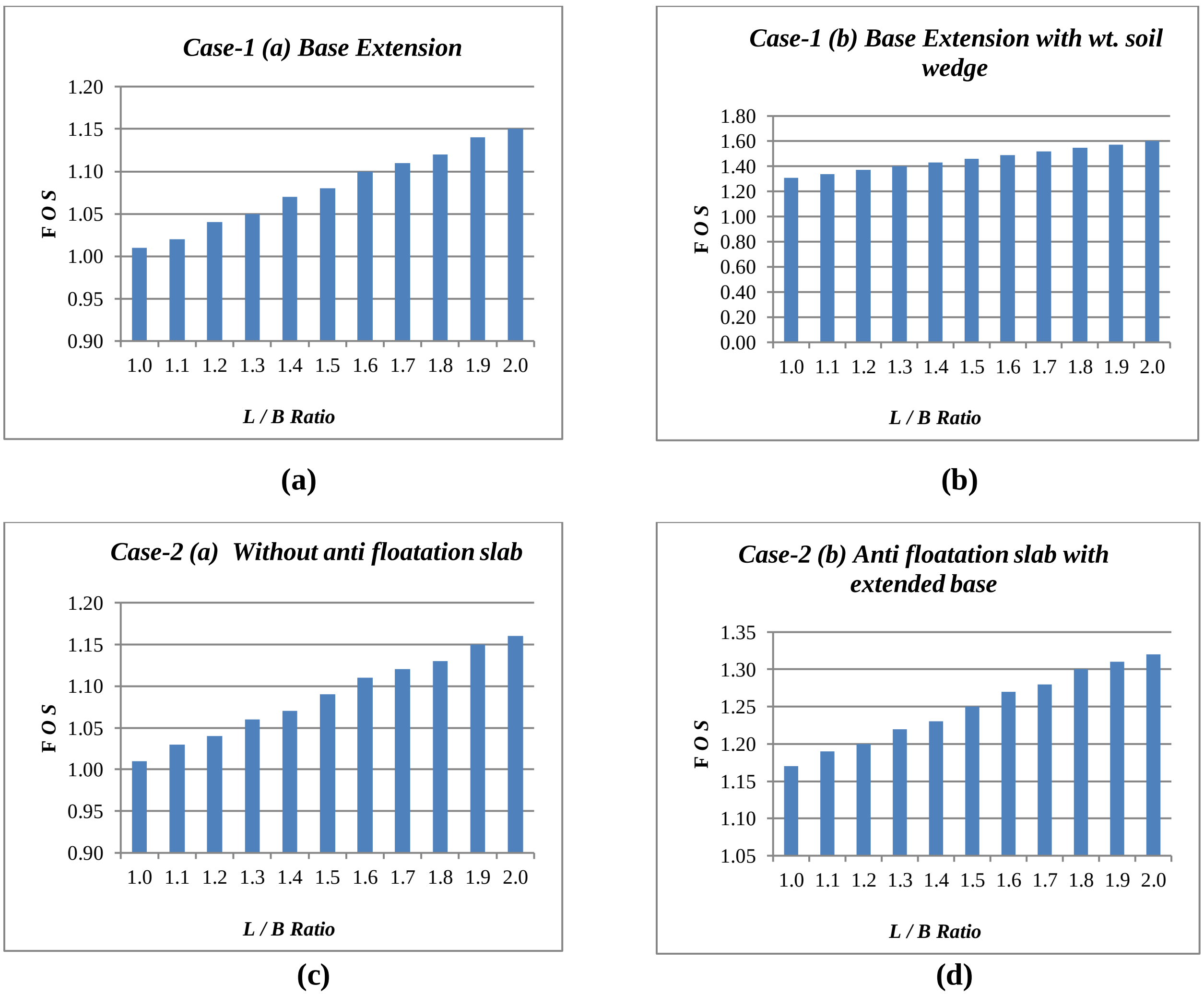


Figure 9.4 (a) to (d): *Graphical representation of different L/B ratio for constant depth of 4.50 m with geometrical inputs as in table 9.2*

From figure 9.1 to 9.4, it is observed that the optimum value of L/B ratio and depth shall be 1.20 & 4.50 m respectively for the geometrical inputs mentioned in Table 9.1 & 9.2. Based on those values, strength analysis is performed to get the behavior of displacement, shear force and bending moment for different values of permissible settlement through a graphical representation as the soil subgrade modulus will change according to net safe earing capacity of soil for the different permissible settlement.

9.2 Strength Analysis (settlement analysis)

For the geometrical inputs in Table 9.1, the stability analysis is satisfactory for the L/B ratio 1.20 & depth 4.50m below the ground level in Figure 9.1 (b) & 9.3 (b). According to Figure 9.1 (b) & 9.3 (b), case 1(b) i.e.; base extension with weight of soil wedge due to angle of internal friction of soil shall be analyzed to perform the strength analysis for different permissible settlement or different soil subgrade modulus.

Similarly, for the geometrical inputs in Table 9.2, the stability analysis is satisfactory for the L/B ratio 1.20 & depth 4.50m below the ground level in Figure 9.2 (b) & (d) and in Figure 9.4 (b) & (d). But the economy can be maintained for L/B ratio 1.20 & depth 4.50 m in Figure 9.2 (d) & Figure 9.4 (d) since the value of FS is exactly equal to the desired standard value of FS. So, for the geometrical inputs in Table 9.2, case 2 (b) i.e.; anti floatation slab with base extension is considered as in Figure 9.2 (d) and in Figure 9.4 (d) to perform the strength analysis for different permissible settlement or different soil subgrade modulus.

Therefore, for L/B ratio of 1.20 and depth 4.50 m, strength analysis shall be done for the following cases

- For geometrical inputs in Table 9.1- case 1(b) i.e.; base extension with weight of soil wedge due to angle of internal friction of soil shall be analyzed.
- Similarly, for geometrical inputs in Table 9.2- case 2 (b) i.e.; anti-floatation slab with base extension shall be analyzed

9.2.1 *For geometrical inputs in Table 9.1 – case 1(b) i.e.; base extension with weight of soil wedge due to angle of internal friction of soil.*

In the strength analysis of the structure for node displacement, shear stress and bending moment, the permissible settlement is considered ranging from 20 mm to 80 mm at an interval of 10 mm. For different permissible settlement values, different net safe bearing capacity is obtained by the field study as shown in Table 9.3. With the help of different net safe bearing capacity of soil and permissible settlement, different value of soil subgrade modulus is obtained.

Table 9.3: *Soil subgrade modulus for different settlement values*

Permissible settlement (mm)	Net Safe Bearing Capacity (kN/m^2)	Soil subgrade modulus (kN/m^2 per meter)
20	65	3250
30	82	2733
40	110	2750
50	130	2600
60	133	2217
70	137	1957
80	145	1813

Figure 9.5 shows the node displacement for the different settlement values. This indicate, that there is no significant change in value of node displacement for permissible settlement upto 50 mm. beyond that, the displacement in the structure will increase considerably.

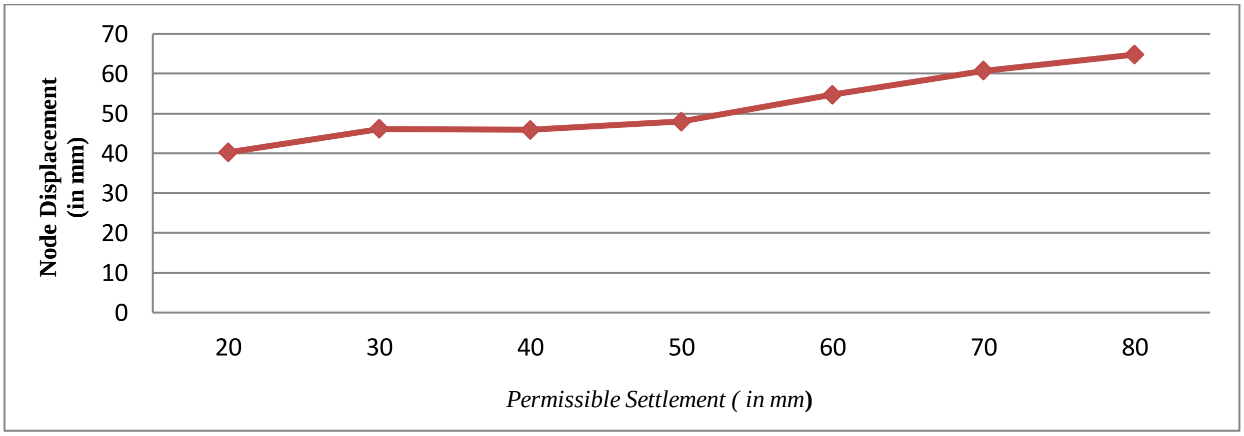


Figure 9.5: *Node displacement- base extension with weight of soil wedge due to angle of internal friction of soil for geometrical inputs in Table 9.1*

From Figure 9.6, the maximum shear stress remains constant after 50 mm permissible settlement. It means that the structure will not experience extraordinary stresses when we increase the permissible settlement value beyond 50 mm.

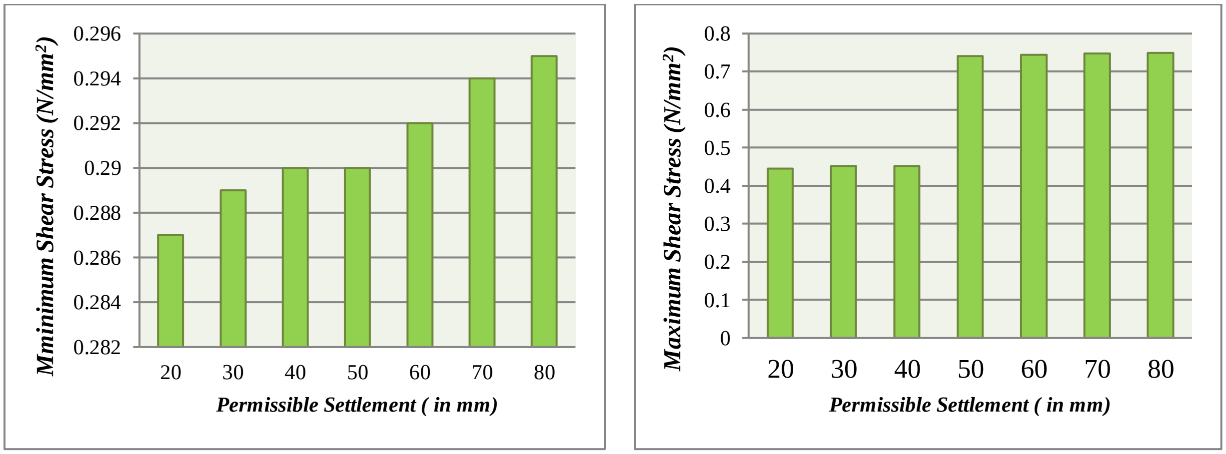


Figure 9.6: *Minimum and maximum shear Stresses- base extension with weight of soil wedge due to angle of internal friction of soil for geometrical inputs in Table 9.1*

The bending moment both in X & Y direction keeps increasing with the settlement values. It will be uneconomical if the permissible settlement value is increased from 50 mm.

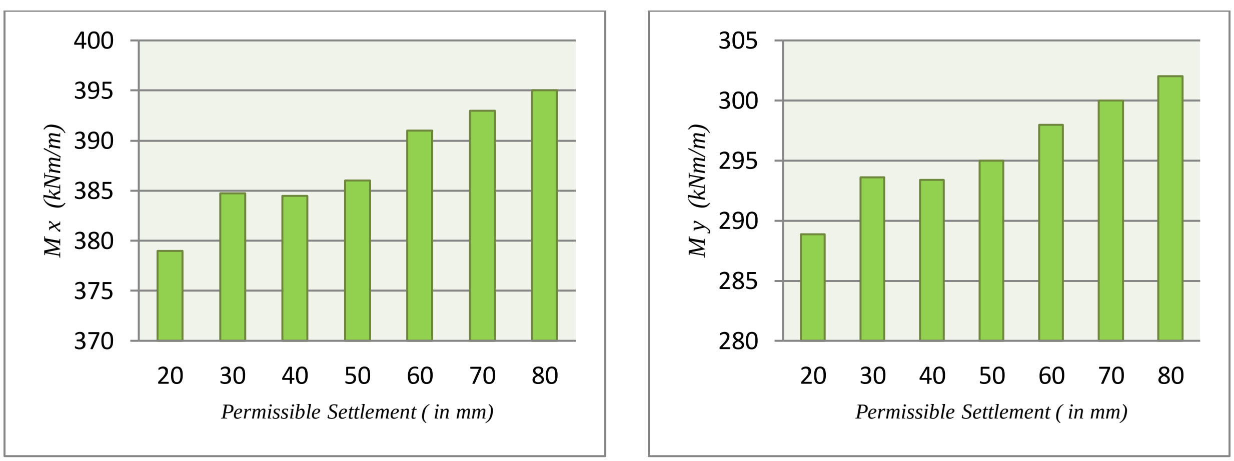


Figure 9.7: *Bending Moment in X & Y directions- base extension with weight of soil wedge due to angle of internal friction of soil for geometrical inputs in Table 9.1*

9.2.2 *For geometrical inputs in Table 9.2- case 2 (b) i.e.; anti-floatation slab with base extension*

The strength analysis of the structure in this structure in terms of node displacement, shear stress and bending moment is shown in Figure 9.8, 9.9 & 9.10 for different values of the permissible settlement from 20 mm to 80 mm at an interval of 10 mm. The soil subgrade modulus for different settlement values is taken from Table 9.3.

In Figure 9.8, the pattern shows that there is no significant change in displacement for permissible settlement upto 50 mm. but beyond it, the displacement in the structure will increase considerably. However, the values of displacement in this case are about 60% as compared in Figure 9.5.

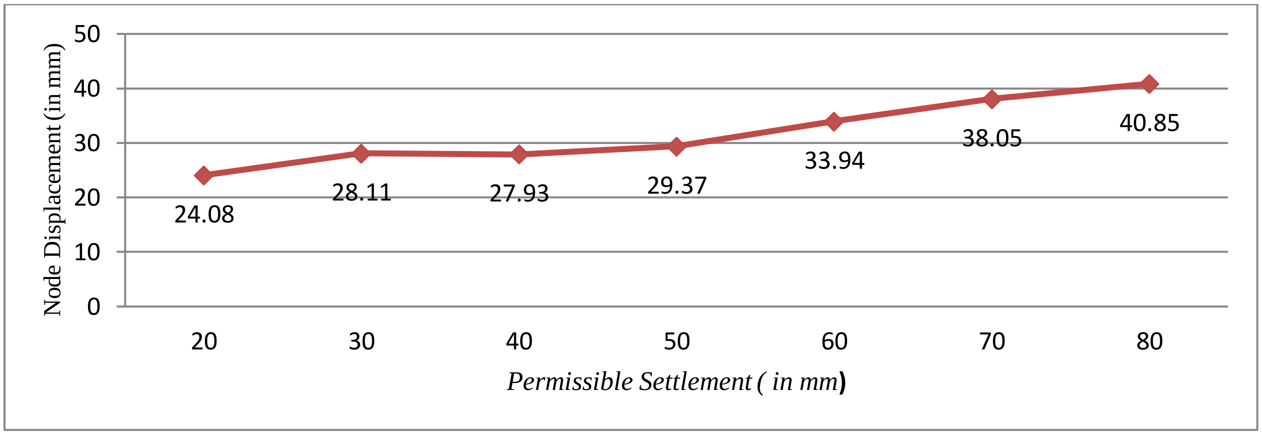


Figure 9.8: *Node displacement- anti floatation slab with base extension for the geometrical inputs in Table 9.2*

From Figure 9.9, it is observed that the shear stress will increase in proportion with the settlement value.

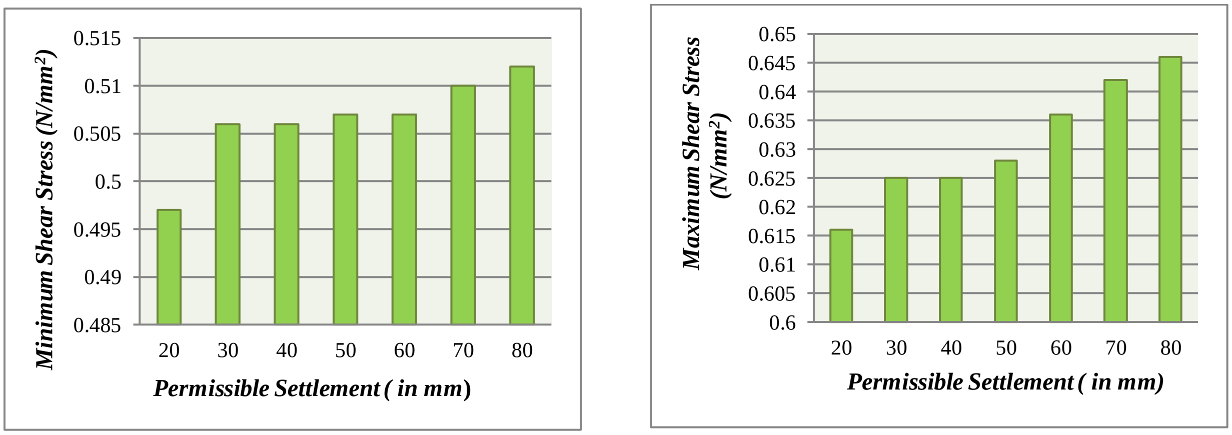
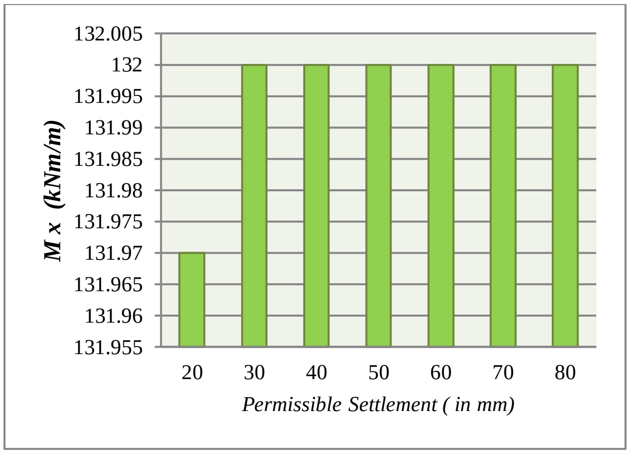


Figure 9.9: *Minimum & Maximum Shear Stresses - anti floatation slab with base extension for the geometrical inputs in Table 9.2*

The bending moment in X direction remains same after 20 mm settlement till 80 mm and similarly the bending moment in Y direction remains constant after 40 mm settlement upto 80 mm but with lower values than the moment in X direction.



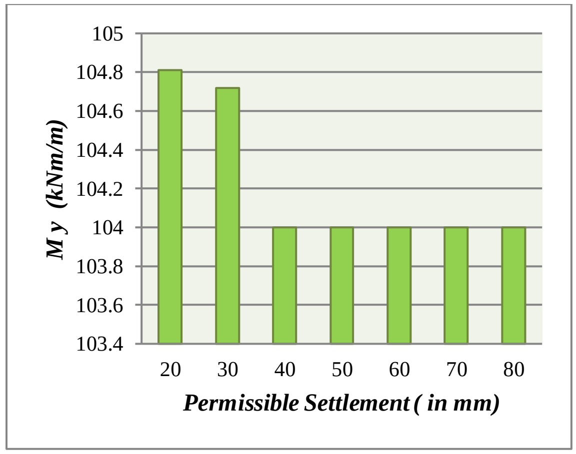


Figure 9.10: *Bending moment in X and Y directions - anti floatation slab with base extension for the geometrical inputs in Table 9.2*

9.3 Base Pressure

Figure 9.11 & 9.12 shows the base pressure for the case mentioned in article 9.2 for L/B ratio 1.20 & depth 4.50 with the geometrical inputs of Table 9.1 & 9.2 considering the base extension with weight of soil wedge due to angle of internal friction of soil and anti- floatation slab with base extension respectively. In both the cases, no major deviation in base pressure is noticed for different settlement values.

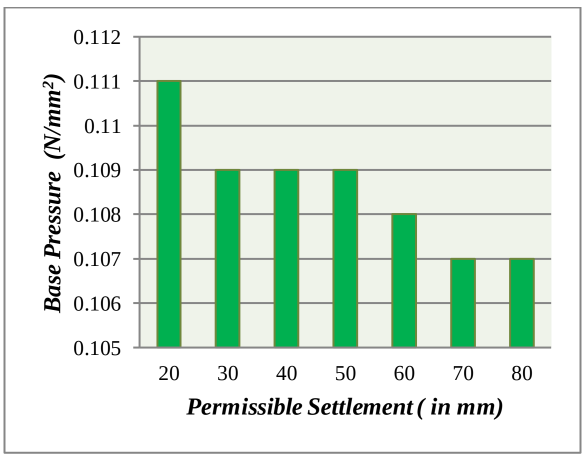


Figure 9.11: *Base Pressure geometrical inputs in Table 9.1 Extended base with weight of soil wedge*

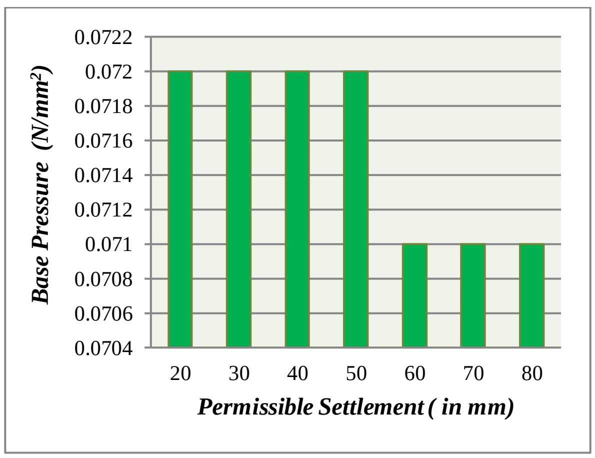


Figure 9.12: *Base Pressure geometrical inputs in Table 9.2 Anti- floatation slab with extended base*

CONCLUSION

A study of the effect of buoyant force exerted by the submerged soil on an underground water reservoir where water table is considered on grade has been studied. The critical condition from the different possibilities of failure considering the effect of seasonal variation on ground water table has been identified. After analyzing the critical condition, various countermeasures have been adopted to make the structure safe against overturning or failure of the structure. Following conclusions are made from this study:

(i) The structure is unstable when the tank is empty and ground water table is maximum. This condition is considered as critical condition.

(ii) By increasing the thickness of the components of the structure, the stability is not achieved against the critical condition.

(iii) By increasing the depth of tank and filling of concrete in the additional depth, the structure is not stable.

(iv) By extending the bottom slab 200mm beyond the walls and incorporate the weight of soil wedge due to angle of internal friction of soil, the desired stability is achieved.

(v) Provision of anti-floatation slab beneath the structure also found suitable to make the structure stable.

(vi) Analyzed the structure in STAAD Pro for the conditions mentioned in (iv) & (v) in accordance with the different geometrical inputs.

(vii) In case of anti-floatation slab, the structure is modeled with the help of dummy columns having zero density to connect the anti-floatation slab with the bottom slab of the structure.

(viii) For both the cases mentioned in (iv) & (v), node displacement, maximum & minimum shear stress and maximum and minimum bending moment obtained and analyzed.

(ix) The value of node displacement and maximum bending moment is less in case of anti - floatation slab than the extended base, whereas the value of shear stress is more in former than later.

(x) It figures out that the design of the structure by providing anti-floatation slab becomes economical due to less bending moment.

(xi) The value of maximum displacement in nodes is less in case of anti-floatation slab signifies that the structure is well within the limits of permissible settlement of 50mm.

(xii) Parametric study has been done with the pre defined two sets of different geometries of the underground reservoir.

(xiii) Both stability and strength (settlement) analysis are done for both the cases as mentioned in (iv) & (v).

(xiv) The stability analysis is done in two ways:

a. By varying the depth of UGR from 3.0 m to 6.0 m below the ground level and the length and breadth remain fixed.

b. By varying the L/B ratio from 1 to 2 and the height of UGR is fixed at 4.50 m

(xv) The optimum value of L/B ratio has been worked out at 1.20 and the depth is at 4.50 m suitable for stability as well as economically.

(xvi) Further, the strength analysis has been done for both the geometrical inputs by fixing the L/B ratio 1.20 and depth of 4.50 m for different permissible settlement ranging from 20 mm to 80 mm at an interval of 10 mm. The soil subgrade modulus for different settlement values has been worked out and it becomes the variable input for the analysis.

(xvii) There is no considerable effect in the values of node displacement in both the cases mentioned in (iv) & (v) upto 50 mm settlement values. Beyond that the values increase considerably.

(xviii) The value of node displacement is about 60% of the values that obtained in case of extended base.

(xix) In case of extended base, the maximum shear stress remains constant after 50 mm permissible settlement and the bending moment both in X & Y direction keeps increasing with the settlement values.

(xx) In case of anti-floatation slab, the shear stress increases with the settlement value. The bending moment in X direction remains same after 20 mm settlement till 80 mm and similarly the bending moment in Y direction remains constant after 40 mm settlement upto 80 mm but with lower values than the moment in X direction.

(xxi) Base pressure remains almost constant in both the cases for different geometrical parameters.

For future study, design shall be performed for both the cases with the pre-defined geometrical parameters and identified the case which is best suitable in terms of economy and fabrication.

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