CHAPTER 1- INTRODUCTION

1.1 GENERAL DESCRIPTION

India has a large population distributed all over the country. In these modern days power has become an essential requirement in our day to day life. Power is an important factor in contribution to the development of any nation. The present Power Generation of our country from all sectors is about 1, 50,000 MW and it is expected to be doubled in another 10 years or so.

The uneven distribution of the primary resources for electrical power generation in India, viz., Coal, Lignite, Hydro Potentials, wind energy resources adds the transmission requirement. The transmission tower structures play a major role in power evacuation from generating to load centres. Failure of towers were observed due to natural calamities such as storm, flood, earthquake, landslides, cyclone, design, construction faults, vandalisms and ageing., Besides above, in the locations where sub soil water salinity is very high like in coastal areas, there are lot of chances for rusting of tower stub encased in the concrete as well as the stub above the ground level. If this is not attended in at proper time the tower may collapse under climatic conditions. The basic structure of transmission tower is shown in fig.1.1.

1.2 OBJECTIVE

The conventional method for design of tower foundation, which is widely used in India, is by increasing the resultant of the loads acting on the steel lattice structure by a factor and applying it on the top of the foundation and then carrying out the design and analysis. But the designing and analysis of transmission tower foundations along with the loads as they are acting on the superstructure are not yet studied. This study can prove to change various parameters of transmission tower foundations with respect to its dimension i.e., size of reinforcement used in the foundation.

i) The present study is divided into two parts. Part one consists of validating the design of transmission tower foundations on a computer based software with the conventional method adopted.

ii) The second part consists of studying the suitability of transmission tower foundations in a particular type of soil i.e., by comparing different type of foundations on the basis of their load bearing capacity. This includes comparison of isolated footing with individual pile and group of pile foundation in wet black cotton soil.

1.3 SCOPE

The scope of study of software based analysis of transmission tower foundation is to compare manual design with the software based design of isolated foundation and pile foundation and also to compare the load carrying capacities of these foundations.

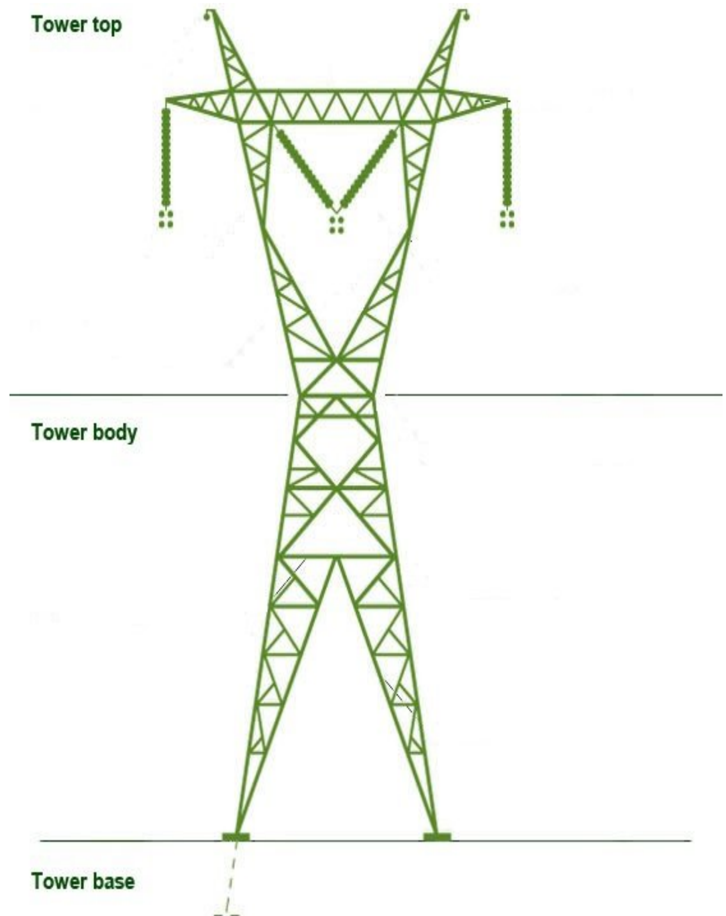
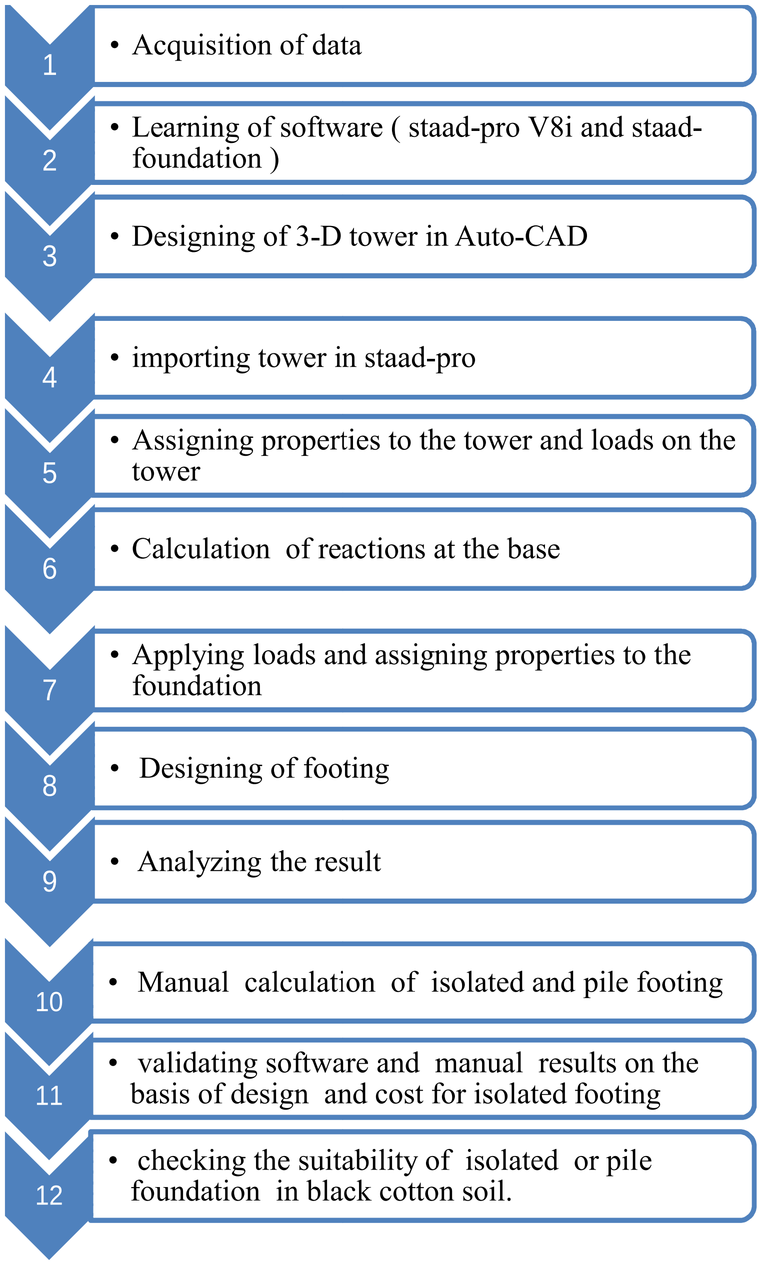


Fig.1.1-Basic structure of transmission tower

1.4 METHODOLOGY

Step by step process for designing of tower foundation in software (STAAD-Pro and STAAD-Foundation). It involves designing of the tower in AutoCADD and STAAD-Pro and designing of foundation in STAAD-Foundation followed by manual calculations for the same.



1.5 ORGANISATION OF THESIS

The thesis is divided into 6 chapters. The content of each chapter has been summarised and given below.

Chapter-1 being the current chapter introduces the various reasons for failure of the foundation of the transmission line tower followed by the objective of the thesis, scope of the study and the methodology to be adopted for the fulfilment of the objective of the thesis.

Chapter-2 discusses the various research works and study carried on designing of transmission tower in STAAD-Pro software, design of tower foundation as isolated footing and pile foundation and includes Indian Standard codes used for designing of isolated and pile footing.

Chapter-3 deals with 3-D designing of transmission tower with the use of software tools such as AUTOCAD and STAAD-Pro. It also deals with the calculation of the reaction at the base of the tower manually.

Chapter-4 deals with designing of isloated footing in black cotton soil (wet) using STAAD- foundation software for the reactions obtained from the chapter 3 and manually designing of isolated footing for the same reactions obtained and for the same soil condition. Calculation of load bearing capacities for isolated footing and pile foundation (single and group) to check the suitability.

Chapter-5 reveals the validation of software designed and manually designed isloated footing. Further analysis is done and compared which finally leads to the suitability of a particular type of foundation (isloated or pile foundation) for a given field condition.

Chapter-6 summary of the results and conlcusions derived from the study is presented in chapter 6. Limitation and future scope are also discussed.

1.6 CLOSING REMARK

This chapter gives an insight about the transmission tower and transmission tower foundation. It reveals about the objective and scope of the thesis . It also includes methodology involved in successful completion of the topics and discusses about the various chapters in the thesis.

CHAPTER 2- LITERATURE REVIEW

2.1 GENERAL

The present investigation aims at comparing the results of software based design and manually designed isolated footing in black cotton soil. Also comparison is done between manually calculated isolated footing and pile foundation for checking their bearing capacities. With regard to this literature review on different types of foundation for transmission tower, Indian standard code for designing of foundation, studies are presented.

2.2 TYPES OF FOUNDATION

B.B Shah (2014) published in his manual that P.C.C type of foundation is the most common type of footing used in India and in some countries. In this type of foundation, the stub angle is taken inside the bottom pad and effectively anchored into it by cleat angles and/or keying roads, and the chimney with or without reinforcement & stub angle inside works as a composite member. The pad may be either pyramidal in shape as shown in fig 2.1 or stepped.

Pile foundation - A typical pile type foundation is shown in Fig 2.2. This type of foundation is usually adopted when soil is very weak and has very poor bearing. The important parameters for design of pile foundation are the type of soil, angle of internal friction, cohesion and unit weight of soil at various depths along the shaft of pile. The downward vertical load on the foundation is carried by the piles through skin friction or by point bearing or both. Load carrying capacity of piles should normally be established by load tests. When it is not possible to carry out load tests,

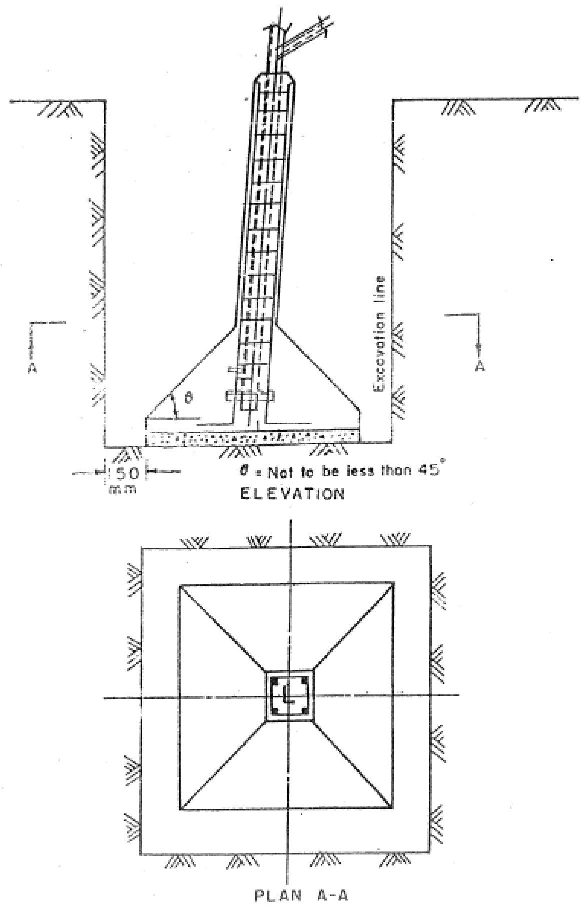


Fig 2.1 Pyramid Chimney Type Foundation (P.C.C.)

the capacity of pile can be determined by static formula as given in IS: 2911 using soil properties obtained from soil investigation of tower location where pile foundation is proposed to be provided.

2.3 DESIGN PROCEDURE FOR FOUNDATION

The design of any foundation consists of following two parts:

1. Stability Analysis

The soil resistances available for transferring the above forces to earth are described in the following paragraphs:

Uplift Resistance

The soil surrounding a tower foundation has to resist a considerable amount of upward force (tension).

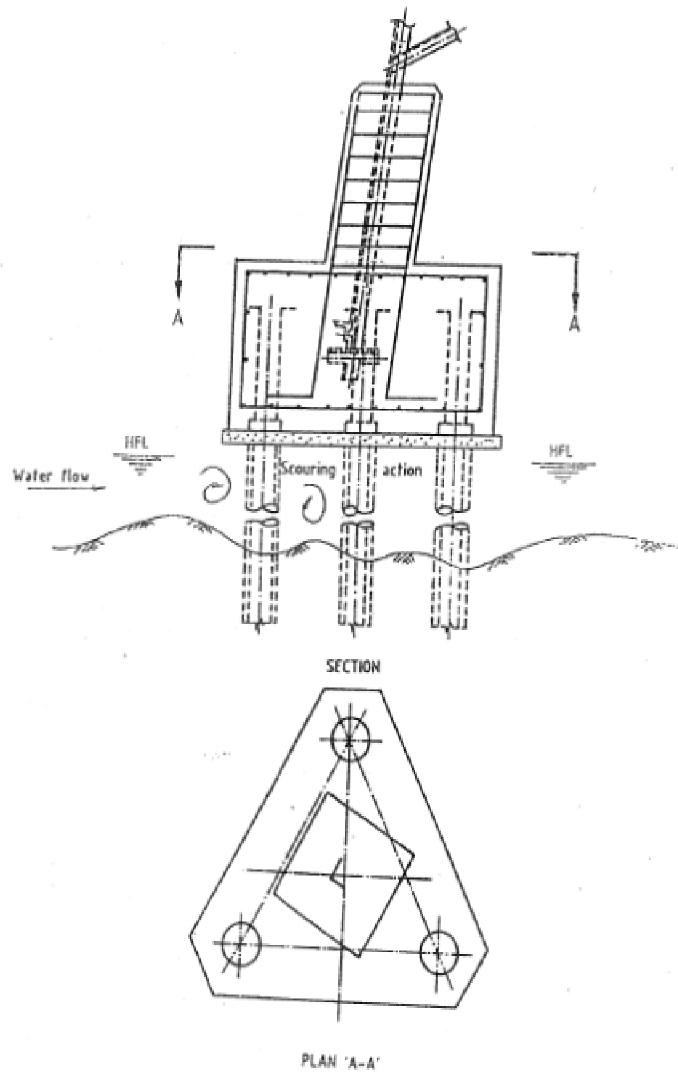


Fig. 2.2 Pile type foundation

It is generally considered that the resistance to uplift is provided by the shear strength of the surrounding soil and the weight of the foundation. Various empirical relationships linking ultimate uplift capacity of foundation to the physical properties of soil like angle of internal friction and cohesion (C) as well as to the dimensions and depth of the footing have been proposed on the basis of experimental results. However, the angle of earth frustum is considered for calculating the uplift resistance of soil. The angle of earth frustum is taken as 2/3 of angle of internal friction or the value given in Annexure I whichever is smaller for the type of soil under consideration.

The uplift resistance is estimated by computing the weight of the earth contained in an inverted frustum of cone whose sides make an angle with the vertical equal to the angle of earth frustum. The formula for calculating volume covered under inverted frustum of a cone.

In the case of shallow isolated foundations, the resistance to uplift is considered to be provided by the weight of the foundation and the weight of the soil volume contained in the inverted frustum of cone on the base of the footing with sides making an angle equal to the angle of earth frustum applicable for a particular type of the soil. The ultimate resistance to uplift is given by

$$UP = W_s + W_f \qquad (2.1)$$

Where

'W_s' is the weight of soil in frustum of cone;

'W_f' is the weight/overload of the foundation (Refer Figure 2.3).

Depending upon the type of foundation i.e., whether dry or wet or partially submerged or fully submerged, the weights 'W_s' and 'W_f' should be calculated taking into account the location of ground water table at critical location.

VOLUME OF UPPER PORTION OF SOIL

$A_1 = B^2 + (4BH_L\tan\alpha) + (\pi H_L^2\tan^2\alpha)$ (2.2)

$A_2 = B^2 + (4B*(H_L\tan\alpha+H_U\tan\beta)) + (\pi(H_L\tan\alpha+H_U\tan\beta)^2)$ (2.3)

$V_U = (H_U/3)*(A_1+A_2+(A_1A_2)^{1/2})$ (2.4)

VOLUME OF LOWER PORTION OF SOIL

$V_L=(B^2H_L)+(2BH_L^2\tan\alpha)+((\pi/3)*(HL^3\tan^2\alpha))$ (2.5)

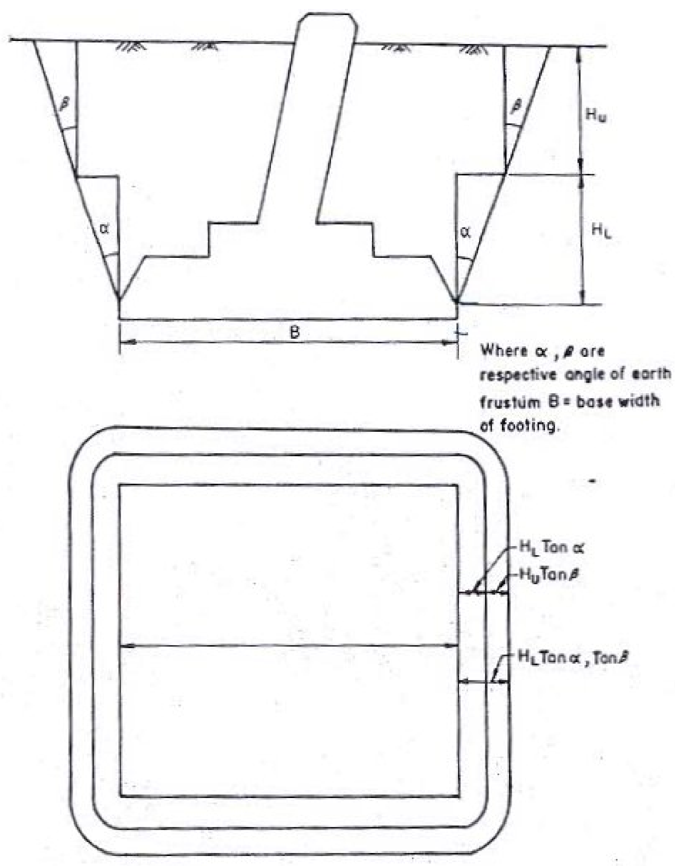


Fig. 2.3 Formula for calculation of the volume of conical frustum of soil

Lateral Soil Resistance:

While designing of towers, the side thrusts (horizontal shears) on the foundation are considered to be resisted by the passive earth pressure mobilized in the adjoining soils due to rotation of the footing. Passive pressure/resistance of soil is calculated based on Rankine's formula for frictional soils and unconfined compressive strength for cohesive soils. Unbalanced horizontal shear is transferred from the foundation to earth through bearing capacity.

Analysis of such foundations and design of the chimney for bending moments combined with down thrust/uplift is very important. Stability of a footing under a lateral load depends on the amount of passive pressure mobilized in the adjoining soil as well as the structural strength of the footing in transmitting the load to the soil (Refer Figure 2.4 and 2.5).

Bearing Capacity:

The downward compressive loads acting on the foundation including moments due to horizontal shears and/or eccentricities, wherever existing, are transferred from the foundation to earth through bearing. The limit bearing capacity of soil is the maximum downward intensity of load which the soil can resist without shear failure or excessive settlement.

The total downward load at the base of footing consists of compression per leg derived from the tower design, buoyant weight of concrete below ground level (i.e., difference in the weight of concrete and soil) and weight of concrete above ground level.

While calculating over weight of concrete for checking bearing capacity of soil, the position of water table should be considered at critical location i.e., which would

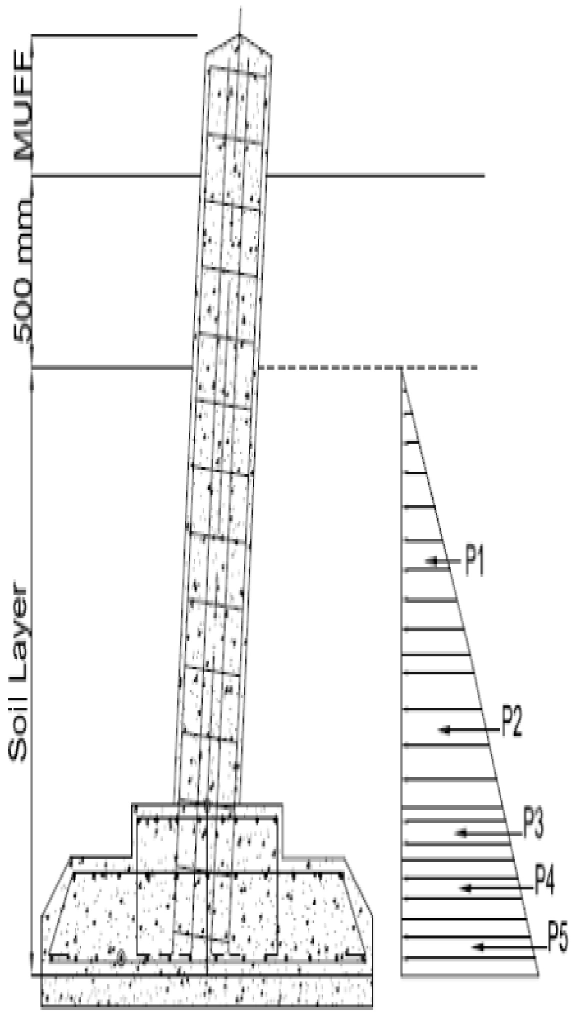


Fig. 2.4

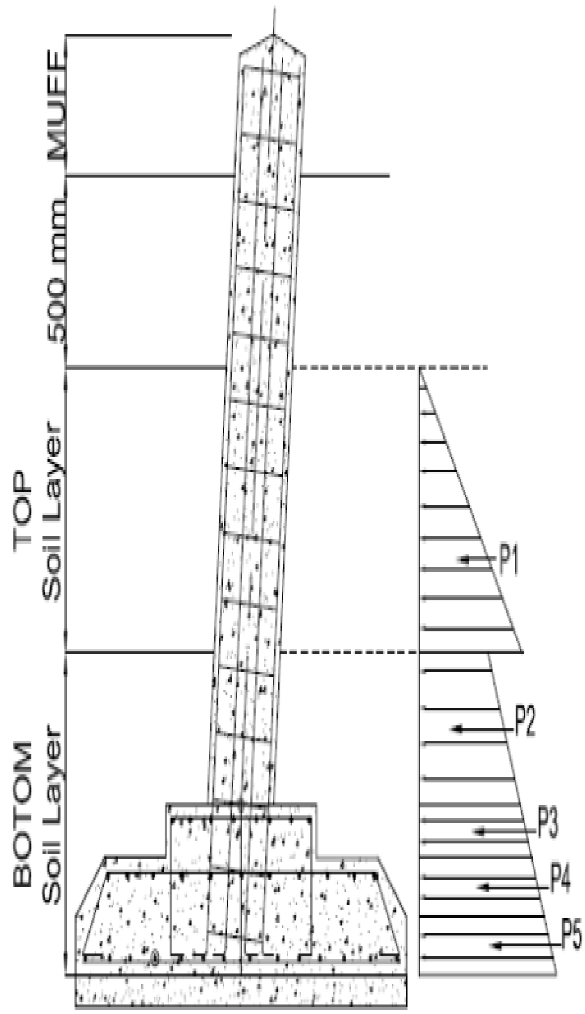


Fig. 2.5

As the foundation is monolithic material structure, The resisting passive pressure would develop in Chimney portion as well as in slab and pyramid portion. Hence it should be considered starting From 500mm below ground (being ignored) up to Bottom of slab/Pyramid.

In case of two layers of soil, Passive Pressure can be calculated for two different layers of soil and there by resisting moment due to it.

give maximum over weight of concrete. In case the centre line of chimney may not coincide with the C.G. of the base slabs/pyramid/block, under such situation; axial load in the chimney can be resolved into vertical and horizontal components at the top of base slab/pyramid/ block. The additional moments due to the above horizontal loads should be considered while checking the bearing capacity of soil.

Further, even in cases where full horizontal shear is balanced by the passive pressure of soil, the horizontal shears would cause moment at the base of footing as the line of action of side thrusts (horizontal shears) and resultant of passive pressure of soil are not in the same line.

Thus, the maximum soil pressure below the base of the foundation (Toe pressure) will depend upon the vertical thrust (compression load) on the footing and the moments at the base level due to the horizontal shears and other eccentric loadings. Under the action of down thrust and moments, the soil pressure below the footing will not be uniform and the maximum toe pressure 'P' on the soil can be determined from the equation :

$$P = (W/B^2)+(M_T/Z_T)+(M_L/Z_L) \qquad (2.6)$$

Where,

W is the total vertical down thrust including over weight of the footing:

B is dimension of the footing base:

M_T & M_L are, moments at the base of footing about transverse and longitudinal axes of footing; and Z_T & Z_L are the section modulus of footing which are equal to (1/6) B^3 for a square footing.

Check For Overturning:

Stability of the foundation against overturning under the combined action of uplift and horizontal shears may be checked by the following criteria.

(i) The foundation over-turns at the toe.

(ii) The weight of the footing acts at the centre of the base; and

(iii) Mainly that part of the earth cone which stands over the heel causes the stabilising moment. However, for design purpose, this may be taken equal to half the weight of the cone of earth acting on the base, it is assumed to act through the tip of the heel.

For stability of foundation against overturning, stabilising moment should be more than overturning moment.

Note: For individual footing of transmission line tower overturning is generally not governing. Hence check against overturning is not required.

Check For Sliding:

In the foundations of transmission towers, the horizontal shear is comparatively small and possibility of sliding is generally negligible. However, resistance to sliding is evaluated assuming that passive earth pressure conditions are developed on a vertical projection above the toe of foundation. The friction between bottom of the footing and soil also resist the sliding of footing and can be considered in the stability of foundation against sliding. The coefficient of friction between concrete and soil can be considered between 0.2 to 0.3. However, the frictional force is directly proportional to vertical downward load and as such may not exist under uplift condition. For cohesive soil the following formula can be applied for calculating the passive pressure to resist sliding:

$P = \mu C + K_p \gamma h$ (For Compression Leg) (2.7)

$P = K_P \gamma H$ (For Tension Leg) (2.8)

where,

C= Compression

θ =Angle of earth frustum

γ = Unit Wt. of soil

μ= coefficient of friction

H= Height of foundation

$K_p = (1-\sin ø)/(1+\sin ø)$ (2.9)

Raghavendra (2012) in his paper proposed that the Transmission-line tower is highly indeterminate structure. In his study, a typical 132-KV double circuit transmission-line tower is considered, for optimizing the structure with respect to configuration and different materials as variable parameters. The tower is modelled and analyzed using STAAD-PRO and ANSYS software's. The basic model of the tower considered is analyzed in STAAD-PRO and the results with respect to the member axial forces are validated in ANSYS. A number of experimental configurations of the tower are obtained by increasing the base width of the tower and also by decreasing the bracing patterns below the waist of the tower.

Punse (2014) In his thesis Analysis and Design of narrow based Transmission Tower (using Multi Voltage Multi Circuit) is carried out keeping in view to supply optimum utilization of electric supply with available ROW and increasing population in the locality, in India. Transmission Line Towers constitute about 28 to 42 percent of the total cost of the Transmission Lines. The increasing demand for electrical energy can be met more economical by developing different light weight configurations of transmission line towers.

In his project, an attempt has been made to make the transmission line more cost effective keeping in view to provide optimum electric supply for the required area by considering unique transmission line tower structure. The objective of this research is met by choosing a 220KV and 110KV Multi Voltage Multi Circuit with narrow based Self Supporting Lattice Towers with a view to optimize the existing geometry. Using STAAD PRO v8i analysis and design of tower has been carried out as a three dimensional structure. Then, the tower members are designed.

IS 2911-1-1 (2010) PART 1 : Design and construction of pile foundations

Section 1 Driven cast In-situ concrete piles

This standard covers the design and construction of driven cast in-situ concrete piles which transmit the load to the soil by resistance developed either at the pile tip by end bearing or along the surface of the shaft by friction or by both.

Piles in cohesive soil :

The ultimate load capacity (Qu) of piles, in KN, in cohesive soils is given by the following formula:

$$Qu = Ap\ Nc\ Cp + \sum \alpha i\ Ci\ Asi \tag{2.10}$$

The first term gives the end-bearing resistance and the second term gives the skin friction resistance.

where

A_p = cross-sectional area of pile tip, in m2

N_c = bearing capacity factor, may be taken as 9

C_p = average cohesion at pile tip, in kN/m^2

αi = adhesion factor for the ith layer depending on the consistency of soil

C_i = average cohesion for the ith layer, in kN/m^2

Asi = surface area of pile shaft in the ith layer, in m2

IS 456-2000 Plain and reinforced concrete (fourth revision)

This code deals with the general structural use of plain and reinforced concrete. It is generally used for the calculation of reinforcement for different types of foundations. It covers details regarding concrete cover, number of bars, spacing, lateral ties etc.

2.4 CLOSING REMARK

In this chapter we have studied the software used for the designing of transmission tower. And also the different types of transmission tower foundations are presented. In the further chapters, reactions at the support of transmission tower are found out and design of different types of foundation is carried out.

CHAPTER 3-DESIGN AND ANALYSIS OF TRANSMISSION TOWER

3.1 GENERAL DESCRIPTION

A transmission tower is a tall structure, usually a steel lattice tower, used to support an overhead power line. They are used in high-voltage AC and DC systems, and come in wide variety of shapes and sizes. Typical height ranges from 15 to 55 metres.

The tower used in this project is 400KV D/C transmission line tower type DV .

3.2 DESIGNING OF TRANSMISSION TOWER IN AUTO CAD 2012

AutoCAD is a commercial software application for 2D and 3D computer-aided design (CAD) and drafting. AutoCAD is used across a wide range of industries, by architect, project manager, engineers, graphic designers and other professionals. AutoCAD 2012 is used in designing of 3-D structure of the transmission tower. 2-D structure of transmission tower is shown in fig.3.1 and 3-D structure in fig.3.2.

3.3 ANALYSIS OF TRANSMISSION TOWER IN STADD-PRO V8i

STAAD.Pro is a structural analysis and design computer program originally developed by Research Engineers International in Yorba, Linda, CA. STAAD-Pro is the premier FEM analysis and design tool for any type of project including tower, culvert, plant, bridge, stadium and marine structure. STAAD.Pro will eliminate the countless man hour required to properly load your structure by automating the force required by wind, earthquake, snow, or vehicles.

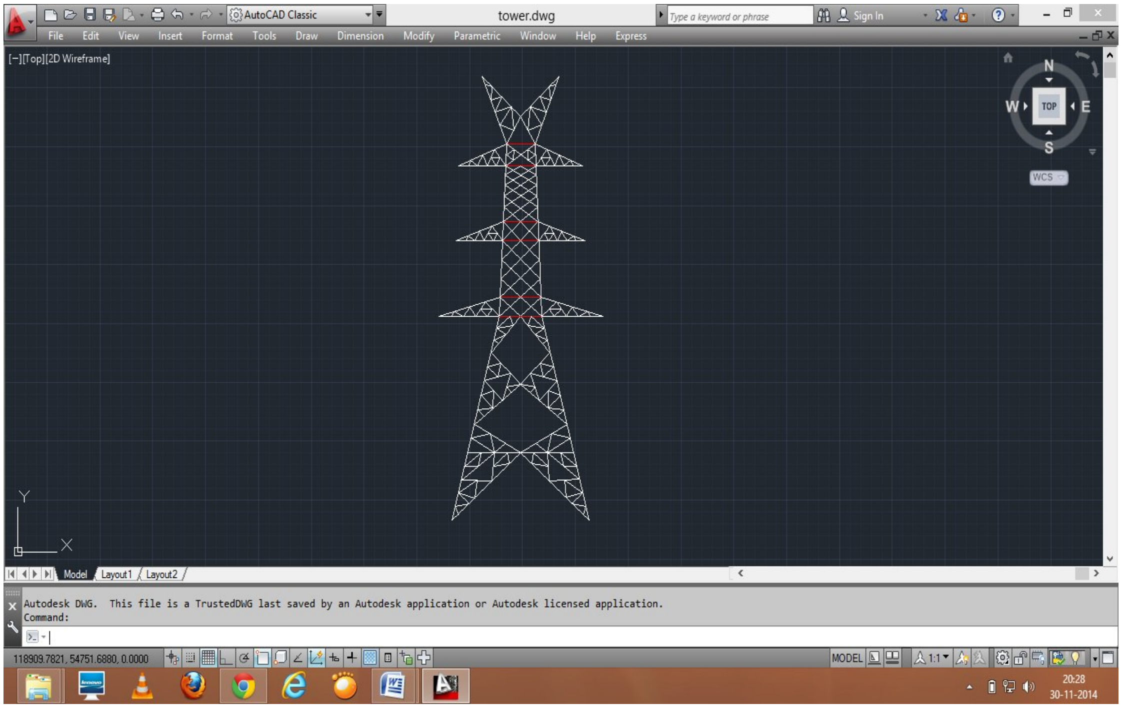


Fig 3.1 A 2-D structure of tower designed in AutoCAD

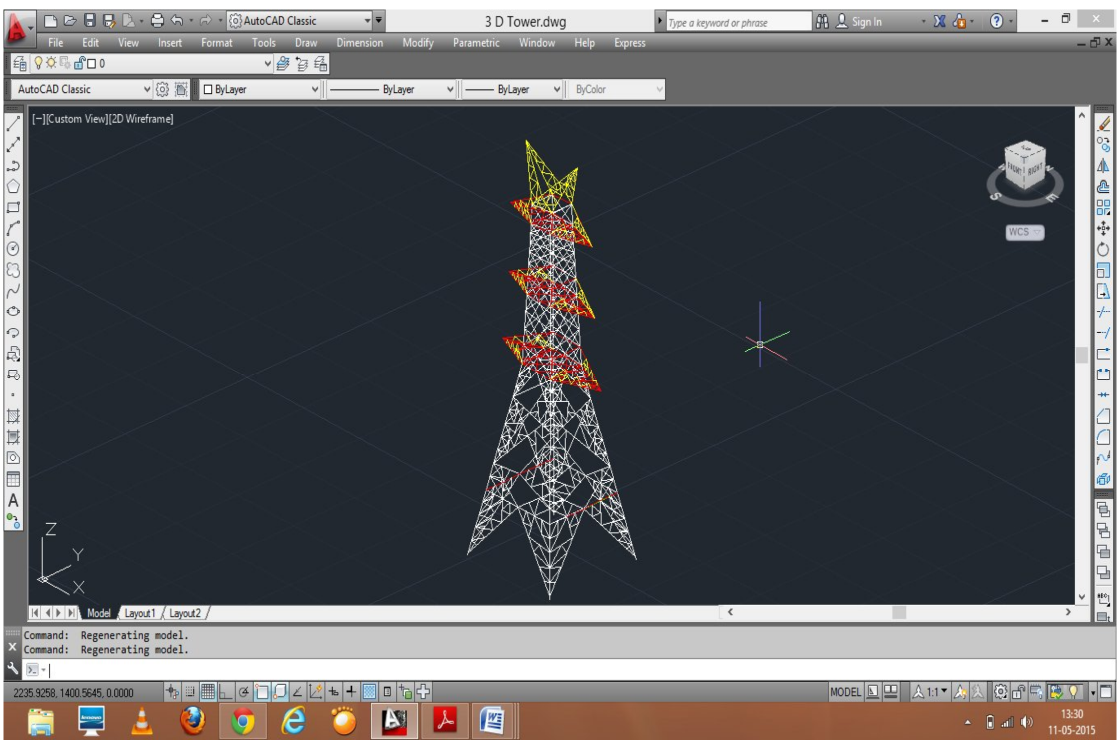


Fig.3.2 3-D structure of tower designed in AutoCAD

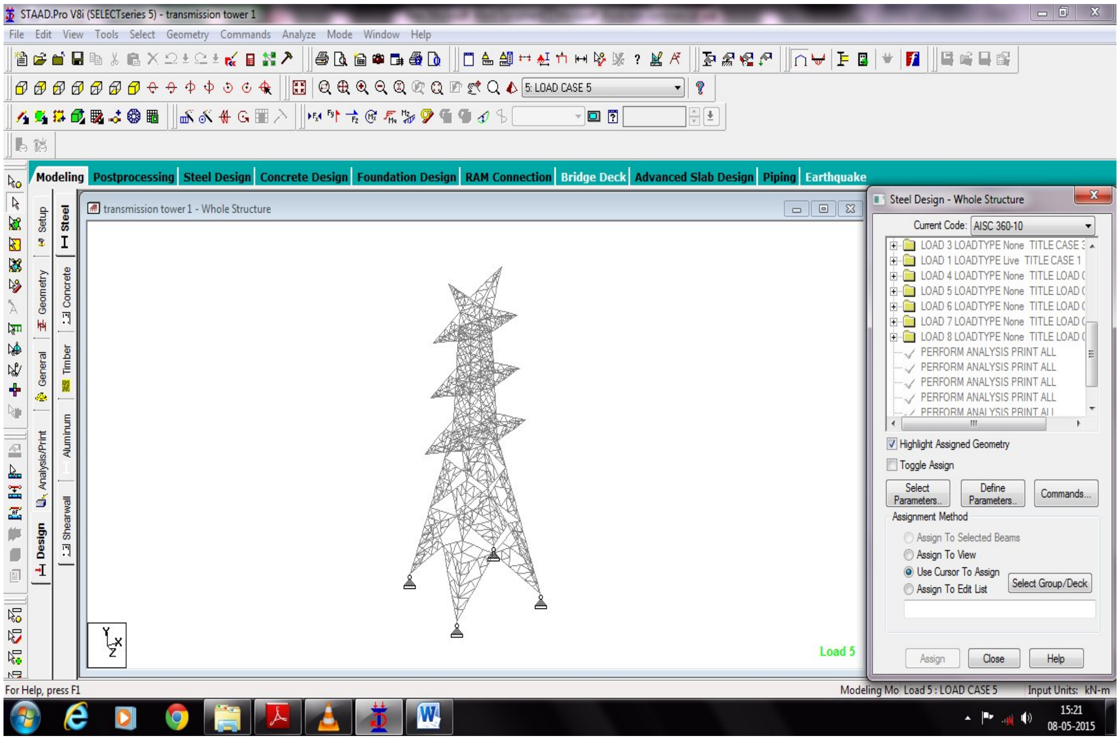


Fig.3.3 3-D transmission tower in STAAD.Pro V8i

The structure is then imported in STAAD.Pro and various parameters like type of material, type of section, dimension of the section and loads are imparted to the structure. Dimensions of the section are given in APPENDIX C and loading tree is given in APPENDIX A.

Run analysis for finding the reactions at the base of the tower. Fig.3.4 is a screenshot of the result of the reactions obtained at the base of the tower after running analysis of the tower.

For designing of foundation, maximum reaction in 3 directions i.e. x, y and z direction is taken.

Table.3.1 Maximum reaction at supports

FORCES	MAXIMUM REACTION (KN)
F_x	93.65
F_y	-159.27
F_z	39.34

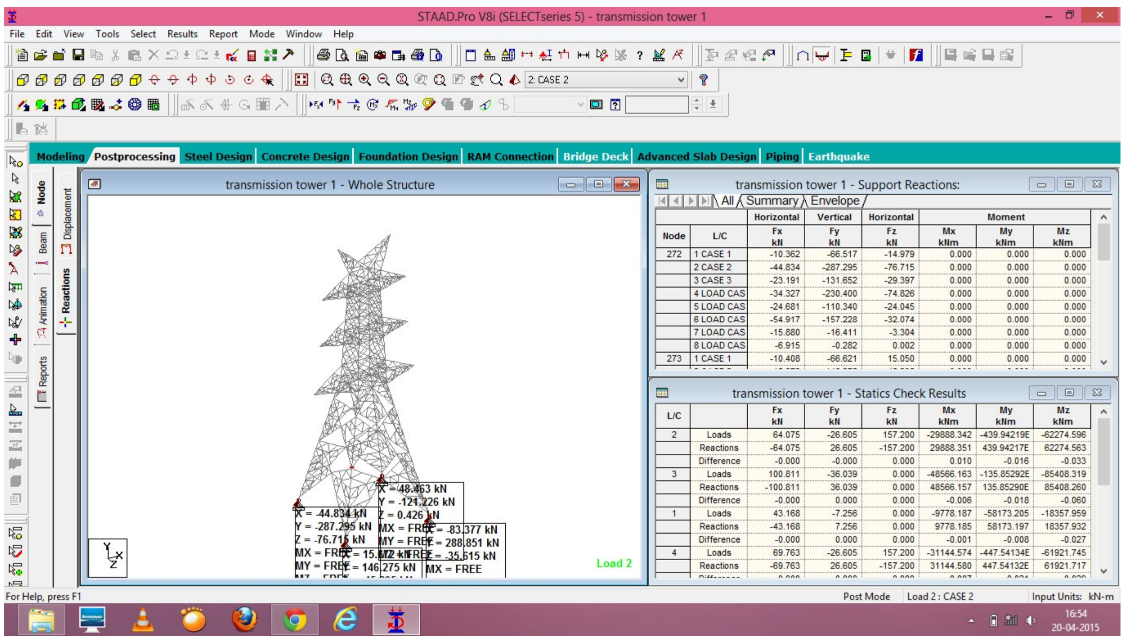


Fig.3.4 Tower reaction forces

3.4 MANUAL CALCULATION OF REACTIONS AT THE BASE OF TOWER

Reactions at the base of the tower are calculated manually using the formulae given below.

Longitudinal and transverse load on each support is given by:-

(a) $$\frac{(\sum T \text{ x } h + \sum L \text{ x } h)}{(2 \text{ x } W \text{ x } \cos^2\alpha)} \quad (3.1)$$

Vertical load on each support is given by:-

(b) $\sum V/4$ (3.2)

Load due to broken wire condition on each support is given by:-

(c) $$\frac{\sum (V_3 - V_4) \text{ x } b}{4W \cos\alpha} \quad (3.3)$$

Load due to dead load of tower on each support is given by:-

(d) DL/4 (3.4)

Maximum transverse load on each support is given by:-

(e) $$\frac{(\sum T \text{ x } h)}{(2 \text{ x } W \text{ x } \cos^2\alpha)} \quad (3.5)$$

Maximum longitudinal load on each support is given by:-

(f) $$\frac{(\sum L \text{ x } h)}{(2 \text{ x } W \text{ x } \cos^2\alpha)} \quad (3.6)$$

Where T= Transverse load

h= height of tower from ground level

W= Width of tower

α= Angle of repose

V= Vertical load

V_3=Vertical load at broken wire point

V_4=Vertical load at broken wire point

DL= Dead load

Maximum download reaction (compression) at the support= (a) + (b) + (c) + (d)

Maximum upward reaction (uplift) at the support= (a) - (b) + (c) - (d)

Maximum reaction in X direction at the support = (e)

Maximum reaction in Z direction at the support = (f)

Comparing table no.1 and 2 we found that the software reactions at the support lie within 3% of the manual reactions at the support.

Table.3.2 Calculations of reactions of tower

FORCES	MAXIMUM REACTION (KN)
F_x	91.37
F_y	-156.64
F_z	38.2

3.4 CLOSING REMARK

In the presented work the reactions at the bottom of the super structure have been calculated using STAAD-Pro. These reactions can be referred from table 3.1.The reactions obtained have also been validated using formulae and the results are within 3-5% (with reference to Shah (2014)). These reactions are to be used for the purpose of designing of the foundations (software and manual), which will be covered in the next chapter.

CHAPTER 4-DESIGN OF FOUNDATION

4.1 GENERAL

Foundations are structural elements that connect a structure to the ground that supports it and are typically composed of concrete, steel, and wood. Foundations can generally be classified into two broad categories: shallow foundations and deep foundations. As discussed in section 1.1 Foundation of any structure plays an important role in safety and satisfactory performance of any structure as the foundation transmits the loads of superstructure on the surrounding soil. It must satisfy two fundamental requirements. Firstly, it must provide an adequate factor of safety against failure of the supporting strata. Secondly, any resulting settlement, and in particular differential settlement, should not be detrimental or interfere with the function of the structure.

In a transmission tower, the foundation is subjected to various kinds of failures which are Sliding, Overturning, Uprooting and tilting. Generally, these failures occur because the applied pressure on soil exceeds its ultimate bearing capacity. Normally, the tower foundation constitutes about 20 to 40 percent of the total cost of tower. Thus, the structural engineer is faced with a difficult task of producing economical and reliable design. Therefore, the study of different types of foundation is an important task to find out its suitability for a particular type of soil. In this section the work consists of designing a foundation on computer based software and to carry out manual design of foundation. The foundation taken into consideration is isolated footing. Further an insight on load carrying capacities of individual pile and pile group has been carried out. The pile group consists of pile group having two piles and pile group having four piles.

4.2 DESIGN OF FOUNDATION USING SOFTWARE

The software in which analysis and design is carried out is Staad foundation. STAAD Foundation Advanced is a Bentley product used for foundation design program that offers the ability to model complex or simple footings, including those specific to Plant facilities such as octagonal footings supporting vertical vessels, strap beam foundations supporting horizontal vessels, ring foundations supporting tank structures, and drilled or driven pier foundations. Common foundations such as isolated footings, combined footings, strip footings, pile caps, and mat foundations can also be designed.

The type of foundation taken into consideration is isolated sloped footing. The soil on which foundation is designed is black cotton soil having depth of water table 750mm. The designing carried out in Staad foundation is described with the screenshots taken during the designing of foundation.

4.2.1 Concrete and reinforcement

Concrete is a material which is used to provide strength to a structure. Concrete in foundation plays an important role for providing resistance against uplift, proper binding between the reinforcements used, etc.

Reinforcement in any structure is to increase the tensile strength of the structure. Reinforcement detailing of footing is as much important as site investigation for the type of footing and structural design of footing. A good detailing reflects the design requirement of the footing for structural stability. A good detailing of reinforcement covers topics like cover to reinforcement based on environmental considerations for durability, minimum reinforcement and bar diameters, proper dimensioning of footing etc.

In Staad foundation concrete parameters to be input includes strength and unit weight of concrete. Reinforcement details include yield strength of steel, minimum and maximum bar spacing, minimum and maximum bar size for footing and minimum and maximum bar size for pedestal.

The parameters presented below are taken with reference to IS 456 and Shah (2014) as mentioned in chapter 2.

The parameters which are input into the software are given below:

Unit weight of concrete= 25 kN/m^3

Minimum bar spacing= 80mm

Maximum bar spacing=450mm

Strength of concrete=20N/mm^2

Yield strength of steel= 415 N/mm^2

Footing minimum bar size= 12mm

Footing maximum bar size= 32mm

Pedestal bar minimum size=12mm

Pedestal bar maximum size= 32mm.

Figure 4.1 shows the various concrete and reinforcement parameters input into the software. In Staad foundation column positions are assigned for design of foundation, based on the geometry of the superstructure i.e., the steel lattice structure. The geometry of the superstructure is mentioned in Annexure C, as shown in the previous chapter. This is followed by assigning the concrete and reinforcement parameters as depicted in fig 4.1.

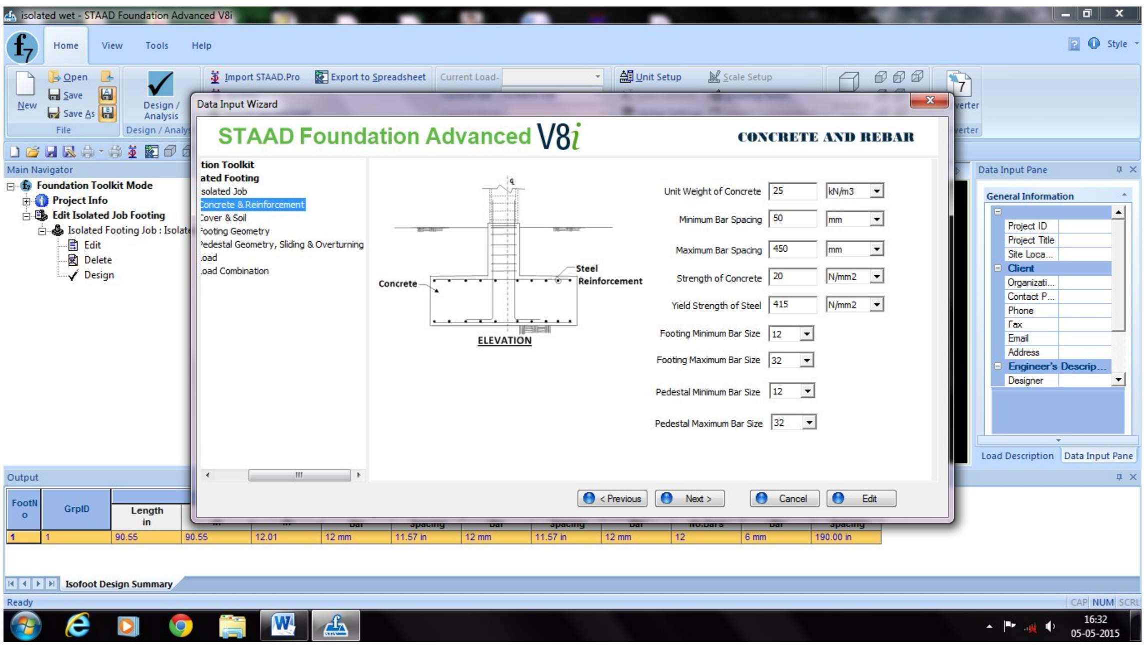


Fig 4.1 Concrete and reinforcement

4.2.2 Soil properties

Information on soil and ground water condition is one of the necessary pre-requisites in designing of foundation because with change of soil properties the design of foundation will also change. It may sometimes happen that if a foundation is suitable for a particular type of soil then it may not be suitable for other type of soil. This may be because of change in soil properties due to change in type of soil. The soil properties include its bearing capacity, its unit weight, depth of water table, etc. The parameters presented below are taken with reference taken from Shah (2014).

The properties of soil which are input into the software are given below:

Unit weight of soil= 9.4kN/m^3

Soil bearing capacity=125kN/mm^2

Depth of water table=750mm

Cohesion=37.73 kN/m^2

Bottom clear cover=50mm

The following figure shows the various soil properties which are input into the software

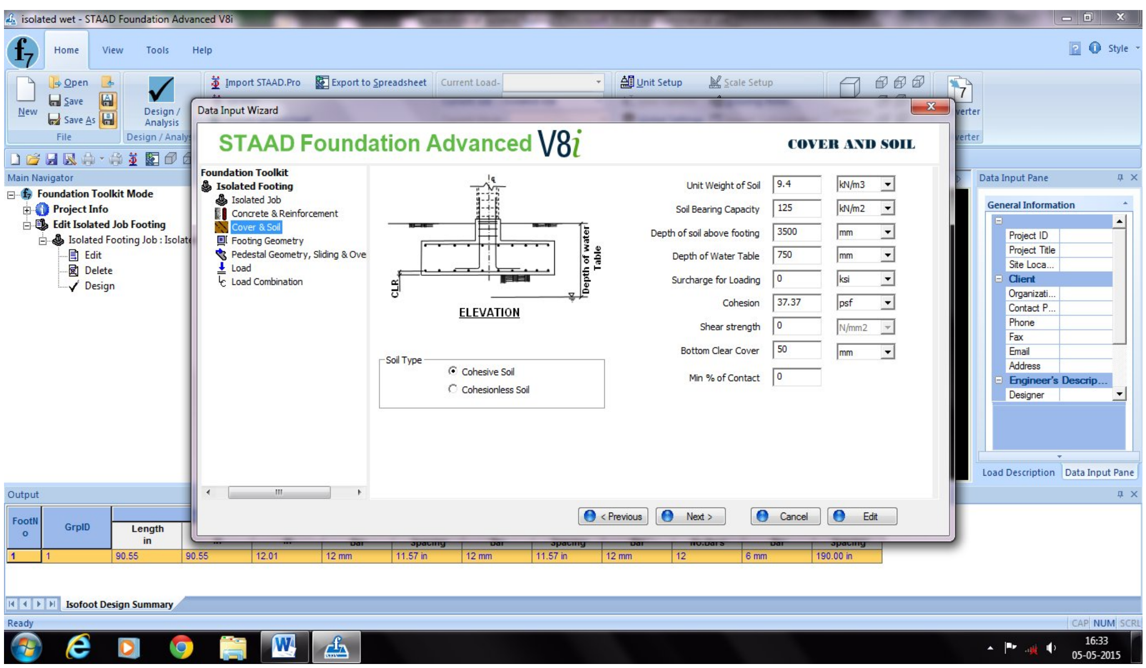


Fig 4.2 Soil properties

4.2.3 Footing geometry

The dimensions of a footing are determined according to bedding conditions so that the average pressure on the bedding doesn't exceed a value that depends on the nature and properties of the earth, the depth of the foundation and the design features of the structure. In assigning the dimensions for the footing, allowance is made for the maximum vertical deformations under which the necessary rigidity of the structure above the foundation will be retained and under which the building will still conform to engineering or architectural requirements.

Footing geometry parameters which are input into the software are given below:

Maximum thickness= 12000mm

Maximum width= 12000mm

Maximum length= 12000mm

Minimum thickness= 305mm

Figure 4.3 shows the footing geometry paramters have to be input in the software for the design of foundation.

4.2.4 Pedestal geometry and factor of safety

As mentioned for footing geometry in section 4.3, the pedestal geometry is also determined according to bedding conditions so that the average pressure on the bedding doesn't exceed a value that depends on the nature and properties of the earth, the depth of the foundation and the design features of the structure. In any type of foundation pedestal is provided for proper distribution of loads on the corresponding soil. For this purpose, its geometry has to be input into the software.

While designing the foundation various factors for sliding and overtunrning moment have to be considered for making the foundation sustain the failures occuring due to sliding and overturning. The overturning and sliding factors are taken with reference to Shah (2014).

Figure 4.4 shows the various pedestal geometry parameters and factor of safety against sliding and overturning which are input into the software.

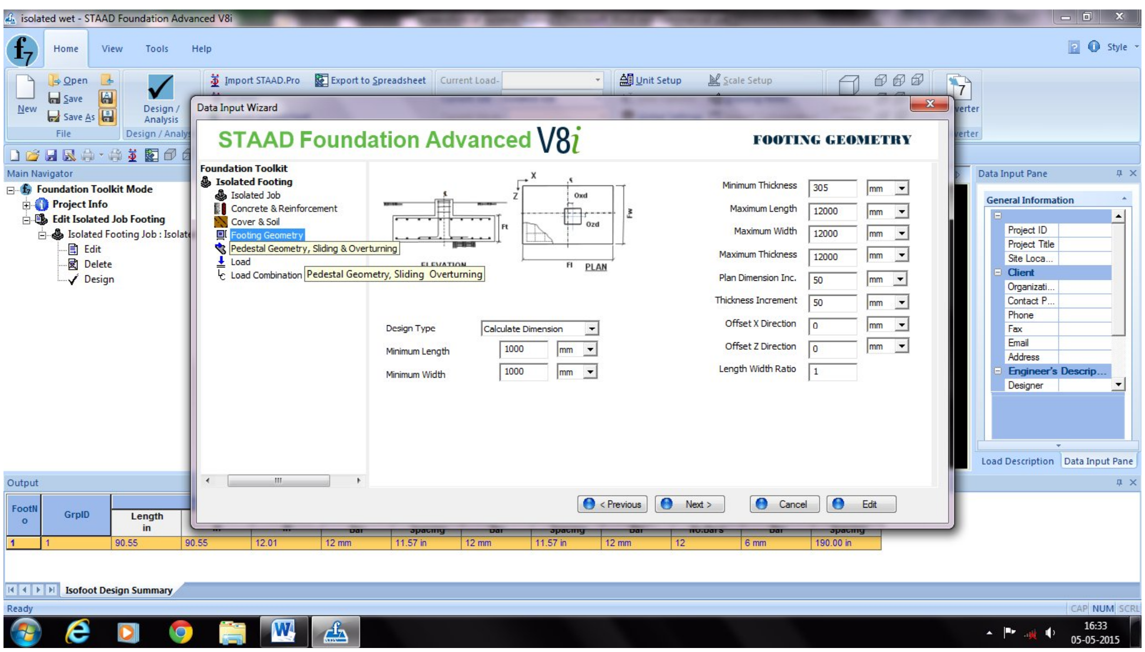


Fig 4.3 Footing geometry

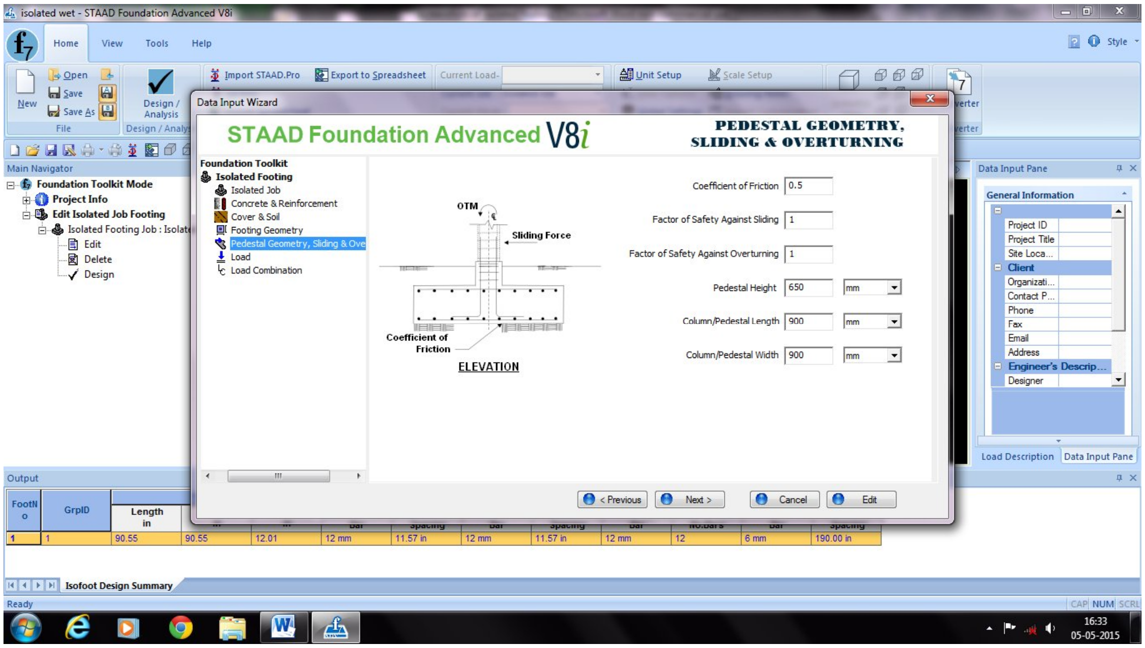


Fig 4.4 Pedestal geometry, sliding and overturning

4.2.5 Load

The foundation of any structure is subjected to various types of loads i.e., dead load of the superstructure, wind loads, earthquake loads. The resultant reactions at the supports of transmission tower are found out in staad pro v8i considering wind load and dead load . The loads are calculated in all the directions i.e., in X,Y and Z direction as mentioned in chapter 3. The footing is to be designed for extreme scenario which would be possible. Hence the reactions obtained at each support are analysed and only maximum value of the reaction in each direction is taken. Hence, the resultant reactions are then applied onto the footing for designing of the foundation.(Note : the sign convention has to be carefully examined before inputing value). Factor for wind load incase of transmission tower is to be taken as one with reference to shah (2014).

The following figure shows the application of loads in all the directions on the footing. The loads applied are listed below and are with reference to chapter 3.

F_x= 91kN

F_y= -156kN (negative sign indicates downward direction)

F_z= 38kN

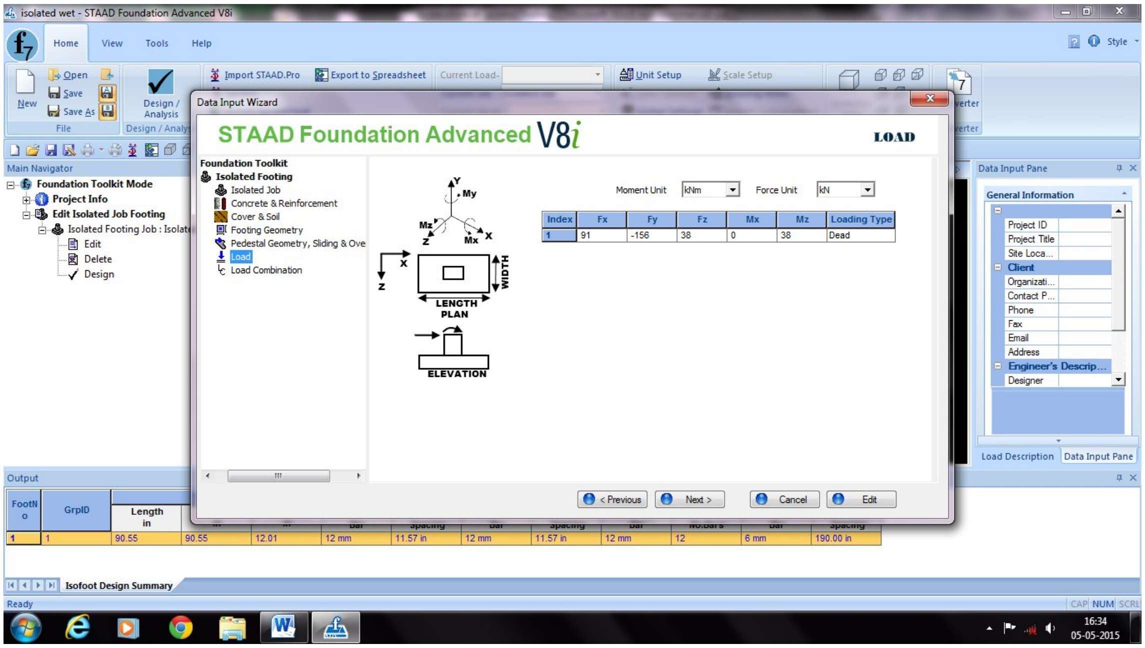


Fig 4.5 Loads

4.2.6 Output file

After inputting all the paramters required for design of foundation. The foundation is designed with its detail and schedule drawing and reinforcemnt details of the footing.

The detail and schedule drawing inlcudes footing dimensions and placement of reinforcement in the footing.

The reinforcement details includes the reinforcement of footing as well as of pedestal. The footing reinforcement includes top and bottom reinforcement with main and transverse steel for both of them. The pedestal reinforcement includes main and transverse steel.

In the present work, screenshots of detail and schedule drawing and table of reinforcement are shown.

The following figure shows the detail and schedule drawing of the foundation designed in the software.

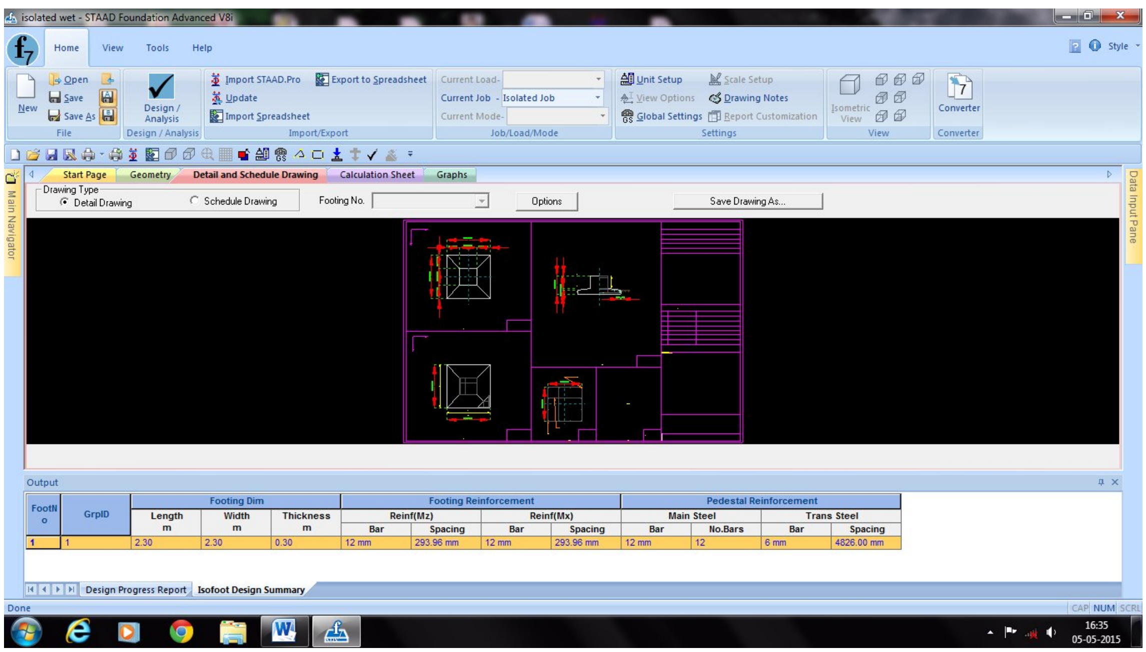


Fig.4.6 Detail and schedule drawing

For estimation of materials required for construction of foundation, it is important to get the dimensions of footing designed.

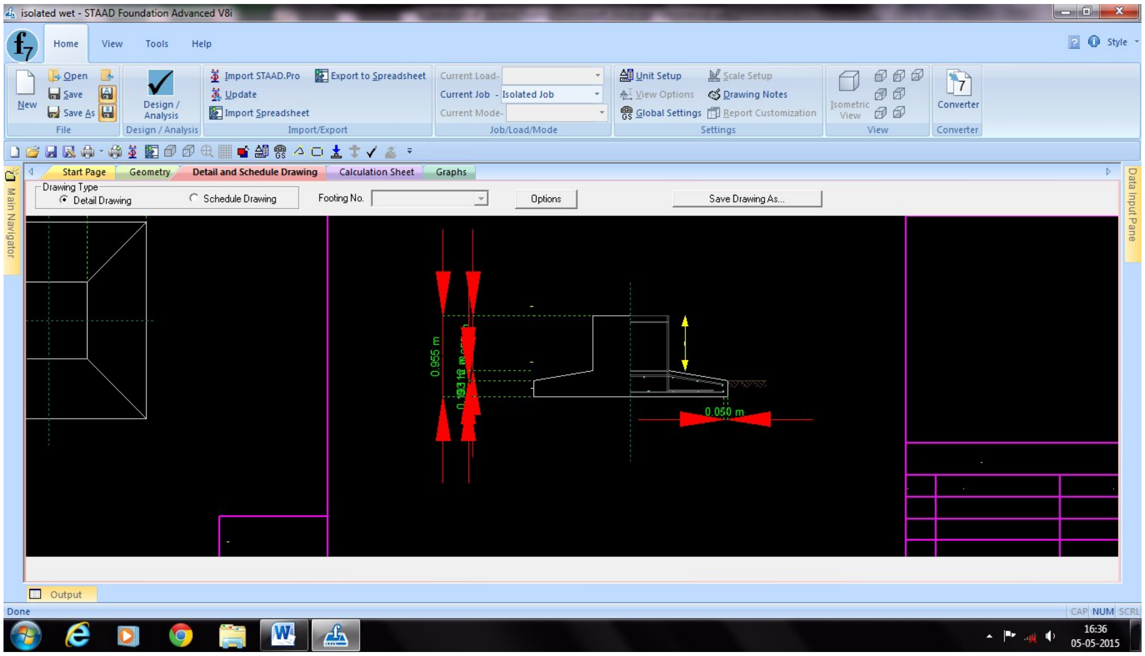


Fig.4.7 Footing dimensions

The following figure shows the footing dimensions.

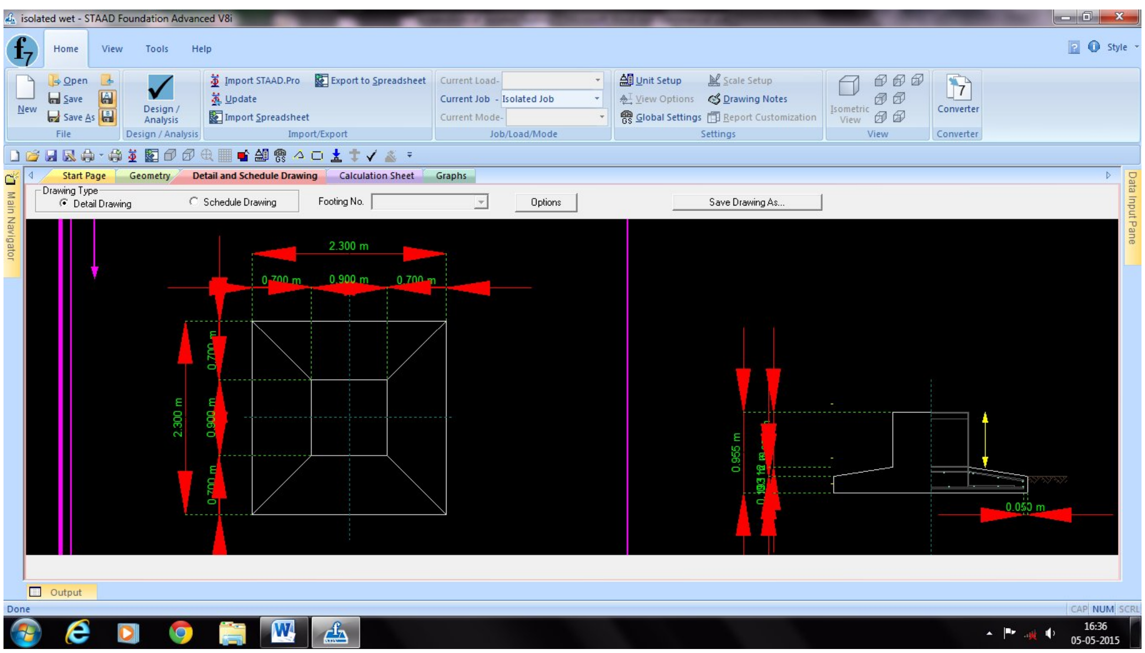


Fig 4.8 Footing dimensions

The following figure shows the footing dimensions.

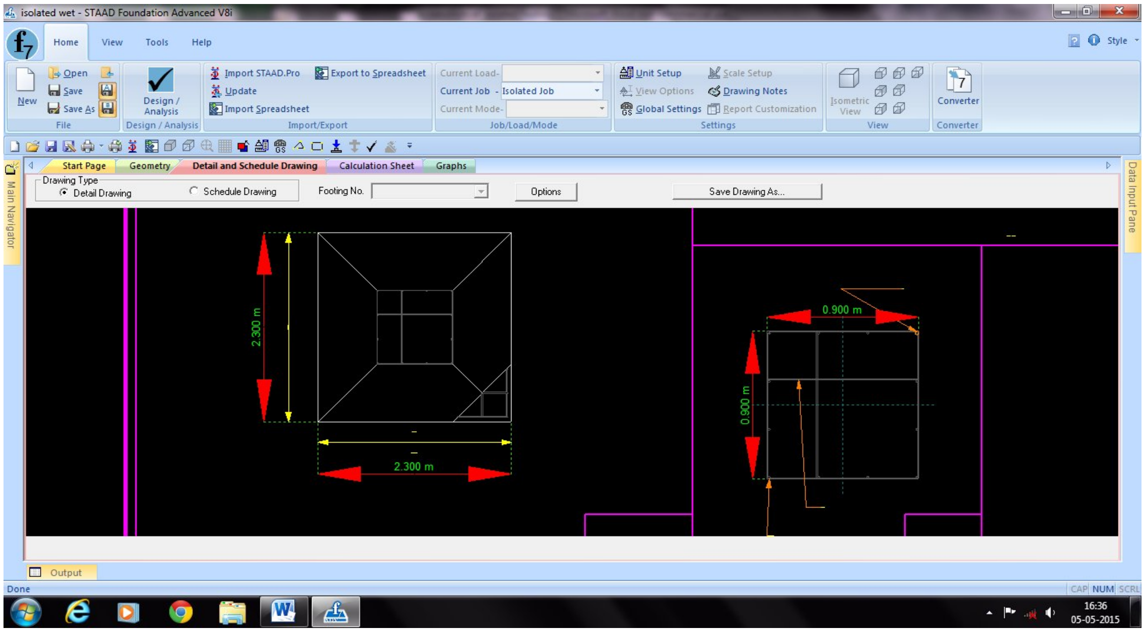


Fig 4.9 Footing dimensions

The following table shows the results of footing and pedestal geometry as obtained from Staad foundation.

Table.4.1 Footing geometry of software designed isolated footing

Footing geometry			pedestal geometry		
Length	width	Thickness	Length	width	thickness
(m)	(m)	(m)	(m)	(m)	(m)
2.2	2.2	0.305	0.9	0.9	0.65

The table showing the detailing of reinforcement for isolated foundation designed in Staad foundation is also presented.

Table 4.2 Reinforcement details of software designed isolated footing

FOOTING REINFORCEMENT				PEDESATL REINFORCEMENT	
Bottom		Top		Main	transverse
Main	transverse	main	transverse		
Φ12mm @ 295mm c/c	Φ12mm @ 295mm c/c	Φ12mm @ 295mm c/c	Φ12mm @ 295mm c/c	12-Φ12mm	Φ6mm @180mm c/c

4.3 MANUAL DESIGN OF FOUNDATION

In the previous section we used the reactions obtained from Staad pro v8i to design the foundation, results of which are shown in table 4.1 and 4.2. This section will cover, using the same reactions to design the foundation, using manual process which will be compared to the results of software design in the next chapter.

Manual design of transmission tower is entirely based on the design procedure and design steps as given in the manual followed by Kalpataru Power Transmission Ltd. As covered in chapter 2, transmission tower should be checked for overturning, uplift, sliding, side thrust etc before designing. The entire manual design will be explained in design steps below in context with theory of chapter 2. The designing will be carried out for the same soil condition, for which software design was made. The necessary input data required for manual design is provided Kalpataru Power Transmission Ltd, and is listed below.

Design of isolated foundation in wet black cotton soil

Transmission tower: 400 KV D/C Transmission line

Tower type: "DB"

Table 4.3 Loading on foundations

Description	Normal Condition (Reliability)(kgs)	Broken Wire Condition (Security)(kgs)
Down thrust	142243	136658
uplift	120392	117806
side thrust (T)	7844	8774
side thrust (L)	3709	4477

Grade of steel = $415 N/mm^2$

Grade of concrete = $20 N/mm^2$

Tower Slopes:

$\tan\theta = 0.1926$

True length factor = 1.036

Properties of soil:

Unit weight of wet soil	=	940kg/cu.m
Unit weight of dry soil	=	1440 kg/cu.m
Limit bearing capacity (wet locations)	=	13675kg/sq.m
Unit weight of concrete (dry)	=	2400kg/cu.m

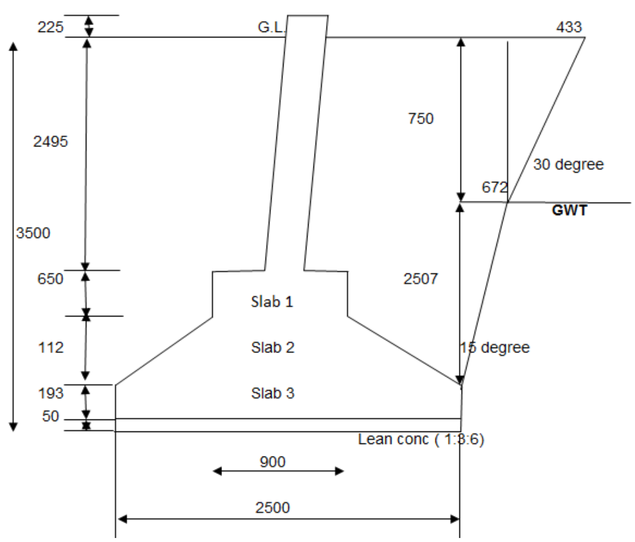


Fig 4.10 Isolated footing designed manually

4.3.1 Design Steps

4.3.1.1 Volume of Concrete (Cu.m.) :

First step is to calculate the total volume of concrete required. This quantity is major factor, along with the soil weight, which will counter act the uplift forces of the transmission tower. The volume of concrete is to be calculated in cubic metre which will later be multiplied by unit weight of concrete to convert it into kN, thus converting into force to counter act against uplift force. The volume of concrete is calculated by using simple geometry and with the help of the above figure.

PCC	2.5^2*0.05	=	3.130
Slab 3	2.5^2*0.193	=	1.732
Slab 2	0.112/3[2.5^2+0.9^2+2.5*0.9]	=	5.340
Slab 1	0.9^2*0.65	=	0.527
Chimney	0.6^2*2.72	=	0.979
Total		=	11.70 cu. m

4.3.1.2 Over Load of Concrete (kgs.):

This is calculated for the additional load that the soil will have to bear because of concrete. At the base of footing i.e., at a depth of 3500mm the load due to surcharge weight of the soil exists. This load is effectively being carried by the soil. When soil will be replaced by a more dense material in the form of concrete the load at the base of the footing will increase. This is the overload of concrete. Also the variation caused due to water table is to be taken into consideration, as for submerged portion of the soil the load exerted will decrease.

These overloads are calculated and shown below

	Compression	Uplift
0.6^2*0.225*2495=	194	194
(11.708-0.081)*(2400-1440) =	12266	-
0.6^2*0.75*(2400-1440) =	-	259
11.708-3.13-0.081-0.27)*(1400-940) =	-	3784
Total	12461Kgs	4238kg

4.3.1.3 Dry Soil Volume: (Cu.m)

Mentioned earlier, dry weight of soil is calculated to be used for counteracting force against uplift. The theory and formula involved for calculating volume of soil is mentioned in chapter 2.

A1=2.5^2+4*2.5*0.672+3.14*0.672^2	=	14.384
A2=2.5^2+4*2.5*(0.434+0.672) +3.14*(0.434+0.672) ^2	=	21.130

V= (0.75/3)[14.385 + 21.13 + SQRT(14.384*21.13)] = 60.267

4.3.1.4 Wet Soil Volume: (Cu.m)

Wet soil volume has similar purpose as those of dry soil i.e., counter act the uplift force.

Referring to the fig 4.10 above wet soil volume has been calculated below.

2.5^2*2.7	=	23.560
2.5*0.672*2*2.507	=	10.420
3.14/3*0.672^2*2.507	=	2.184
Total	=	36.164

4.3.1.5 Check for uplift

The check for uplift resistance is done based in context with the theory present in the literature review. The calculations are shown below. The resistance against uplift is compared to with the actual uplift and the latter should be less than the former. In calculations below validate the same.

Resistance against Uplift

60.27*1440+36.16*940+4238 = 125017kgs.

F.O.S (NC) =125017/1203 = 1.038>1.0

Hence o.k.

F.O.S (BWC) =125017/117806 = 1.061>1.0

Hence o.k.

4.3.1.6 Moment due to side thrust at foundation toe

The earth pressure is the major responsible force which will act against the overturning along with the dead load. The theory behind this has been discussed in chapter 2, which will be implemented here to calculate the moment due to side thrust. The earth pressure acting in a given direction (longitudinal or transverse) is calculated and is compared with the force acting in the in that direction to see whether the earth pressure will be mobilised or not. Based on this moment acting will be calculated which will be used in bearing capacity calculation.

The same theory will be applicable for both normal condition and broken wire condition.

A) Normal condition (transverse side thrust)

Earth pressure is given by: 1/2*w*h^2*B3*(1+sinΦ/1-sinΦ)

Where,	W	=940kg/m^3
	Φ =Angle of Earth Frustum	=15°
	B3	=0.6m

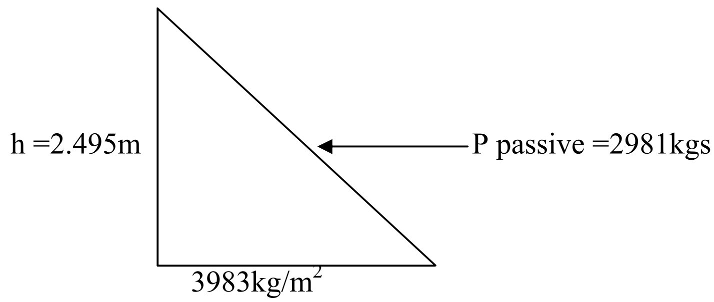
Height of mobilization = (2.495-0) =2.495m

Maximum passive force that can be mobilized

=0.5*w*h^2*B3*(1+sinΦ/1-sinΦ)

=2981kgs<7844kgs

Therefore, full passive pressure is mobilized.



Moment due to side thrust at the base of the footing

=7844*(3.45+0.225) -2982*(0.955+2.495/3)

=23500kg-m

B) Normal condition (longitudinal side thrust)

Side thrust force (F) =1/2*w*h^2*B3*(1+sinΦ/1-sinΦ)

Where	W	=940kg/m^3
	Φ =Angle of Earth Frustum	=15°
	B3	=0.6m

Height of mobilization = (2.495-0) =2.495m

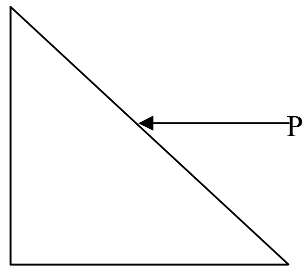
Maximum passive force that can be mobilized

=P passive

=0.5*w*h^2*B3*(1+sinΦ/1-sinΦ)

=2981kgs>3709kgs Therefore, full passive pressure is not mobilized.

h =2.495



P passive =3709kgs

4955kg/m2

Moment due to side thrust at the base of footing

=3709*(3.45+0.225) -3709*(0.955+2.495)/3)

=7004kg-m

C) Broken wire condition (transverse side thrust)

Side thrust force (F) =1/2*w*h^2*B3*(1+sinΦ/1-sinΦ)

Where	W	=940kg/m^3
	Φ =Angle of Earth Frustum	=15°
	B3	=0.6m

Height of mobilization = (2.495-0) =2.495m

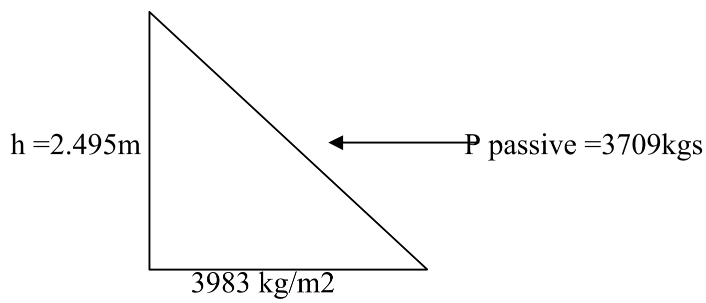
Maximum passive force that can be mobilized

=P passive

= 0.5*w*h^2*B3*(1+sinΦ/1-sinΦ)

2981kgs<8774kgs

Therefore, full passive pressure is mobilized.



Moment due to side thrust at the base of footing

=8774*(3.45+0.225) -2981.47*(0.955+2.495)/3

=26918kg-m

D) Broken wire condition (longitudinal side thrust)

Side thrust force (F) = 1/2*w*h^2*B3*(1+sinΦ/1-sinΦ)

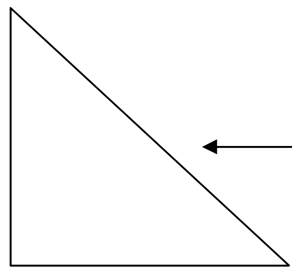
Where	W	= 940kg/m^3
	Φ =Angle of Earth Frustum	=15degrees
	B3	=0.6m
Height of mobilization = (2.495-0)		=2.495m

Maximum passive force that can be mobilized

=0.5*w*h^2*B3*(1+sinΦ/1-sinΦ)

=2981kgs<4477kgs

Therefore, full passive pressure is mobilized.



Moment due to side thrust at the base of footing

=4477*(3.45+0.225) -2981.47*(0.955+2.495)/3) =11126kg-m

4.3.1.7 Check for bearing capacity

This check is a very important check which determines whether the soil will be able to take the load or not. From chapter 2, the formula to be used to calculate the stress produced is given below

Bearing pressure = (P/A) +/- (M_x/Z_x) +/- (M_z/Z_z) (4.1)

Value of P will be taken from the given data i.e. it's the value of down thrust. Values of moments will be obtained from the previous steps.

A) Normal condition (NC)

=

[(142243/1.036+12461)/2.5^2]+[2*(142243/1.036)*0.193*1.005/(1/6 *2.5^ 3)]+[23499.822/(1/6*2.5^3)] + [7003.829/(1/6*2.5^3)]

=2309+1906+6098+2689

=13002kg/m^2<13675kg/m^2

Hence O.K.

B) Broken wire condition (BWC)

=(136658/1.036+12461)/2.5^2+2*(136658/1.036)*0.193*1.005/(1/6*2.5^3)+26917.572/(1/6*2.5^3)+11126.097/(1/6*2.5^3)

=2143+1809+5849+4272

=14073kg/m^2<13675kg/m^2

Hence O.K.

4.3.1.8 Design of base slab

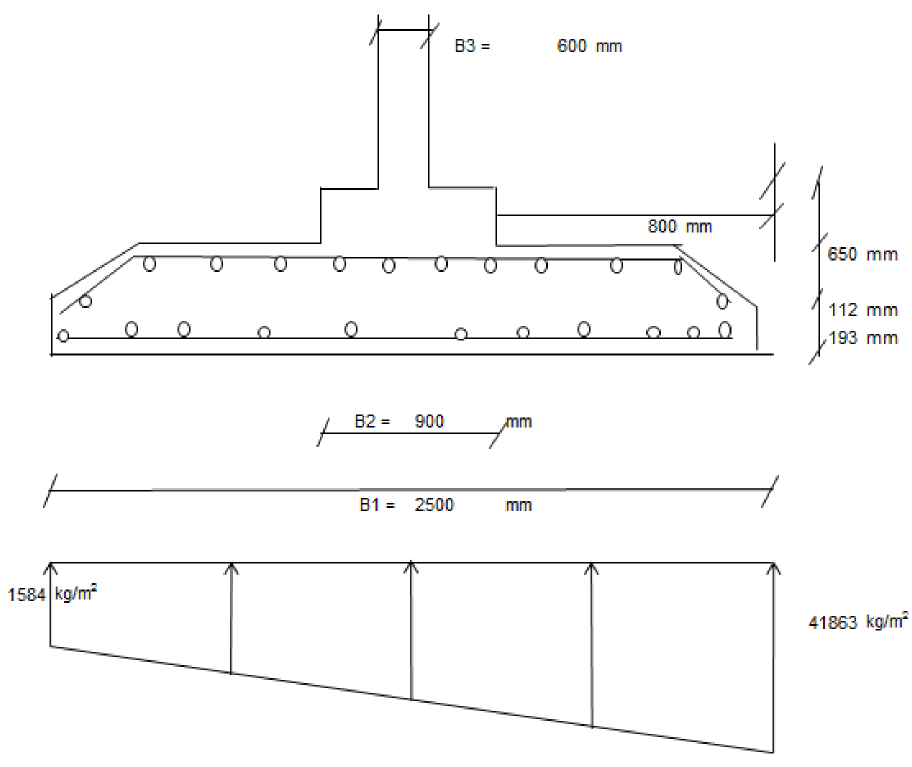


Fig 4.11 Stress Distribution

Design of base slab is in accordance with IS 456: 2000. Values of clear cover and diameter of bar are assumed and listed below.

Clear cover = 50mm

Diameter of main bar =12mm

Diameter/2 =6mm

Design Bearing Pressure

$$= (P/A) +/- (P*e_x/Z) +/- (P*e_z /Z) \tag{4.2}$$

= 21724 +/- (19067/2) +/- 10337

P_{max} $=41863 kg/m^2$

P_{max} $=0.41068 N/mm^2$

P_{min} $=1584 kg/m^2$

P_{min} $=0.01554 N/mm^2$

d1 = Eff. Depth =955-50-12-6 =887mm

Moment for Compression Reinforcement

M_{U1} =26557*2.5*(2.5-0.6)^2/8+(41863-26557)*2.5*0.95/2*2/3*0.95

=29960+11511 kg-m

=41471 kg-m

=406.83kN-m

Moment for Uplift Reinforcement

The top reinforcement are calculated using this moment. This reinforcement is essential in case of transmission tower foundation where uplift force plays a major role. Its calculated by dividing the uplift force acting by the area of footing as shown below.

Bearing pressure (p2) =120392/(2.5^2-0.6^2)

=20440 kg/m2

M_{U2} =20440.07*2.5*(2.5-0.9)^2/8 kg-m

=16352kg-m =156.97kN-m

Concrete & reinforcement steel properties

Grade of concrete 'fck'=20 N/mm2

Grade of steel 'fy'=415 N/mm2

A) Design of bottom pad under down thrust

M_{U1}	=406.83kN-m
Effective depth 'd1'	= 887mm
M_{u1}/(B2*d1*d1)	= 0.575
Mu/bd^2,limiting =0.133*f_{ck}	=2.66
Section is singly reinforced as 0.575<2.66	
From SP: 16 the values of pt for Mu/bd^2	= 0.575
P_t	=0.163
A_{st}	=1301 mm^2
Number of bars (12mm diameter)	=12
Spacing of Bars	= 217mm c/c
A_{st} provided	=1414 mm^2

Design of bottom pad under uplift

The design for uplift reinforcement will be done in the same way as above based IS 456-2000, only the value of moment will be changed.

Therefore,

M_{U2} =156.97kN-m

Number of bars (12 mm diameter) =14

Spacing of Bars = 182mm c/c

A_{st} provided =1583mm^2

The following table shows the footing and pedestal geometry

Table 4.4 Footing geometry of manually designed isolated footing

footing geometry			pedestal geometry		
Length	width	Thickness	Length	width	thickness
(m)	(m)	(m)	(m)	(m)	(m)
2.5	2.5	0.355	0.9	0.9	0.65

The following table shows the footing and pedestal geometry

Table 4.5 Reinforcement details of software designed isolated footing

FOOTING REINFORCEMENT				PEDESTAL REINFORCEMENT	
Bottom		Top		Main	transverse
Main	Transverse	main	transverse		
Φ12mm @ 217mm c/c	Φ12mm @ 217mm c/c	Φ12mm @ 180mm c/c	Φ12mm @ 180mm c/c	14-Φ12mm	Φ6mm @175mm c/c

4.4 PILE FOUNDATION

In the previous sections of this chapter, isolated sloped footing has been designed for the transmission line tower. This design for the foundation has been done manually and by using software, staad-foundation. This section will cover the concept of pile foundations, including both individual and group piles and computing the load carrying capacity of the pile foundation. The results obtained will be discussed further, commenting on suitability of the pile foundation to replace the isolated footing designed in previous sections.

Pile foundation is known as deep foundation. A pile is a long slender foundation member, made either of timber, structural steel or concrete. Pile foundation is required when the soil bearing capacity is not sufficient for the structure to withstand. This is due to the soil condition or the order of bottom layers, type of loads on foundations, conditions at site and operational conditions. A pile foundation consists of two components: Pile cap and single or group of piles. Piles transfer the loads from structures to the hard strata, rocks or soil with high bearing capacity.

4.4.1 Single pile

Pile capacity can be defined as the maximum load on the piles or group of piles that will produce failure. In other words it can be thought of as the maximum load value which the pile can transfer to the soil, without failing. To determine the capacity of pile and group of pile there are many static, dynamic and other empirical relations available. This section is limited to the calculation of load carrying capacity of piles using static formulas. Based on this formula the ultimate load carrying capacity of pile is given as below.

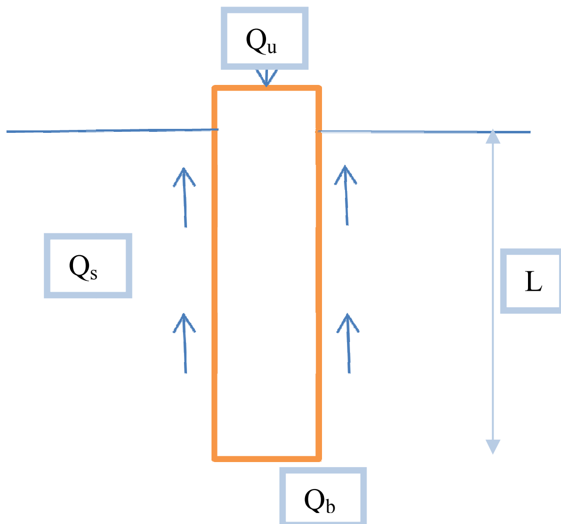


Fig 4.12 Load carrying capacity of pile

$$\boxed{Q_u=Q_s+Q_b} \quad (4.3)$$

Q_s=resistance due to skin friction. Depends on the length and diameter of pile

Q_b=end bearing resistance.

Q_u=ultimate load carrying capacity of pile in kN.

The above formula can be further written as:

$$\boxed{Q_u=f_sA_s+f_bA_b} \quad (4.4)$$

A_s=surface area of the pile (ΠdL for circular piles)

A_b=end bearing area of the pile ($(\Pi/4)d^2$ for circular piles)

f_s=minimum strength of surface pile= αC_α for cohesive soils

f_b=base resistance= CN_c for cohesive soils

As described in previous chapters the site of the transmission tower, based on various experimental results and survey, indicated soil type as black cotton soil with ground water table present at a depth of 750mm from the ground level. Other important soil parameters include angle of internal friction of soil(Φ)=0°, cohesion value(C) = 35.37, 37.73 kN/m^2 for dry and wet soils respectively.

Pile capacity will be calculated for circular pile of diameter 0.25m and length 6m. The diameter of the pile will be increased from 0.25m to 0.5m and then from 0.5m to 0.75m and capacity of each of these piles will be calculated. Another change in dimensions of the pile will include keeping the diameter constant and increasing the depth of the pile from 6m to 8m, 8m to 10m and 10m to 12m, again calculating load carrying capacity of each combination. Thus by changing the values of diameter from 0.25 to 0.75m and length from 6m to 12m, capacity of all 12 possible combinations of length and diameter will be calculated. For present site condition above formula can be modified and written as shown below.

$$\boxed{Q_u= (\alpha*C_\alpha*\Pi*D*L)+(C*N_c*(\Pi/4)*D*D)} \qquad (4.5)$$

Where

Q_u= ultimate capacity of a pile

a= adhesion factor

C_α = cohesion value w.r.t to skin of pile

D = diameter of pile

L= length of pile

C= Cohesion at pile base

N_c=coefficient of cohesion =9 for piles

Using above equation the load that can be transferred by a circular pile can be calculated and calculation for pile of length 6m and diameter 0.25 is shown below (note: value of α is taken as 1 for normally consolidated clays and 0.5 for over consolidated clays)

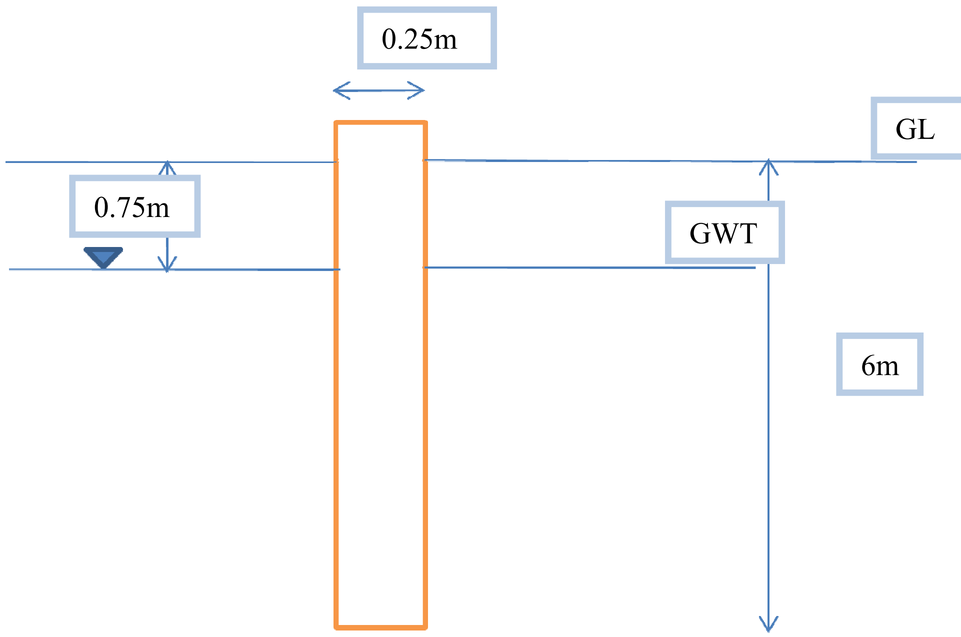


Fig 4.13 6m pile of of 0.25m dia in black soil

Resistance due to skin friction (Q_s)

$Q_s = (1*35.37*\Pi*0.25*0.75) + (1*37.73*\Pi*0.25*(6\text{-}0.75))$

=176.31 kN

$Q_b = (37.73*9*(\Pi/4)*0.25\text{^}2)$

=16.69 kN

Q_u=Q_s+Q_b=192.98 kN

Hence load carrying capacity of pile of 0.25m diameter and 6m depth is 192.98 kN. Similarly value for capacity of pile with 0.25m diameter and 8m depth, 0.5m diameter and 6 m depth, 0.75m diameter and 12m depth etc can be calculated. The table below shows the capacity of piles for all 12 possible combinations of selected length and diameters.

Table 4.6 Capacity (Q) of a single pile with different combinations in kN

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	192.98	419.31	678.97
8	252.22	537.78	856.68
10	311.45	656.25	1034.39
12	370.69	774.72	1212.09

4.4.2 Group piles

Pile group can be defined as a combination of two or more piles placed together. The number of piles in a group may vary depending upon the requirement and amount of load to be transferred. In the present work two group piles and four group piles have been taken into consideration.

Capacity of pile group is the sum of the individual capacities of piles, but it is influenced by the spacing between the piles. Piles are driven generally in groups in regular pattern to support the structural loads. The structural load is applied to the pile cap that distributes the load to individual piles. If piles are spaced sufficient distance apart, then the capacity of pile group is the sum of the individual capacities of piles. However, if the spacing between piles is too close, the zones of stress around the pile will overlap and the ultimate load of the group is less than the sum of the individual pile capacities specially in the case of friction piles, where the efficiency of pile group is much less.

Group action of piles is evaluated by considering the piles to fail as a unit around the perimeter of the group. Both end bearing and friction piles are considered in evaluating the group capacity. End bearing pile is evaluated by considering the area enclosed by the perimeter of piles as the area of footing located at a depth corresponding to the elevation of pile tips. The friction component of pile support is evaluated by considering the friction that can be mobilized around the perimeter of the pile group over the length of the piles. Final group capacity of the pile will be taken as the minimum of group action of and sum of individual capacity.

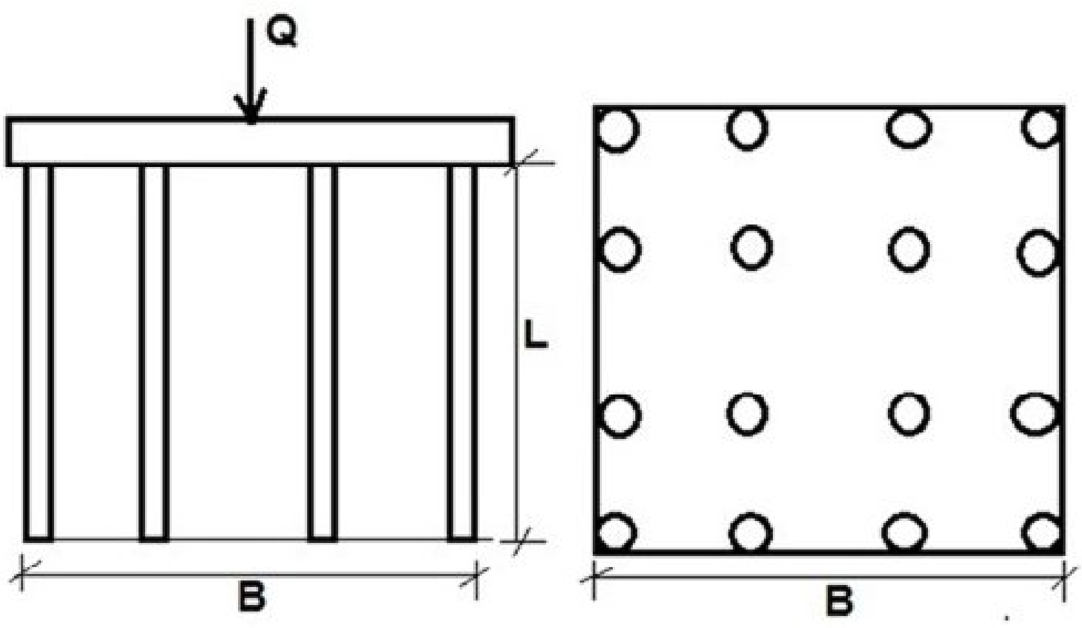


Fig 4.14 group pile

Q_u= nos of piles * capacity of individual pile

$$= n*[(\alpha*C_\alpha*\Pi*D*L)+(C*N_c*\Pi*D*D)] \quad (4.6)$$

Q_u= considering failure of entire block of piles

$$= (\alpha*C_\alpha*L*4*B)+(C* N_c *(B\^2))$$

$$= (\alpha*C_\alpha*L*4*(3S+d))+(C* N_c *((3S+d)\^2)) \quad (4.7)$$

S= spacing b/w two piles in a pile group. It's taken as 3*diameter of the pile

Minimum of the above two values will be taken as the capacity of the pile group.

4.4.2.1 Pile group- 4 piles

As discussed above the scope of this project is to find out the capacity of pile group with four piles. In previous section, for single piles, we considered different combinations of diameter and length of piles. The values of diameter were 0.25m, 0.5m, 0.75m and of length were 6, 8, 10, 12m. The same set combinations will be considered for group piles i.e., for both 2 group and 4 group piles. Hence, group capacity for 12 combinations of pile group will be calculated and tabulated. The calculation for group pile consisting of four piles of diameter 0.25m and depth 6m is shown below. The rest combinations have been calculated in the same and tabulated in the table below. The soil properties are considered to be similar to that of single piles with water table also at the same level of 750mm from ground level.

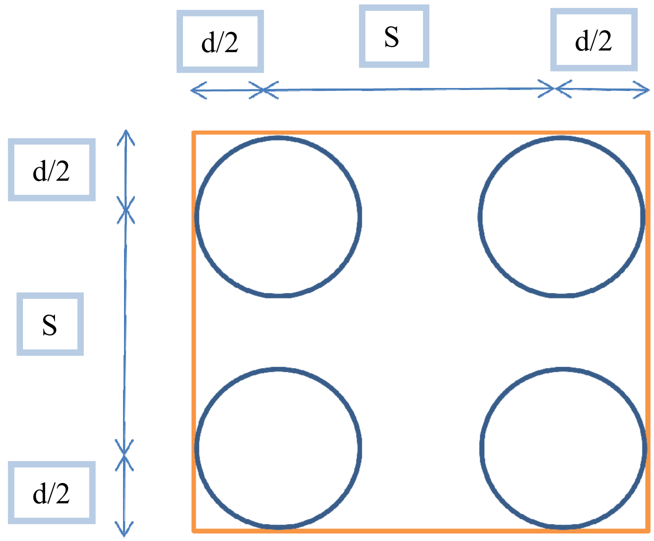


Fig 4.15 pile group- 4 piles

For pile with diameter (d) = 0.25m and length (L) =6m, above formula becomes

$$Q_u = n*[(\alpha*C\alpha*P*\Pi*L)+(C*Nc*\Pi*D*D)] \qquad (4.8)$$

$$= 4*[((1*35.37*\Pi*0.25*0.75) + (1*37.73*\Pi*0.25*(6-0.75)) + (37.73*9*(\Pi/4)*0.25^2)]$$

$$= 771.94 \text{ kN}$$

$$Q_u = (\alpha*C_\alpha*L*4*(S+d)) + (C* N_c *((S+d)^2)) \qquad (4.9)$$

$$=[(1*35.37*0.75*4*(0.25*3+0.25)) + (1*37.73*(6-0.75)*4*(0.25*3+0.25))] + (37.73*9*(3*0.25+0.25)^2)$$

$$=1238.01 \text{ kN}$$

Hence taking minimum of the two values the capacity for the pile group with four piles is 771.94 kN. Similarly values for other combination are calculated. Values obtained from the block failure criteria are tabulated in table 4.6. Values for group capacity considering individual sum of pile capacities are in table 4.4 and the final values of the group capacities i.e., minimum of the two listed above are finally listen in table 4.7, which is the final result for group capacity of pile.

Table 4.7 group capacity of piles based on individual failure criteria

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	771.94	1677.24	2715.89
8	1008.89	2151.13	3426.73
10	1245.83	2625.02	4137.56
12	1482.78	3098.91	4848.39

Table 4.8 Group capacity of piles based on block failure criteria

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	1238.01	3155.16	5751.45
8	1539.85	3758.84	6656.97
10	1841.69	4362.52	7562.49
12	2143.53	4966.2	5751.45

Table 4.9 actual group capacity of piles, minimum (Table 4.7,Table 4.8)

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	771.94	1677.24	2715.89
8	1008.89	2151.13	3426.73
10	1245.83	2625.02	4137.56
12	1482.78	3098.91	4848.39

4.4.2.2 Pile group- 2 piles

This is similar to 4 grouped piles. The basic formula involved to calculate the load carrying capacity of pile group is same as listed in section 4.4.2.1 the changes involved in the formula will be in the number of piles, which will become 2 for this case while considering formula 4.9 and changes in the block area for formula 4.10 Hence as per the figure shown below the formula modifies as,

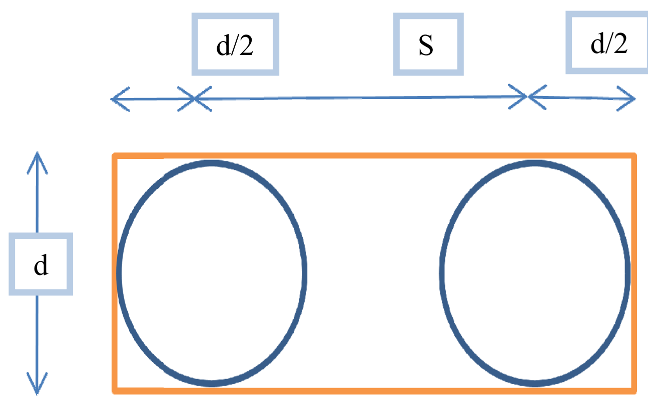


Fig 4.16 pile group- 2 piles

$$Q_u = (\alpha * C_\alpha * L * 2 * ((S+d)+d)) + (C * N_c * (((S+d)+d)\hat{}2)) \qquad (4.10)$$

$$Q_u = n*[(\alpha * C\alpha * d * \Pi * L)+(C * Nc * \Pi * D * D)] \qquad (4.11)$$

Using above formula 4.10 block capacity of piles is calculate in similar way as shown in section 4.4.2.1 and is tabulated in table 4.11. Formula 4.11 yields table no 4.10 which is the group capacity based on sum of capacity of individual pile. And final table yields the final load carrying capacity of pile which is the minimum of table 4.10 and table 4.11

Table 4.10 Group capacity of piles based on individual failure criteria

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	385.97	838.62	1357.94
8	504.44	1075.56	1713.36
10	622.91	1312.51	2068.78
12	741.39	1549.45	2424.19

Table 4.11 Group capacity of piles based on block failure criteria

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	646.41	1462.62	2448.60
8	835.06	1839.92	3014.55
10	1023.71	2217.22	3580.50
12	1212.36	2594.52	3467.31

Table 4.12 Actual group capacity of piles, minimum (table 4.11,table 4.12)

	diameter(m)		
	0.25	**0.5**	**0.75**
length(m)			
6	385.97	838.62	1357.94
8	504.44	1075.56	1713.36
10	622.91	1312.51	2068.78
12	741.39	1549.45	2424.19

Table 4.3, 4.4and 4.12 are the final tables which show the amount of load that can be carried be individual pile, four grouped pile and two grouped pile respectively. These capacities will be compared to the load carrying capacities of isolated footing in the upcoming chapter to discuss the feasibility to replace isolated footing with pile foundation.

CHAPTER 5- RESULTS AND DISCUSSIONS

5.1 GENERAL

In the previous chapter 4, DESIGN OF FOUNDATIONS, isolated footing is designed in staad foundation as well as manually. The results of both of their designs are given in table 4.1, 4.2 and figure 4.6, 4.7, 4.8, 4.9 respectively. Also the capacity of isolated footing is calculated in section and the capacities of single pile foundation and group pile foundation are calculated and given in table 4.2, 4.5 and 4.8 respectively.

The present chapter is divided into two parts. Part one consists of comparison of software design of isolated footing in black cotton soil having depth of water table 750mm with manual design of isolated foundation in black cotton soil having depth of water table 750mm. Part two consists of comparing capacity of isolated footing with capacities of single pile foundation and group pile foundation and thereby commenting on the suitability of foundation in black cotton soil having depth of water table 750mm.

5.2 COMPARISON OF THE MANUAL DESIGN AND SOFTWARE DESIGN OF FOUNDATION

As described in section 5.1 the results of isolated footing designed manually and with software are given in chapter 4. Results of footing geometry for both of the footings are represented with the help of figures and tables.

Reinforcement details of isolated footing designed in Staad foundation is given in table 4.1. The following figure shows the footing geometry of isolated footing designed in software. The figure and the table given below will be used for estimation of quantity of work.

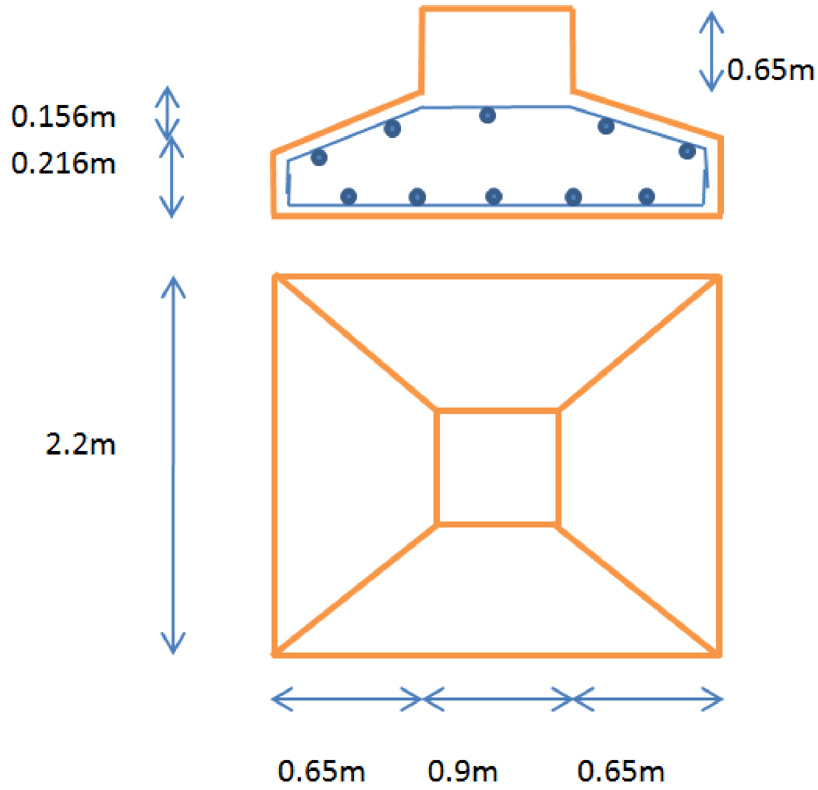


Fig 5.1 Footing geometry of software designed isolated footing

Footing geometry obtained by designing foundation in staad foundation is tabulated below

Table.5.1 Footing geometry of software designed isolated footing

Footing geometry			pedestal geometry		
Length	width	thickness	Length	width	Thickness
(m)	(m)	(m)	(m)	(m)	(m)
2.2	2.2	0.305	0.9	0.9	0.65

Reinforcement details of isolated footing designed manually is given in table 4.2. The following figure shows the footing geometry of isolated footing designed manually. The figure and the table given below will be used for estimation of quantity of work.

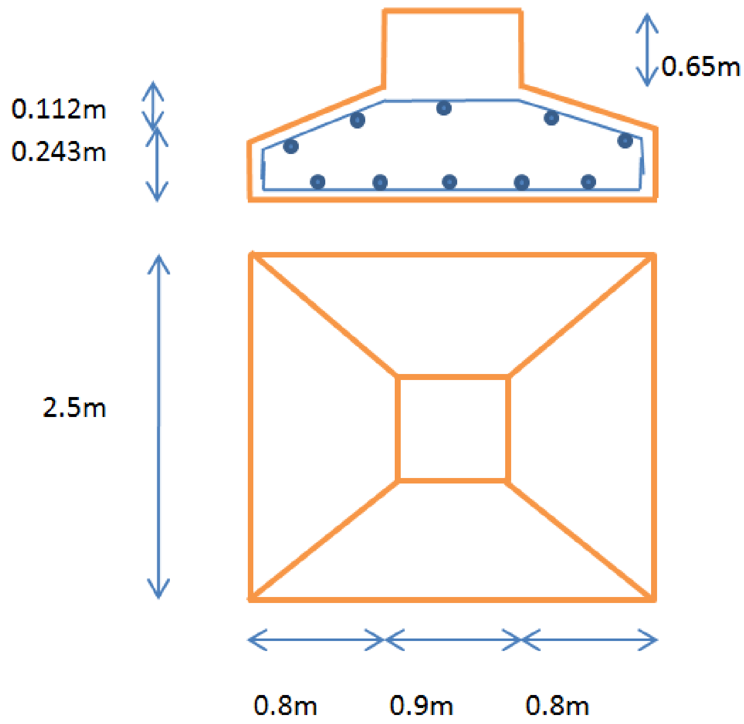


Fig. 5.2 Footing geometry of isolated footing designed manually

Footing geometry obtained by designing foundation in staad foundation is tabulated below.

Table 5.2 Footing geometry of manually designed isolated footing

footing geometry			pedastal geometry		
length	width	thickness	Length	width	Thickness
(m)	(m)	(m)	(m)	(m)	(m)
2.5	2.5	0.355	0.9	0.9	0.65

With the help of table 4.1 the estimate of quantity of steel for isolated foundation designed in software has been presented below.

Table 5.3 Estimation of quantity of steel required in footing designed in software

Sr. No.	Reinforcement	Bar dia(mm)	Spacing/No of bars	Steel required (kg)	Rate (Rs)	Amount (Rs)
1	Bottom				54,871/ton	6226.42
	Main	12	@' 295mm c/c	43		
	Transverse	12	@' 295mm c/c			
2	Top					
	Main	12	@' 295mm c/c	45.2		
	Transverse	12	@' 295mm c/c			
3	Pedestal					
	Main	12	12	25.274		
	Transverse	6	@' 180mm c/c			
Total				113.47		

With the help of table 4.2 the estimate of quantity of steel for isolated foundation designed manually is presented below

Table 5.4 Estimate of the quantity of steel required in footing designed manually

Sr. no.	Reinforcement	Bar dia(mm)	Spacing/No of bars	Steel required (kg)	Rate (Rs)	Amount (Rs)
1	Bottom				54,871/ton	6937.42
	Main	12	@' 217mm c/c	46.48		
	Transverse	12	@' 217mm c/c			
2	Top					
	Main	12	@' 180mm c/c	52.3		
	Transverse	12	@' 180mm c/c			
3	Pedestral					
	Main	12	14	27.64		
	Transverse	6	@' 175mm c/c			
Total				126.43		

Comparing table 5.3 and 5.4 it is observed that the total amount of steel required for isolated footing designed using software is 113.47kg whereas the total amount of steel required for isolated footing designed manually is 126.43kg. This shows that the isolated footing designed manually is requiring 12.96kg of steel more as compared to that of isolated footing designed in software. It is also observed that the total cost for steel for isolated footing designed using software is Rs.6226.42 whereas the total cost for isolated footing designed manually is Rs 6937.42. This shows that the isolated footing designed manually is requiring Rs 711 more amount of money as compared to that of isolated footing designed in software.

Referring to table 5.1, the following table of estimation of materials for isolated footing designed in software is given below

Table 5.5 Estimate of the quantity of work to be done in isolated footing designed in software

Sr. No.	Name of items and details of work	Quantity (cu.m)	Rate (Rs.)	Amount (Rs.)
1.	Earthwork in excavation in foundation	18.515	236	4369.54
2.	Cement concrete(1:3:6 base)	0.2645	2604	688.75
3.	R.C.C work in footing	1.324	3692	4888.208
4.	R.C.C work in pedestal	0.5265	4792	2522.98
	Total			12469.478

Referring to table 5.2, the following table of estimation of materials for isolated footing designed in software is given below

Table 5.6 Estimate of the quantity of work to be done in isolated footing designed manually

Sr. No	**Name of the items**	**Quantity(cu.m)**	**Rate (Rs)**	**Amount(Rs)**
1	Earthwork in excavation	21.875	236	5162.5
2	Cement concrete(1:3:6 base)	0.313	2604	815.052
3	RCC work in footing			
	Slab1	1.206	3692	4452.552
	Slab2	0.534	3692	1971.528
	Slab3	0.527	3692	1945.684
Total				14347.316

Comparing table 5.5 and 5.6 it is observed that for quantity of earthwork in excavation in isolated footing designed manually is 3.36 cum more than that of isolated footing designed in software. Looking onto cement concrete it is observed that isolated footing designed manually has 0.0485 cum of concrete work more than that of isolated footing designed in software. For RCC work, it has been observed that isolated footing designed manually has 0.4165 cum of concrete work more than that of isolated footing designed in software. Taking into account the total cost required for carrying out the work for construction of foundation, the total cost for work to be done for isolated footing designed manually is Rs 1877.83 more than that of isolated footing designed in software.

5.3 COMPARISON OF CAPACITY OF PILE FOUNDATION AND ISOLATED FOOTING

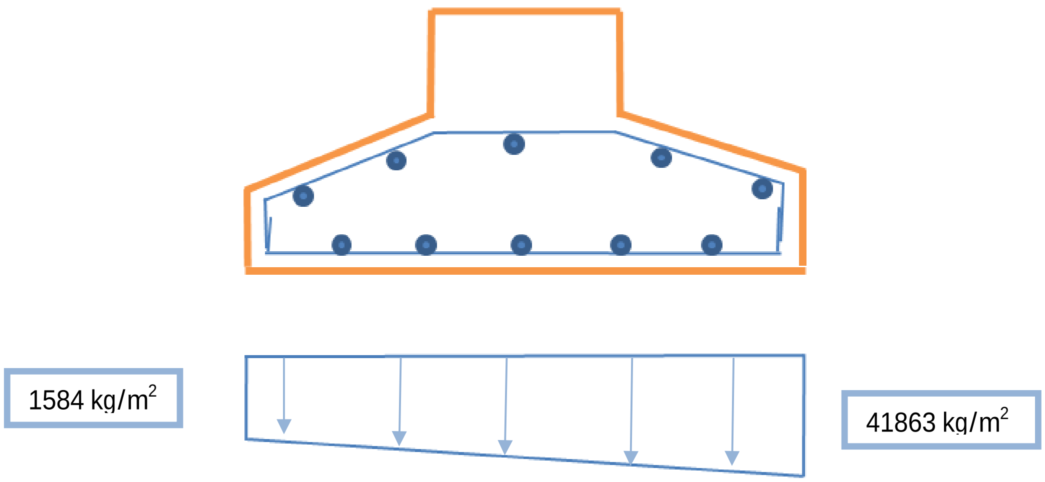


Fig 5.3 Stress distribution under isolated footing

Total load distributed by isolated footing 1350 kN as seen in chapter 4. This section will aim to compare this value with load carrying capacity of pile foundation. The different types of pile used for comparison purpose include both single pile and group piles (4 piles group and 2 piles group) the results of which have been calculated in the previous chapter.

First comparison is done between single pile and isolated footing, the soil conditions being black cotton soil with depth of water table 750mm. When pile of diameter 0.25m and length 6m is considered, its load carrying capacity comes out to be

192.98 kN. Load transferring capacity of this pile is very less than that of isolated footing. As the diameter increased from 0.25m to 0.5m and further from 0.5m to 0.75m it is observed that the load carrying capacity of pile increases, but still it is less than that of the isolated footing. Hence increase in diameter is accompanied with an increase in depth of the pile. The depth of pile is increased from 6m to 8m, 8m to 10m and consequently from 10m to 12m. Significant increase in the pile capacity is observed by referring to table 4.6 of where capacities have been calculated. It increases to 1212 kN. But still isolated footing can transfer greater amount of loads as compared to single pile foundation.

Hence the study is done for group pile. There are two sets of group piles one with two piles and other with four piles as discussed in chapter 4. Capacities of each of these groups can be referred from table 4.12 and table 4.9 respectively. Conclusion drawn from the comparisons done between pile foundation with isolated footing is that, that in case of two grouped pile, the load carrying capacity of pile group is still less than that of isolated footing. Referring table 4.12 pile group with diameter 0.25m and length 6m,8m,10m,12 m, all have load carrying capacities less 1350kN(load transferred by isolated footing). However when diameter is increased from 0.25m to 0.5m for depth of 12m, the capacity is 1550kN. This pile group can be used effectively in place of the above designed isolated footing in case of weak soils. If diameter is further increased to 0.75m, all depths of pile under consideration can be considered for replacement of isolated footing.

On comparing with capacities in table 4.9 i.e., the capacity for four grouped pile, apart from piles with diameter 0.25m and depth 6m, 8m, 10m, all other set of groups have sufficient capacities. Hence they can be used for foundation purpose of transmission tower in replacement of isolated footing for transmission tower. A few set of groups can also be used for even worse soil conditions for e.g. weak soil. These sets include group piles consisting of four piles with diameter 0.75m and depth 8m, 10m, and 12m. In two grouped pile as well as piles with diameter 0.75m depth 10m and 12m have load carrying capacities of 2070 kN and 2430 kN respectively. These sets can also be considered for soil which has more swampy nature or less bearing capacity.

CHAPTER 6 - CONCLUDING REMARKS

6.1 GENERAL

The study has presented a comparison between manual design and software design of an isolated footing of transmission tower for validation purpose and comparing load carrying capacity of isolated footing and pile foundation including both individual pile and group pile. The group pile consisted of two pile group and four pile group only in the scope of the study. This chapter is divided in two parts. Part one consists of conclusion of present investigation and part two consists of future scope of the study.

6.2 CONCLUSION

The study shows that the designing of footing of transmission tower using software gave a slight different result from the manual design of the same footing, under same type of soil and same soil conditions. Though the design obtained by both methods were different, this difference was not significant. Minute difference was observed between the dimensions of the footing. The manually designed footing has a footing of dimensions 2.5 x 2.5 x 0.355 m whereas software yielded a design of 2.2 x 2.2 x 0.305m, referring from previous chapter. Similarly the difference in reinforcement details didn't show significant variation as well. Hence, a conclusion can be derived that both software based design and manual design of the isolated footing of transmission tower yield results with very minute variation. Thus, it can be concluded that manual design validates the designing done by the software.

The present work further compared the load carrying capacities of pile foundation and isolated footing. It was found that in case of transmission towers, isolated footing is best suited in most of the cases. It functioned even better than individual piles as shown in the previous chapter. However where adverse soil or when weak condition exists, pile foundation of group pile proves to be a better alternative. However, individual piles don't show satisfactory result and group piles with four piles or two piles can be used. Four grouped piles show the best result and two grouped pile can also be used.

6.3 FUTURE SCOPE

Even after the final analysis and conclusion some areas transmission tower foundation remain untouched, and can be investigated in future. These areas include the following points given below:

6.3.1 Cost analysis of pile foundation has not been covered in this study. Costing of pile and pile group can be done and compared with cost of isolated footings. It can give results which can determine which type of foundation is more cost effective.

6.3.2 Another study that can be carried out in future is that that the validation of software can also be carried out using other type of foundations. For e.g., Mat foundation, combined foundation, etc.

6.3.3 In the present investigation the load carrying capacities of isolated and pile are compared to find out the suitability in black cotton soil. In future, the same study can be carried out using different types of foundation and then comparing their capacities. Different types of foundation include combined foundation and mat foundation.

6.3.4 For validation of software results of foundation, only staad foundation used. In future work validation can be done for different software for e.g., Rivet, Plaxis, etc.

6.3.5 In the present study, analysis and design of foundation is done only in black cotton soil having depth of water table 750mm. In future, analysis and design of foundation can be carried out for different type of soil. For e.g., dry soil, sandy soil, etc.