Foundation Design of a Shopping complex at Pratap Vihar, Ghaziabad, Uttar Pradesh

By

SHIRAGRA SINGH - 111654

Under the guidance of
Dr. S.K. Jain
(Former Associate Professor)

Submitted in partial fulfillment

of

the Degree of Bachelor of Technology

to

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JAYPEEUNIVERSITY OF INFORMATION TECHNOLOGY
WAKNAGHAT

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CERTIFICATE

This is to certify that the work entitled, "**Foundation Design of a Shopping complex at Pratap Vihar, Ghaziabad, Uttar Pradesh**" submitted by **SHIRAGRA SINGH** in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering at Jaypee University of Information Technology has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

(Dr. S.K. JAIN)
Professor,
Department of Civil Engineering
Jaypee University Anoopshahr, Distt- Bulandshahr,
Former Associate Professor,
JUIT Solan, Waknaghat - 173234

Certified the above mentioned project work has been carried out by the said group of students.

(Dr. ASHOK K. GUPTA)
Professor and Head
Department of Civil Engineering
Jaypee University of information Technology
Solan, Waknaghat - 173234

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Shiragra Singh (111654)

ABSTRACT

This Project work titled “**Foundation Design of a Shopping complex at Pratap Vihar, Ghaziabad, Uttar Pradesh.”** involves the study and analysis of the soil profile at Pratap vihar, Ghaziabad to calculate the bearing capacity of the soil at various depths with the help of the geotechnical data obtained from Plate load test, Standard Penetration test and direct cone penetration test which were performed on the soil by the geotechnical experts.

Further, this work involves the Design of foundation of a shopping complex on the same soil profile using R.C.C. analysis with the help of the bearing capacities calculated from different tests for various depths.All the design considerations and norms according to the Indian standard codes have been thoroughly followed. Main aim of this Project work is to provide ready to implement design tecchniques and requirements.

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1.1 General:

Basic Definitions

Bearing capacity: It is the load carrying capacity of the soil.

Ultimate bearing capacity or Gross bearing capacity (q_u): It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

Net ultimate bearing capacity (q_{nu}): It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it. Assuming the density of the footing (concrete) and soil (γ) are close enough to be considered equal, then

$\mathbf{q_{nu} = q_u - \gamma D_f}$.

Where, D_f = depth of the footing

Safe bearing capacity: It is the bearing capacity after applying the factor of safety (FOS). These are of two types:

- **Safe net bearing capacity (q_{ns})** : It is the net soil pressure which can be safety applied to the soil considering only shear failure. It is given by,

$$q_{ns} = \frac{q_{nu}}{FOS}$$

- **Safe gross bearing capacity (q_s)**: It is the maximum gross pressure which the soil can carry safely without shear failure. It is given by,

$$q_s = q_{ns} + \gamma D_f.$$

Allowable Bearing Pressure: It is the maximum soil pressure without any shear failure or settlement failure.

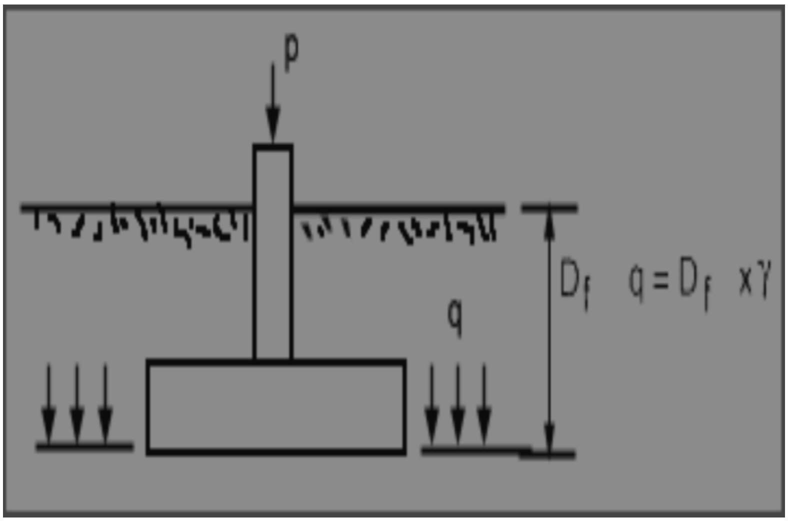
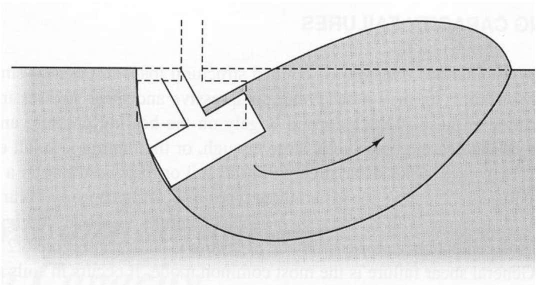


Fig.1 Bearing capacity of footing

Bearing capacity:

Building codes of various organizations in different countries gives the allowable bearing capacity that can be used for proportioning footings. These are "Presumptive bearing capacity values based on experience with other structures already built. As presumptive values are based only on visual classification of surface soils, they are not reliable. These values don't consider important factors affecting the bearing capacity such as the shape, width, depth of footing, location of water table, strength and compressibility of the soil. Generally these values are conservative and can be used for preliminary design or even for final design of small unimportant structure. IS1904-1978 recommends that the safe bearing capacity should be calculated on the basis of the soil test data. But, in absence of such data, the values of safe bearing capacity can be taken equal to the presumptive bearing capacity values given in Table 1 for different types of soils and rocks. It is further recommended that for non-cohesive soils, the values should be reduced by 50% if the water table is above or near base of footing.

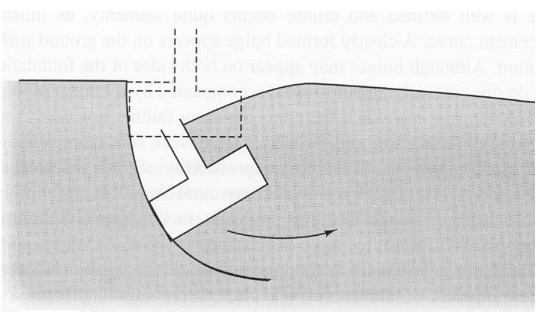
1.2 Types of failures in soil



(a) General Shear Failure

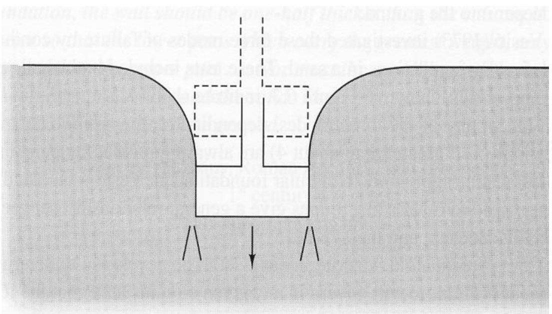
General shear

Dense soils,
Rock, NC clays
Defined failure surf.
Fast failure



Local Shear

Immediate case
Gradual failure



Local shear

Intermediate case
+/- gradual failure

Fig 2:- Types of failures in soil

1.3 Methods of analyzing the soil

The various methods of computing the bearing capacity can be listed as follows:

1. Analytical Methods
2. Plate Bearing Test
3. Penetration Test

Analytical Methods:-

This method includes:

Terzaghi's Bearing Capacity Theory

Skempton's Analysis for Cohesive soils

Meyerhof's Bearing Capacity Theory

Hansen's Bearing Capacity Theory

Vesic's Bearing Capacity Theory

IS code method

1.4 Terzaghi's Bearing Capacity Theory

There are certain assumptions in Terzaghi's Bearing Capacity Theory,

- Depth of foundation is less than or equal to its width.
- Base of the footing is rough.
- Soil above bottom of foundation has no shear strength; is only a surcharge load against the overturning load
- Surcharge upto the base of footing is considered.
- Load applied is vertical and non-eccentric.
- The soil is homogenous and isotropic.
- L/B ratio is infinite.

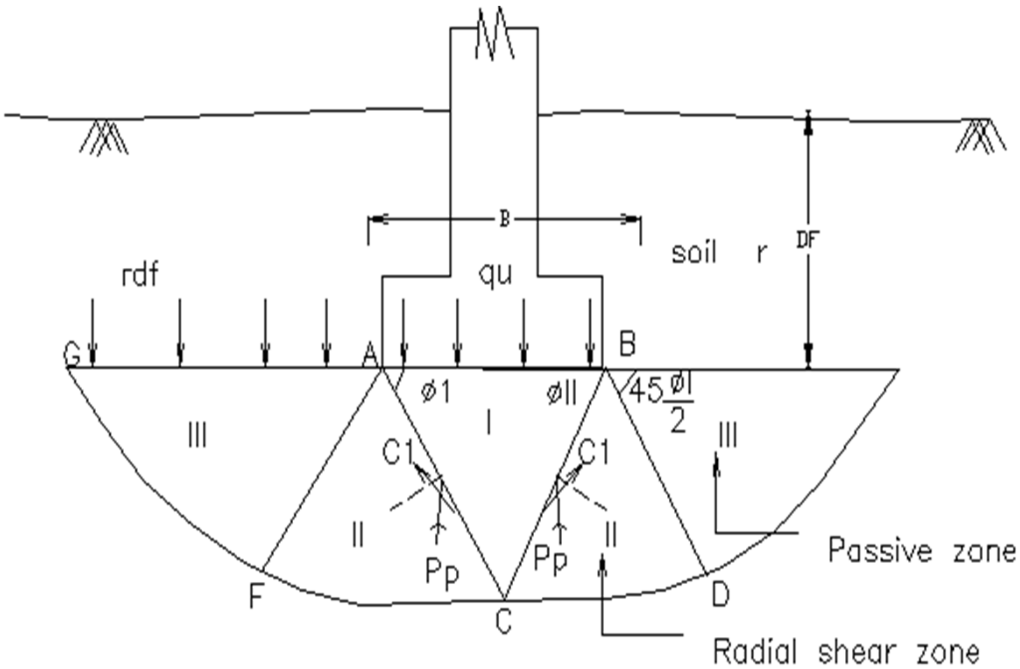


Fig.3 Terzaghi's Bearing Capacity Theory

Extensions of Terzaghi's Original Theory

(a) Local Shear Failure

Terzaghi's theory can still be used but with the reduced values of c and $\emptyset$. Reduce c and $\emptyset$ such that,

$$(c)_{new} = \frac{2}{3}c$$

$$tan(\emptyset)_{new} = \left(\frac{2}{3}\right) tan\emptyset$$

Then, Terzaghi's equation of bearing capacity becomes,

$$q_u = \left(\frac{2}{3}\right) c\, N_c + qN_q + 0.5\, \gamma B N_\gamma$$

where $N_c, N_q, and\ N_\gamma$ are read from the tables of general shear failure using,

$$(\emptyset)_{new} = tan^{-1}(0.667\, tan\emptyset)$$

Here, c and $\emptyset$ are the shear strength parameters for the soil experiencing local shear failure.

(b) Square and Circular Footings

(i) for square footing

$$q_u = 1.2\, cN_c + qN_q + 0.4\, \gamma B N_\gamma$$

(ii) for circular footing

$$q_u = 1.2\, cN_c + qN_q + 0.3\, \gamma B N_\gamma$$

For circular footings, B is taken as the diameter of the footing.

(c) Effect of the Ground Water: Rules are,

1) If the water table is below the rupture zone (depth B below base level), the water table has no effect on bearing capacity.
2) If the water table is in the rupture zone, then modify γ in Terzaghi's equation.
3) If the water table is above the base level of the footing, then modify γ and q both in Terzaghi's equation.

We modify γ in Terzaghi's equation as per 3 cases:

Case 1: $D_w \geq D_f + B \rightarrow \gamma_{new} = \gamma(no\ change)$

Case 2: $D_f < D_w < D_f + B$

Interpolate between γ'at D_f and, γ at $(D_f + B)$. The Interpolation formula is,

$$\gamma_{new} = \gamma - \gamma_w \left(1 - \frac{D_w - D_f}{B}\right)$$

Case 3: $D_w \leq D_f \rightarrow \gamma_{new} = \gamma' = \gamma - \gamma_w$

Note, γ' is the submerged or effective unit weight.

Different books, and I.S. codes, introduce the water table correction differently. But the end result is nearly the same. The following thumb rules are helpful in checking your computations.

- In granular soils, when the water table rises from below the rupture zone to the base of the footing, the bearing capacity goes down by 25%.
- In granular soils, when the water table rises from below the rupture zone to the ground surface, the bearing capacity goes down by 50%.

First thumb rule is often used in engineering practice. A designer computes the bearing capacity assuming that the water table is touching the base of the footing even though the observed position of water table is below the rupture zone.

1.5 Types of foundation

Types of Foundations

Based on the position with respect to ground level, Footings are classified into two types;

1. Shallow Foundations
2. Deep Foundations

Shallow Foundations are provided when adequate SBC is available at relatively short depth below ground level. Here, the ratio of Df / B < 1, where Df
is the depth of footing and B is the width of footing.

Deep Foundations are provided when adequate SBC is available at large depth below ground level. Here the ratio of Df / B >= 1.

Types of Shallow Foundations

The different types of shallow foundations are as follows:

- Isolated Footing
- Combined footing
- Strap Footing
- Strip Footing
- Mat/Raft Foundation

Isolated Column Footing

These are independent footings which are provided when

- SBC is generally high
- Columns are far apart
- Loads on footings are less

The isolated footings can have different cross section Some of the popular shapes of footings are;

- Square
- Rectangular
- Circular

The isolated footings essentially consists of stepped or sloping in nature. The bottom of the slab is reinforced with steel mesh to resist the two internal forces namely bending moment and shear force.

The sketch of a typical isolated footing is shown

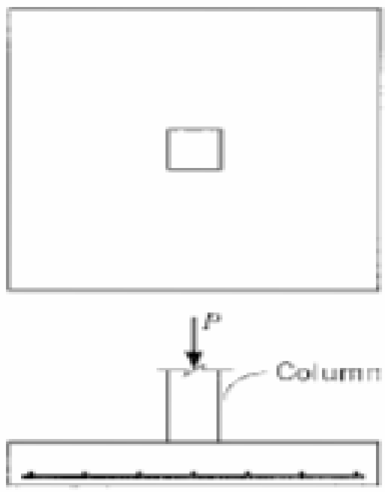


Fig 4- Plan and section of isolated footing

SECTION 2: VBA PROGRAM

2.1 Introduction

- The Windows version of Excel supports programming through Microsoft's Visual Basic for Applications (VBA), which is a dialect of Visual Basic. Programming with VBA allows spreadsheet manipulation that is awkward or impossible with standard spreadsheet techniques. Programmers may write code directly using the Visual Basic Editor (VBE), which includes a window for writing code, debugging code, and code module organization environment. The user can implement numerical methods as well as automating tasks such as formatting or data organization in VBA and guide the calculation using any desired intermediate results reported back to the spreadsheet.
- A common and easy way to generate VBA code is by using theMacro Recorder. The Macro Recorder records actions of the user and generates VBA code in the form of a macro. These actions can then be repeated automatically by running the macro. The macros can also be linked to different trigger types like keyboard shortcuts, a command button or a graphic. The actions in the macro can be executed from these trigger types or from the generic toolbar options. The VBA code of the macro can also be edited in the VBE. Certain features such as loop functions and screen prompts by their own properties, and some graphical display items, cannot be recorded, but must be entered into the VBA module directly by the programmer. Advanced users can employ user prompts to create an interactive program, or react to events such as sheets being loaded or changed.
- VBA code interacts with the spreadsheet through the Excel Object Model a vocabulary identifying spreadsheet objects, and a set of supplied functions or methods that enable reading and writing to the spreadsheet and interaction with its users (for example, through custom toolbars or command bars and message boxes). User-created VBA subroutines execute these actions and operate like macros generated using the macro recorder, but are more flexible and efficient.

The program is used to determine bearing capacity using various input data from user as per Terzaghi Analysis discussed earlier. Program is made on excel using VBA coding. Various inputs that has been taken from user are:- length, breadth, depth of footing, etc. All the parameters like depth of water table eccentricity etc. are considered. The program will give net ultimate, net safe, allowable bearing capacity plus allowable loading.

2.2 VBA Coding

The following code written in macros in excel, will calculate the results.

```
Sub BEARING()
Dim L As Single
Dim B As Single
Dim D As Single
Dim Y As Single
Dim C As Single
Dim O As Integer
Dim W As Single
Dim Ey As Single
Dim Ex As Single
Dim Qnu As Single
Dim Qns As Single
Dim Qa As Single
Dim Qs As Single
Dim fos As Single
Dim Ysat As Single
Dim Ysub As Single
Dim Yn As Single
Dim Nc As Single
Dim Nq As Single
Dim Ny As Single

L = Range("B6").Value
B = Range("B7").Value
```

```
D = Range("B8").Value
Y = Range("B9").Value
C = Range("B10").Value
O = Range("B11").Value
Ex = Range("B13").Value
Ey = Range("B14").Value
W = Range("B15").Value
fos = Range("B16").Value
Ysat = Range("B17").Value
If (B < D) Then
   MsgBox ("deep foundation")
   Else
   MsgBox ("shallow foundation")
 End If
 B = B - 2 * Ex
 L = L - 2 * Ey
 Ysub = Y - Ysat
Yn = Y - Ysat * (1 - ((W - D) / B))

Select Case O
     Case 0
       Nc = 5.7
       Nq = 1
       Ny = 0
      Case 1
       Nc = 6
       Nq = 1.1
       Ny = 0.01
       Case 2
       Nc = 6.3
       Nq = 1.22
       Ny = 0.04
       Case 3
       Nc = 6.62
```

Nq = 1.35

Ny = 0.06

Case 4

Nc = 6.97

Nq = 1.49

Ny = 0.1

Case 5

Nc = 7.34

Nq = 1.64

Ny = 0.14

Case 6

Nc = 7.73

Nq = 1.81

Ny = 0.2

Case 7

Nc = 8.15

Nq = 2

Ny = 0.27

Case 8

Nc = 8.6

Nq = 2.21

Ny = 0.35

Case 9

Nc = 9.09

Nq = 2.44

Ny = 0.44

Case 10

Nc = 9.61

Nq = 2.69

Ny = 0.56

Case 11

Nc = 10.16

Nq = 2.98

Ny = 0.69

Case 12

Nc = 10.76

Nq = 3.29

Ny = 0.85

Case 13

Nc = 11.41

Nq = 3.63

Ny = 1.04

Case 14

Nc = 12.11

Nq = 4.02

Ny = 1.26

Case 15

Nc = 12.86

Nq = 4.45

Ny = 1.52

Case 16

Nc = 13.68

Nq = 4.92

Ny = 1.82

Case 17

Nc = 14.6

Nq = 5.45

Ny = 2.18

Case 18

Nc = 15.12

Nq = 6.04

Ny = 2.6

Case 19

Nc = 16.56

Nq = 6.7

Ny = 3.07

Case 20

Nc = 17.69

Nq = 7.44

Ny = 3.64

Case 21

Nc = 18.92

Nq = 8.26

Ny = 4.31

Case 22

Nc = 20.27

Nq = 9.19

Ny = 5.09

Case 23

Nc = 21.75

Nq = 10.23

Ny = 6

Case 24

Nc = 23.36

Nq = 11.4

Ny = 7.08

Case 25

Nc = 25.13

Nq = 12.72

Ny = 8.34

Case 26

Nc = 27.09

Nq = 14.21

Ny = 9.84

Case 27

Nc = 29.24

Nq = 15.9

Ny = 11.6

Case 28

Nc = 31.61

Nq = 17.81

Ny = 13.7

Case 29

Nc = 34.24

Nq = 19.98

Ny = 16.1

Case 30

Nc = 37.16

Nq = 22.46

Ny = 19.13

Case 31

Nc = 40.41

Nq = 25.28

Ny = 2.65

Case 32

Nc = 44.04

Nq = 28.52

Ny = 26.87

Case 33

Nc = 48.09

Nq = 32.23

Ny = 31.94

Case 34

Nc = 52.64

Nq = 36.5

Ny = 38.04

Case 35

Nc = 57.75

Nq = 41.44

Ny = 45.41

Case 36

Nc = 63.53

Nq = 47.16

Ny = 54.36

Case 37

Nc = 70.01

Nq = 53.8

Ny = 65.27

Case 38

Nc = 77.5

Nq = 61.5

Ny = 78.61

Case 39

Nc = 85.97

Nq = 70.61

Ny = 95.03

Case 40

Nc = 95.66

Nq = 81.27

Ny = 115.31

Case 41

Nc = 106.81

Nq = 93.85

Ny = 140.51

Case 42

Nc = 119.67

Nq = 108.75

Ny = 172

Case 43

Nc = 134.58

Nq = 126.5

Ny = 212

Case 44

Nc = 151.95

Nq = 147.74

Ny = 261.6

Case 45

Nc = 172.28

Nq = 173.28

Ny = 325.4

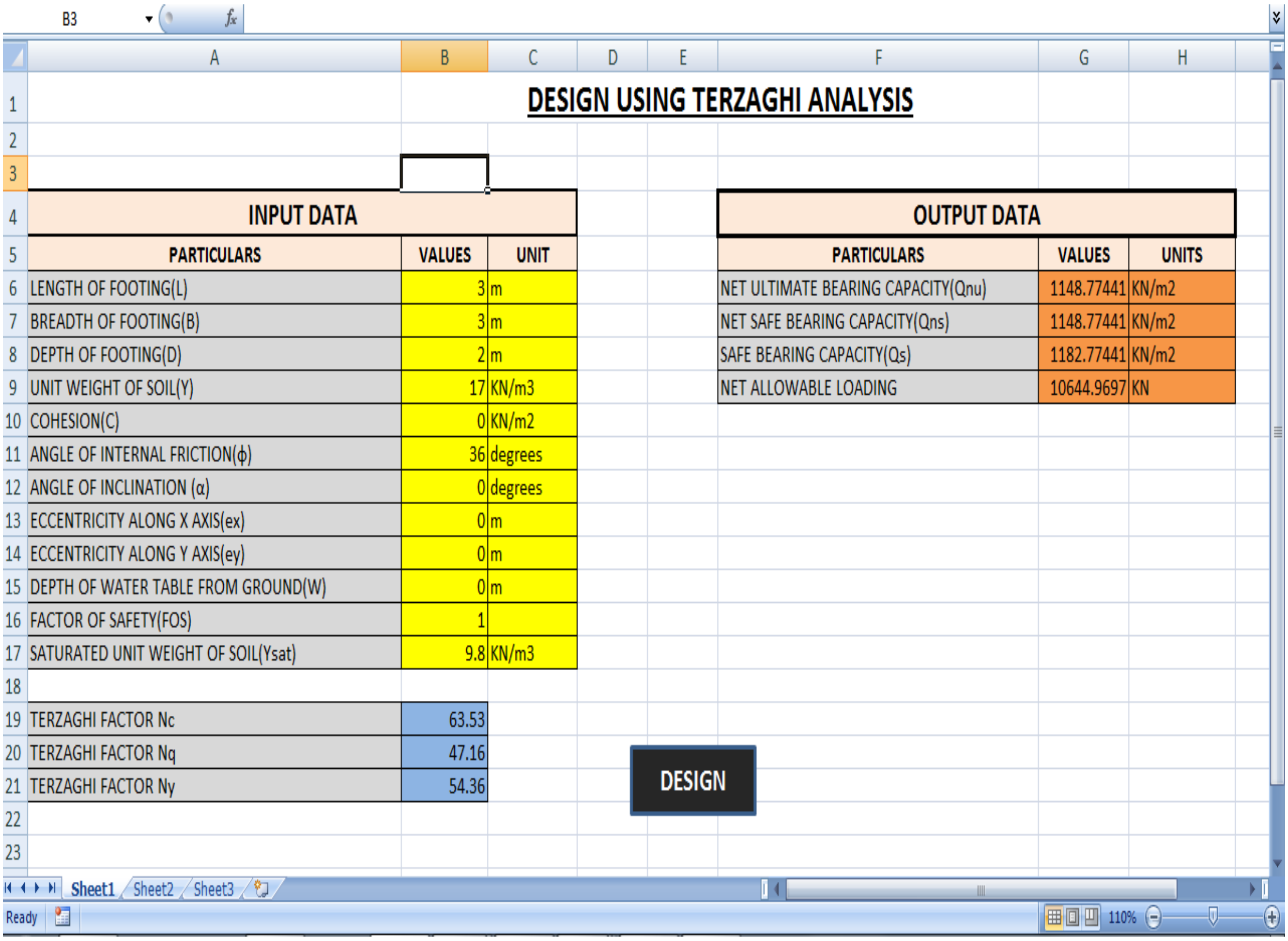
```
        Case 46
        Nc = 196.2
        Nq = 204.19
        Ny = 407.11
        Case 47
        Nc = 224.55
        Nq = 241.8
        Ny = 513
        Case 48
        Nc = 258.28
        Nq = 287.85
        Ny = 650.67
        Case 49
        Nc = 298.7
        Nq = 345
        Ny = 832
        Case 50
        Nc = 348
        Nq = 415
        Ny = 1073
End Select
q = Y * D
        If (B = L) Then
  MsgBox ("Square foundation")
      If (W < D) Then
       q = Ysub * D
       Qnu = (1.2 * C * Nc) + q * Nq + (0.4 * Ny * B * Ysub)
      ElseIf (W > D + B) Then
       Qnu = (1.2 * C * Nc) + (q * (Nq)) + (0.4 * Ny * B * Y)
      Else
        q = Yn * D
      Qnu = (1.2 * C * Nc) + (q * (Nq)) + (0.4 * Ny * B * Y)
      End If
 Else
```

```
MsgBox ("Rectangular footing")
 If (W < D) Then
  q = Ysub * D
  Qnu = (C * Nc) + q * Nq + (0.5 * Ny * B * Ysub)
 ElseIf (W > D + B) Then
  Qnu = (C * Nc) + (q * (Nq)) + (0.5 * Ny * B * Y)
 Else
   q = Yn * D
 Qnu = (C * Nc) + (q * (Nq)) + (0.5 * Ny * B * Y)
 End If
End If
Qns = Qnu / fos
Qa = (Qns + Y * D) * B * L
Qs = Qns + Y * D
Range("B19").Value = Nc
Range("B20").Value = Nq
Range("B21").Value = Ny
Range("G6").Value = Qnu
Range("G7").Value = Qns
Range("G8").Value = Qs
Range("G9").Value = Qa

End Sub
```

End sub indcates the end of the code and now the program is ready to run and execute and will give the desired result.

User interface in EXCEL



DESIGN USING TERZAGHI ANALYSIS

INPUT DATA		
PARTICULARS	VALUES	UNIT
LENGTH OF FOOTING(L)	3	m
BREADTH OF FOOTING(B)	3	m
DEPTH OF FOOTING(D)	2	m
UNIT WEIGHT OF SOIL(γ)	17	KN/m3
COHESION(C)	0	KN/m2
ANGLE OF INTERNAL FRICTION(ϕ)	36	degrees
ANGLE OF INCLINATION (α)	0	degrees
ECCENTRICITY ALONG X AXIS(ex)	0	m
ECCENTRICITY ALONG Y AXIS(ey)	0	m
DEPTH OF WATER TABLE FROM GROUND(W)	0	m
FACTOR OF SAFETY(FOS)	1	
SATURATED UNIT WEIGHT OF SOIL(γsat)	9.8	KN/m3

TERZAGHI FACTOR Nc	63.53
TERZAGHI FACTOR Nq	47.16
TERZAGHI FACTOR Nγ	54.36

OUTPUT DATA		
PARTICULARS	VALUES	UNITS
NET ULTIMATE BEARING CAPACITY(Qnu)	1148.77441	KN/m2
NET SAFE BEARING CAPACITY(Qns)	1148.77441	KN/m2
SAFE BEARING CAPACITY(Qs)	1182.77441	KN/m2
NET ALLOWABLE LOADING	10644.9697	KN

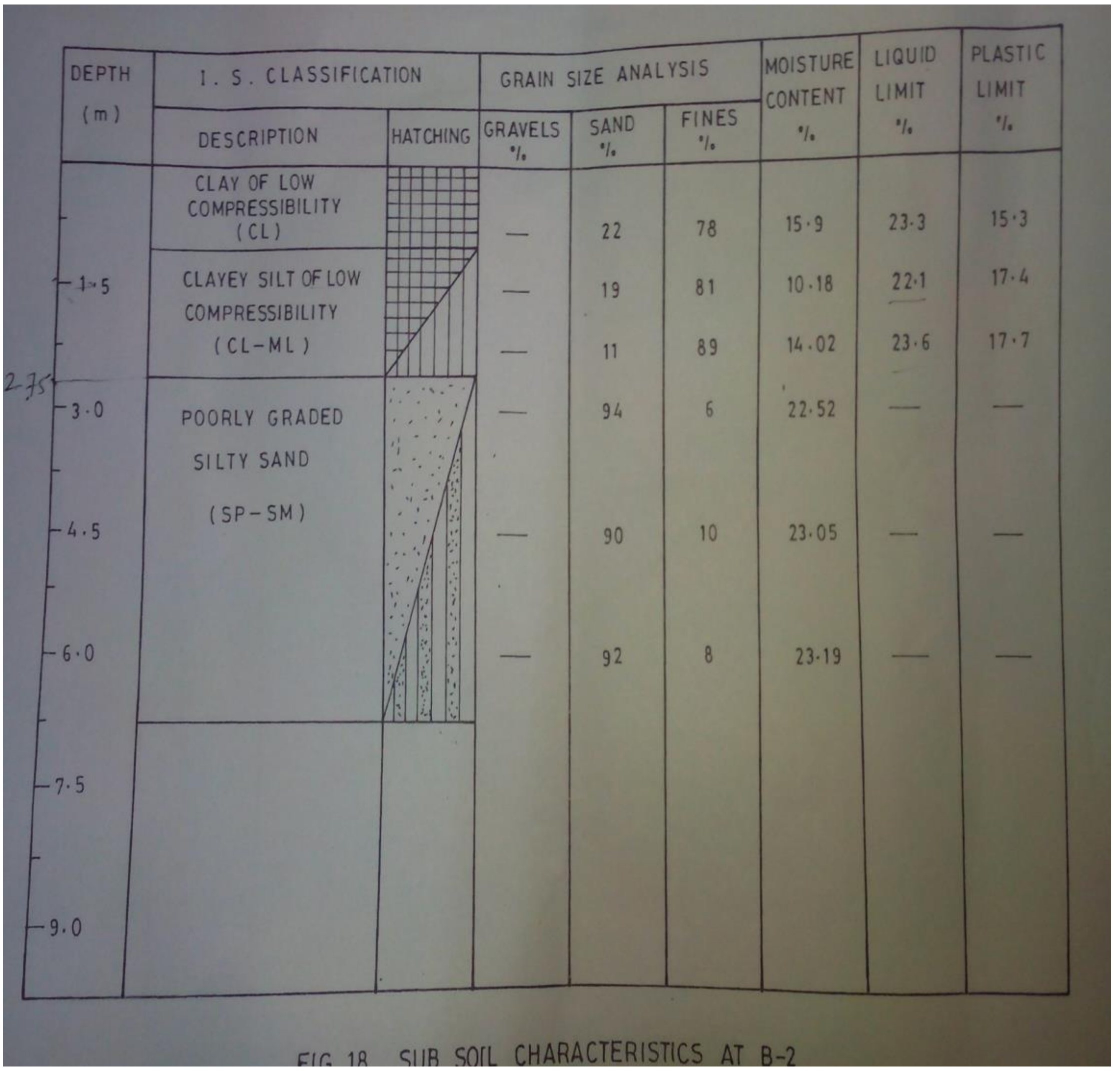
Fig.5- Output in excel

SECTION 3: ANALYSIS OF SOIL

Estimating the bearing capacity of the soil from the field data given for PLT, SPT, DCPTn tests to calculate width of footing for required load of the building, considering both settlement and shear criteria. Also various parameters such as unconfined shear and compressive strength, cohesion, angle of internal resistance etc. can be also known by conducting test and properly analyzing them.

3.1 Location And Soil Profile:

The test data analyzed here is taken from Pratap Vihar in Ghaziabad. Soil profile of the site is shown below,



DEPTH (m)	I. S. CLASSIFICATION DESCRIPTION	HATCHING	GRAIN SIZE ANALYSIS GRAVELS %	SAND %	FINES %	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %
	CLAY OF LOW COMPRESSIBILITY (CL)		—	22	78	15·9	23·3	15·3
1·5	CLAYEY SILT OF LOW COMPRESSIBILITY (CL–ML)		—	19	81	10·18	22·1	17·4
			—	11	89	14·02	23·6	17·7
2·75 / 3·0	POORLY GRADED SILTY SAND (SP–SM)		—	94	6	22·52	—	—
4·5			—	90	10	23·05	—	—
6·0			—	92	8	23·19	—	—
7·5								
9·0								

FIG 18 SUB SOIL CHARACTERISTICS AT B-2

Fig. 6- Soil profile

3.2 Standard Penetration Test

- The **standard penetration test** (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil.
- The main purpose of the test is to provide an indication of the relative density of granular deposits, such as sands and gravels from which it is virtually impossible to obtain undisturbed samples. The great merit of the test, and the main reason for its widespread use is that it is simple and inexpensive.

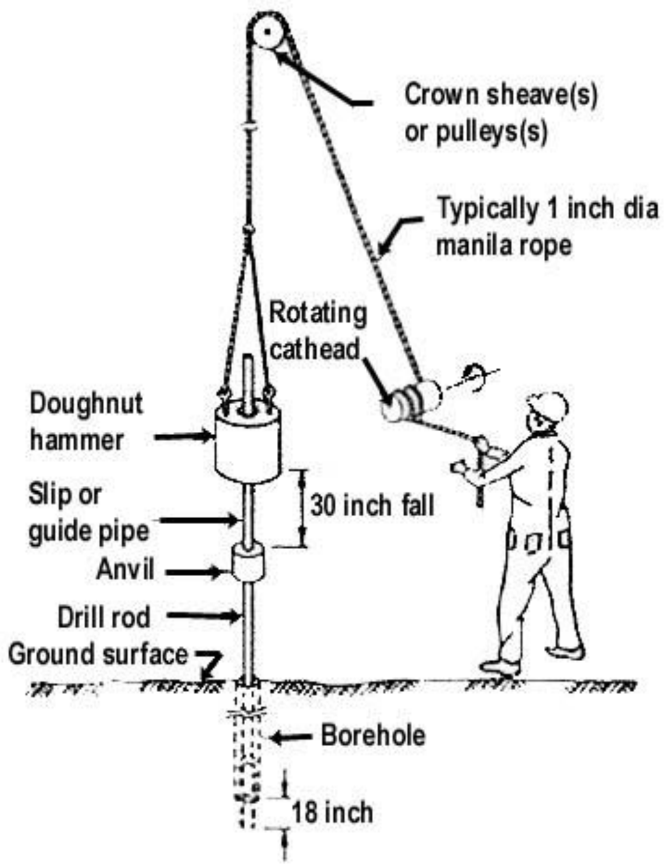


Fig 7:Setup of SPT

The following is the test data obtained when SPT test is performed on the soil,

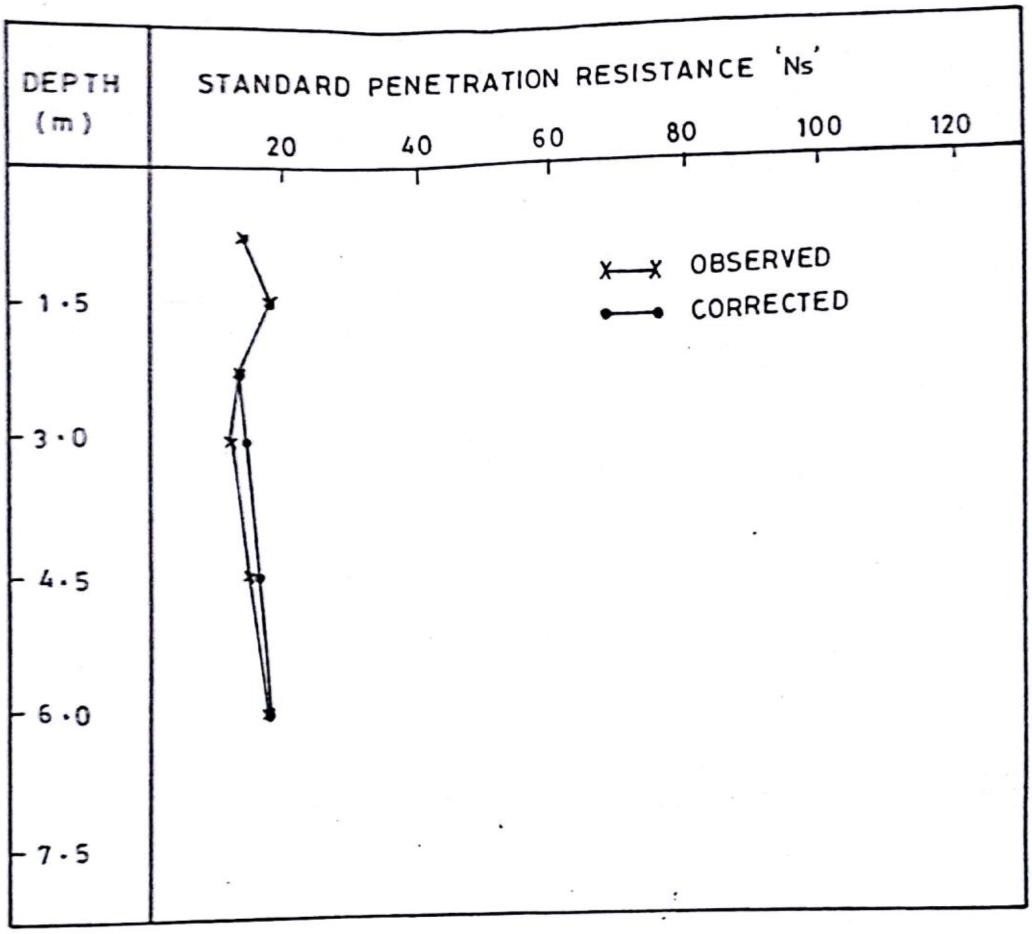


Fig.8 Standard penetration resistance vs depth

Corrections for SPT VALUE

- **Overburden pressure**

  $N' = C_N N_{spt}$

  $C_N=0.77 \log_{10} \frac{2000}{P'}$ where $P'= \gamma*D$ γ=unit wt of soil D=depth

- **Dilatancy correction**

  $N''=15+0.5(N'-15)$ for $N'>15$

  $N''=N'$ for $N'<15$

Bearing Capacity of soil at a depth of foundation 1.5 m.

Calculating SPT value for depth=1.5m using γ= 17kN/m^3 ,we get SPT value N= 18

Terzaghi's Analysis-

Shear criteria-

Net ultimate bearing capacity ,

$q_{nu}= cN_cS_c + q(N_q -1) + 0.5\gamma BN_\gamma S_\gamma$

For undrained cohesive soil-

ϕ_u=0

$N_\gamma = 0$

$N_q = 1$

$N_c = 5.14$

For strip footing

$S_c = S_\gamma = 1$

From SPT data –

Corrected SPT value = 18 for depth = 1.5m

TABLE 1: Correlation with N value for cohesive soil

N VALUE	**UNCONFINED COMPRESSIVE STRENTH(kg/CM^2)**	**CONSISTENCY**
<2	<0.25	VERY SOFT
2-4	0.25 – 0.50	SOFT
4-8	0.50 – 1.0	MEDIUM
8-16	1.0 – 2.0	STIFF
16-32	2.0 – 4.0	VERY STIFF
>32	>4.0	HARD

C_u=100 KN/m^2

q_{nu}= 706 KN/m^2

q_{ns}= 260 KN/m^2

q_{ns}=26 t/m^2

Settlement criteria-

The following shows the relation between settlement and width of footing.

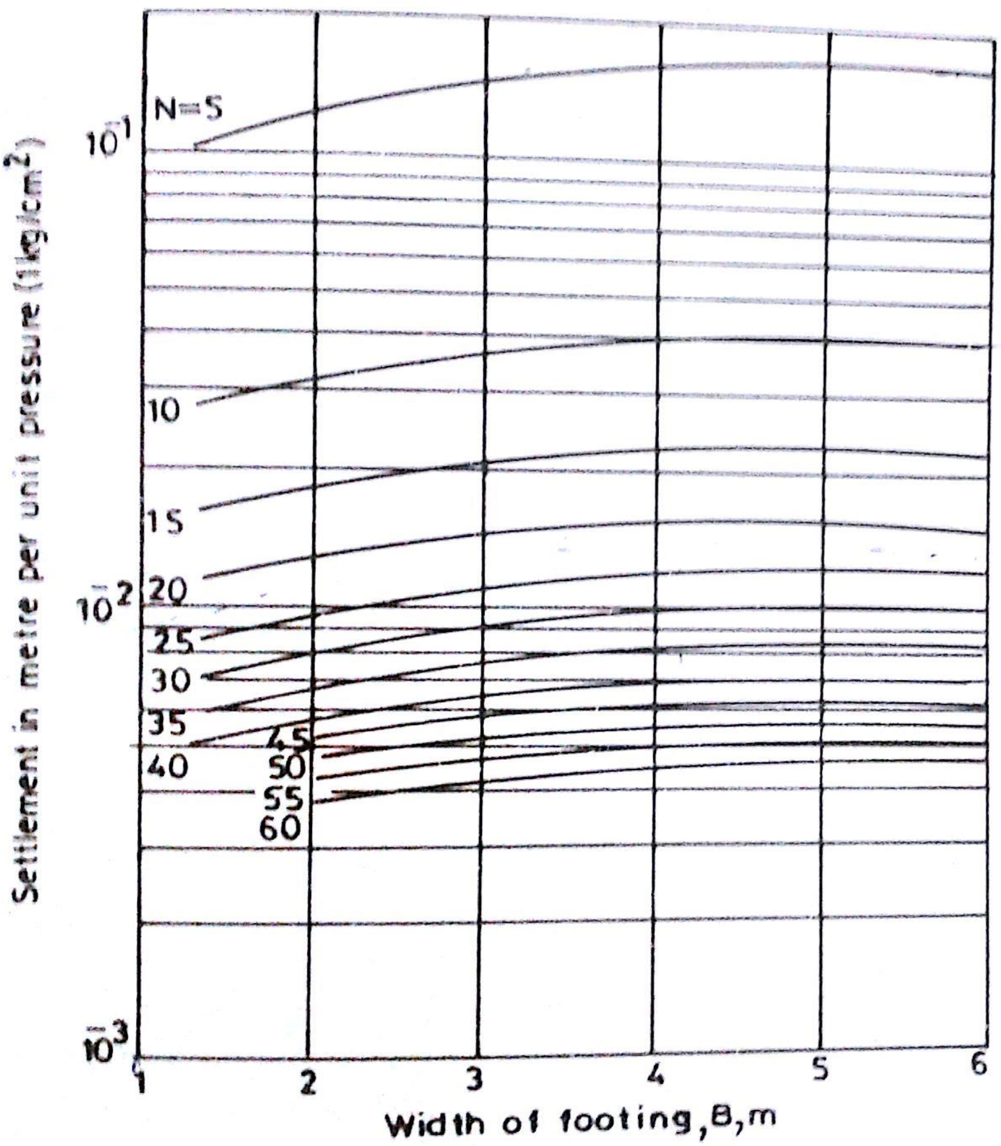


Fig. 9 Settlement vs width of footing curve

N=18, B=1.5m

Permissible settlement =50 mm

q_{np} =285.7 KN/m^2 .

q_{np} =28.57 t/m^2

q_{nabp}= min.(q_{ns}, q_{np})

q_{nabp}=20.5 t/m^2

3.3 Dynamic Cone Penetration Test

The following is the test data obtained when performed DCPT test on the soil,

- In geotechnical and foundation engineering, in-situ penetration tests have been widely used for site investigation in support of analysis and design.**Dynamic cone penetration** test is one of them.
- Its procedure is quite similar to the SPT , however a cone is used to obtain the penetration depth instead of using the split spoon sampler.

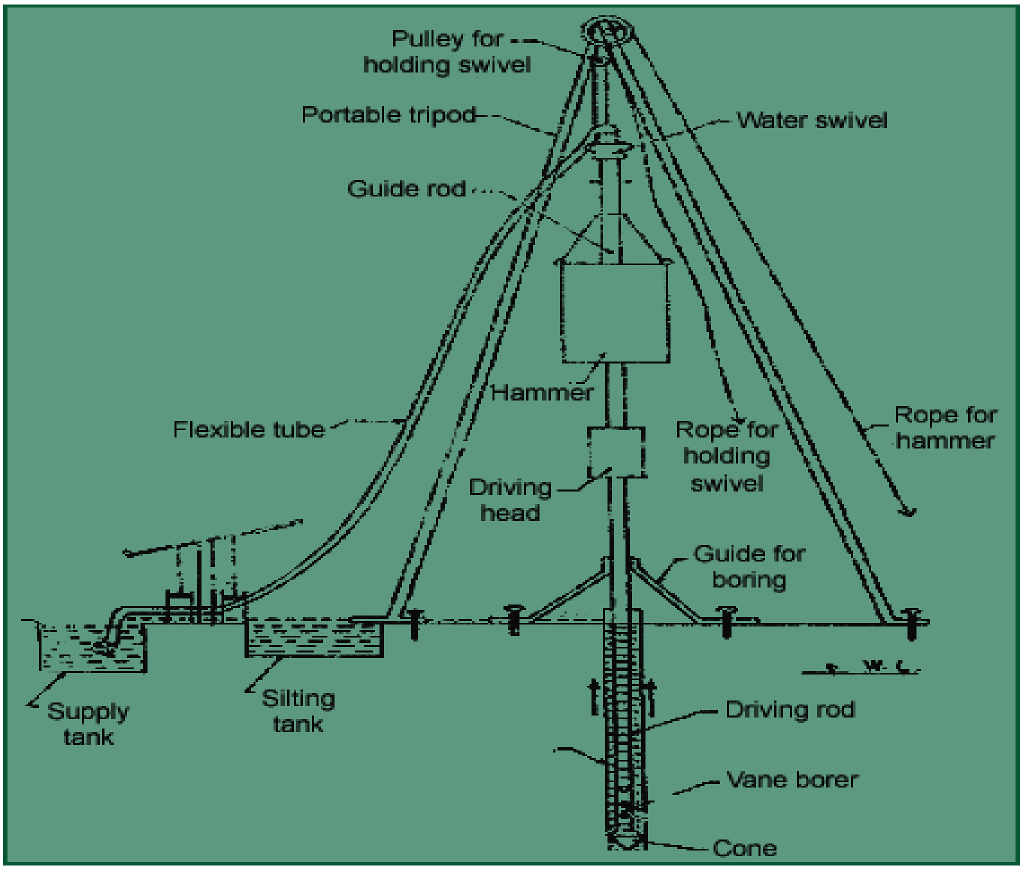


Fig 10:- Setup for DCPT test

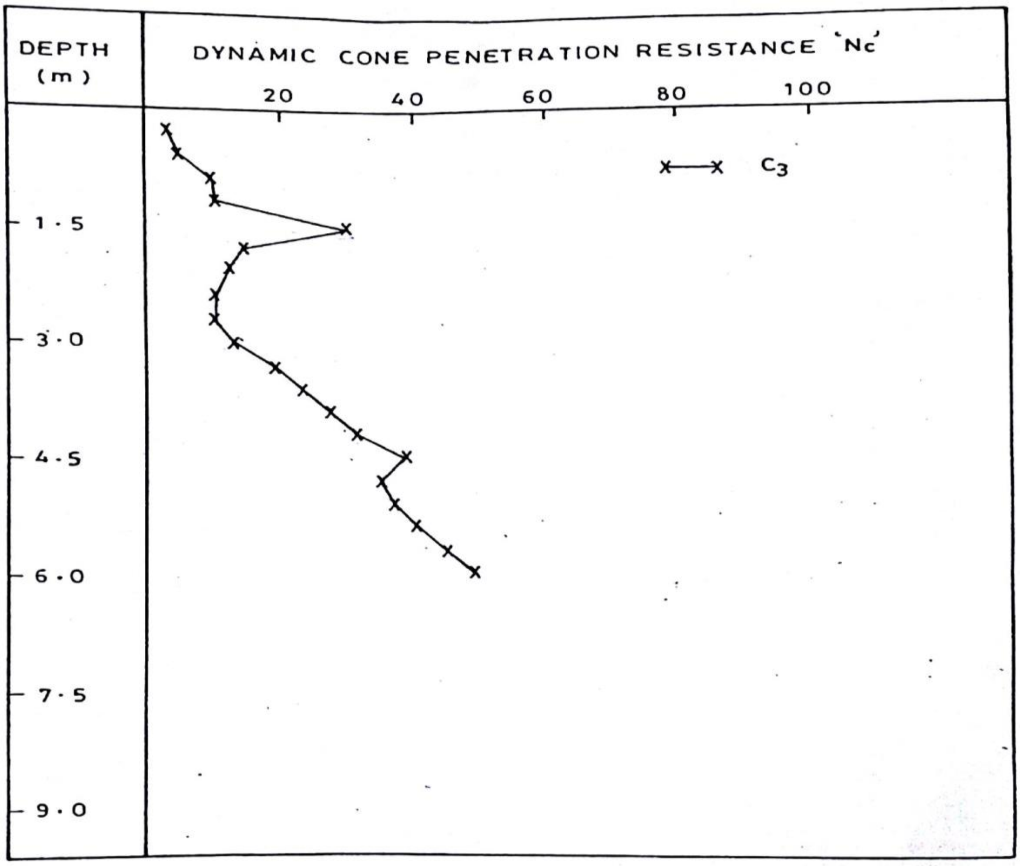


Fig. 11 Dynamic cone penetration resistance vs depth

Table 2: DCPT Test Data

$\mathbf{N_{spt} = N_{dcpt} / 1.5}$ till depth= 3m

DEPTH	N (SPT)	N (DCPT)	N (CON.,SPT)	N (CORR.)	N (DESIGN)
.5	12	5	3.3	4.96	5
1	15	10	6.6	9.5	10
1.5	18	30	20	22	22
2	13	17	11.3	15.56	16
2.5	11	12	8	11.6	12
3	14	14	9.3	13.46	14
3.5	15	22	14.6	18	18

4	**15**	**30**	**20**	**22**	**22**
4.5	**16**	**37**	**24.6**	**25.52**	**26**
5	**16**	**38**	**25.3**	**25.86**	**26**
5.5	**17**	**42**	**28**	**27.8**	**28**
6	**18**	**48**	**32**	**30.7**	**31**

Shear criteria:

From here,

$N_{corrected} = 22$ for depth=1.5m

C_u=100 KN/m^2 (Table 4)

$q_{nu} = cN_cS_c + q(N_q - 1) + 0.5\gamma BN_\gamma S_\gamma$

For undrained cohesive soil-

$\phi_u = 0$

$N_\gamma = 0$

$N_q = 1$

$N_c = 5.14$

q_{nu}= 614 KN/m^2

q_{ns}= 215 KN/m^2

q_{ns}=21.5 t/m^2

Settlement criteria

N=22, B=1.5m

Permissible settlement =50 mm

q_{np} =333.3KN/m^2 .

q_{np} =33.33 t/m^2 .

q_{nabp}= min.(q_{ns}, q_{np})

q_{nabp}=21.5 t/m^2

Now the following curve is plotted.

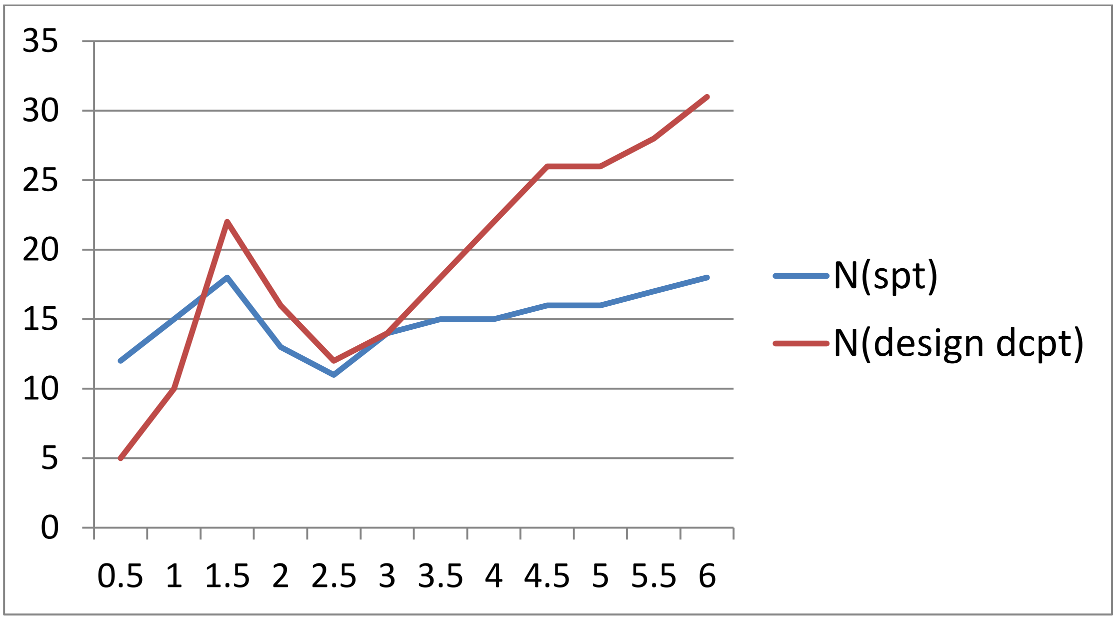


Fig. 12 N values vs depth

3.4 Plate Load Test.

- **PLATE LOAD TEST** is used to find the bearing capacity and settlement of the soil and settlement .

- **DESCRIPTION**:: A Test Plate(Square or circular in shape) are used in PLT test. The plate is placed at proposed level of the foundation and is subjected to incremental loading. Settlement at each increment of the loading is measured and a load settlement curve is plotted. Bearing capacities and settlement are determined from the load settlement curves.

The following is the test data obtained when performed DCPT test on the soil,

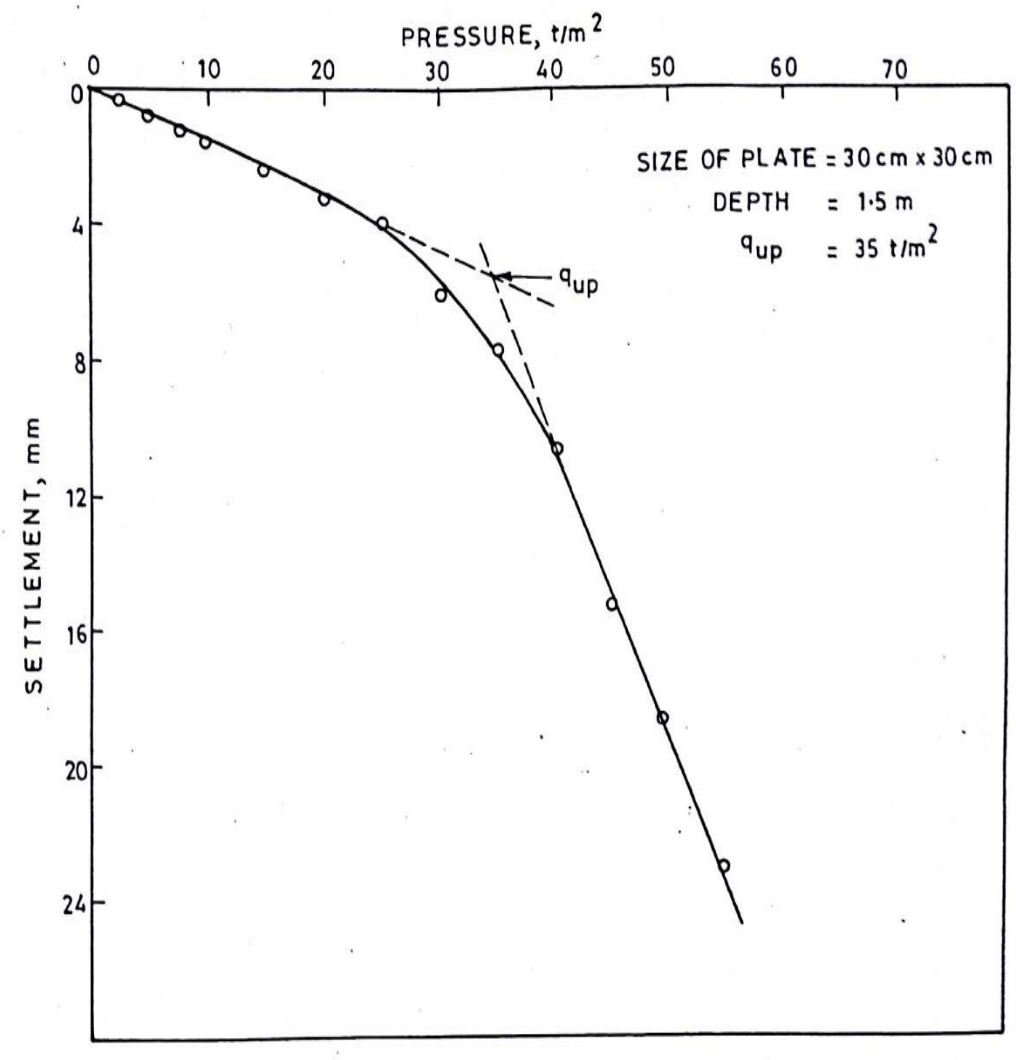


Fig.13 Settlement curve

Plotting graph between load and settlement and applying tangent method we get.

Shear criteria

(q_{nu}) plate =35 t/m^2

For clays;

(q_{nu}) plate = (q_{nu}) footing

(q_{nu}) footing = 35 t/m^2

q_{ns}= 14 t/m^2

q_{nabp}= 14 t/m^2

.

Settlement criteria

q_{np}=38 t/m^2

Sf/Sp = Bf/Bp

Sp= (0.05×0.3)/1.5

=0.1m

=10mm

Corresponding to 10 mm penetration net allowable bearing pressure = 25.3N/mm^2.

Minimum net allowable bearing pressure from SPT,DCPT and PLT test data from above is,

q_{nabp}= 14 t/m^2.

SECTION 4: FOUNDATION DESIGN OF A STORAGE UNIT

4.1 Assumptions:

- Live load on structure =4 KN/m^2
- Dead Load on structure =1.5 KN/m^2
- Thickness of masonry wall =230 mm
- Diameter of bar used = 8 mm
- Length of footing = 8 m
- Depth of footing =1.5 m
- Height of superstructure = 5 m
- Width of superstructure = 3 m
- Provided a Strip footing
- Fe 415 grade HYSD Reinforcement
- M-25 grade Concrete

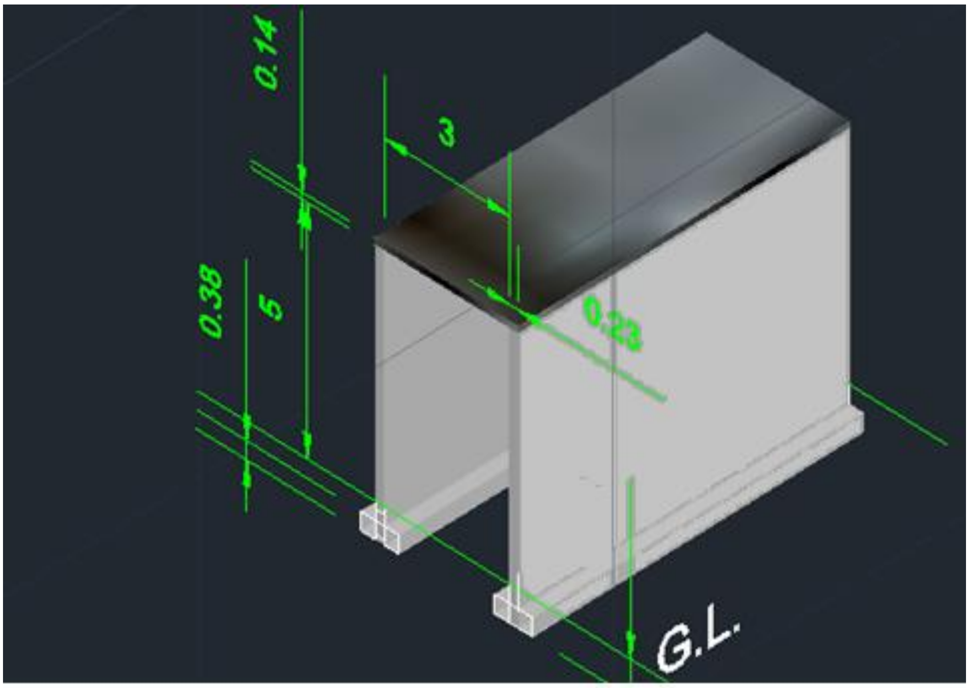


Fig. 14 Isometric view of storage unit

4.2 Calculations:

Span between centers of bearing = $3000 + \frac{230}{2} + \frac{230}{2}$ = 3.23 m

Assuming P_t = 0.3%, Modification factor = 1.4 and slab is simply supported.

$$\frac{\text{Span}}{\text{Effective Depth}} = 20 \times \text{M.F.}$$

$$d_{eff} = \frac{230}{20\times1.4}$$

$$= 116.35 \text{ mm}$$

d_{eff} provided = 117 mm

Provide 8mm diameter bars at a clear cover of 15mm.

Effective cover = 15+8/2 = 19mm

Overall depth required = 117+20 = 137 mm

Provided overall depth = 140mm

Now, d_{eff} = 140-19 =120mm

Dead load due to slab = 25×0.140 = 3500 N/m^2

Dead load due to finish partition = 1500 N/m^2

Total load = D.L. + L.L.

= (3500+1500)+4000

=9000 N/m^2

Design load = Factored load

= 1.5×9000

= 13500 N/m^2

Effective span = min (Distance between centers of bearing, clear span + effective depth)

= min (3.23, 3+0.121)

=3.121m

Consider 1m wide strip of slab,

Factored moment $M_u = \frac{w_u \times l^2}{8}$

$= \frac{13300 \times 3.121^2}{8}$

= 16337.33 N-m

Moment of resistance $M_{u,limit}$ for Fe 415

$M_{u,limit} = 0.138 f_{ck} b d^2$

$M_u = M_{u,limit}$

$16337.33\times10^3 = 0.138\times25\times1000\times d^2_{eff}$

d_{eff} = 63 mm

d_{eff} provided = 121 mm (safe as d_{eff} provided > d_{eff})

M_u/bd^2 =1.1226

Percentage of steel required, $P_t = 50 \times \left[\frac{1-\sqrt{1-\frac{4.6\times M_u}{f_{ck}bd^2}}}{f_{y/f_{ck}}}\right]$

=0.33%

$\frac{A_{st}}{b\times d_{eff}}\times100 = 0.33\%$

A_{st} =400 mm^2

Spacing of 8 mm dia. bars = $\frac{1000\times\frac{\pi}{4}\times82}{400}$

=125.21 mm

Spacing of bars = 125 mm c/c.

Number of bars = $\frac{1000}{\text{spacing}} \approx 8$

Actual area of steel provided, $A_{st,\,provided} = \frac{\pi}{4} \times 8^2 \times 8$ = 402.12 mm^2

Actual percentage of steel provided = $\frac{402.12}{1000\times121}\times100$

$= 0.332\%$

Modification factor corresponding to 0.332% steel (Fe 415) = 1.39 (by interpolation)

⇨ $\frac{\text{span}}{d_{eff}\times M.F.} = 20$

⇨ $d_{eff} = 111.83$ mm

d_{eff} provided = 121mm

Hence the design is safe in serviceability conditions.

4.3 Check For Shear:

Nominal shear stress, $\tau_v = \frac{V_u}{bd}$

$V_u = \frac{1360\times3\times1}{2}$

$= 20280$ N

$\tau_v = \frac{20280}{1000\times121} = 0.166$ N/ mm^2

Percentage of steel available near support = 0.332/2

$= 0.166\%$

Design shearing strength of concrete corresponding to 0.16% of steel, $\tau_c = 0.3\text{N/mm}^2$

For 140 mm thick slab K = 1.30

$\tau_c' = K\,\tau_c$

$= 1.30\times0.3$

$= 0.39\text{N/mm}^2$

$\tau_v < \tau_c'$

So the design is safe against shear.

4.4 Distribution Steel:

A_{st} of distribution bar ≥ 0.12% of gross area for Fe 415

A_{st} of distribution bar = 0.0012×1000×140

$= 168\text{ mm}^2$

Spacing of 8mm dia. bars = $\frac{50\times1000}{168}$

=297.6mm

Provide 8 mm dia. bars @ 290mm c/c spacing.

4.5 Design of Main Bars:

No. of bars = 8000/125 ≈ 64

No. of equally spaced bars = (64-1) = 63

Total spacing between first and last bars = 63×125 = 7870 KN/m^3

Development length = 40 × ø = 40 × 8 ≈ 315 mm

Length = 3460-(15+15) = 3430mm

Volume of steel used in main reinforcement = Area × Length

V_{st} main bars = 11034567.71mm^3

Weight = Volume × γ_{steel}

= 880.2 N

4.6 Load Calculations:

Total load (Weight of slab) = 9000×8×3.46 866.169

= 250363.55 N or 250.36 KN

Load distribution:-

Weight of slab on one masonry wall = 125.81 KN

Load per unit length of masonry wall = 15.64 KN/m

Unit length volume of masonry wall = 0.23×5×1 = 1.15 m^3 per m length of the wall.

Unit weight of bricks = 16 KN/m^3

Weight of bricks = 16×1.15 = 18.4 KN/m

Total load on foundation, Q = 18.4 + 15.64 = 34.04 KN/m

4.7 Design of RCC Footing

Minimum net allowable bearing pressure from SPT,DCPT and PLT test data from above is, q_{nabp}= 14 t/m^2

Width of the foundation-

q_{nabp} = Q/A

Q = 34 KN/m

Area = B×1 m^2

B (Calcuated) = 0.25 m

Provided width B= 1 m

Net upward soil pressure = $\frac{34}{0.25\times1}$ = 134 KN/m^2

Factored upward soil pressure per meter length, P_o = 1.5×134×1 = 201 KN/m

The critical section for moment is at the face of the wall.

$M_{u,\,lim} = \frac{P_0}{8} \times (B-b)^2$

$= \frac{51}{8} \times (1-0.23)^2$

= 11.8 KN-m

$M_{u,\,lim} = M_u$

$11.8 \times 10^6 = 0.138\ f_{ck}bd^2$

$= 0.138\times25\times1000\times d^{2+}$

d = 55 mm

Provided depth = 1.4 × 55 = 77 mm

But according to IS 456, minimum footing depth should be 150 mm.

Actual depth provided, D = 150mm

Take clear cover of 50mm and dia. of bar 8mm.

d_{eff} = 150-50-4 = 96mm

$M_u/bd^2 = \frac{3.8167\times10^6}{1000\times96^2} = 0.4319$

Percentage of steel required, $P_t = 50 \times \left[\frac{1-\sqrt{1-\frac{4.6\times M_u}{f_{ck}bd^2}}}{f_y/f_{ck}}\right]$

=0.12%

A_{st} = 0.0012 × 1000 × 150 = 180 mm^2

No. of bars provided of 8mm dia. = 4

Spacing = 288 mm c/c. (spacing should be less than 2 times depth=300mm => safe)

4.8 Consolidation Settlement in Clay-

$$\Delta H = \frac{C_c}{1+e_0} H \log_{10} \frac{\sigma_v + \Delta\sigma_v}{\sigma_v}$$

C_c= Compression index =.009(w_L-10)

$\boldsymbol{\sigma}_v$ = Effective initial overburden pressure .

$\Delta\sigma_v$=Vertical stress increment due to footing load.

H=Thickness of clay layer (m)

Assuming Load dispersion at 30˚ from the vertical.

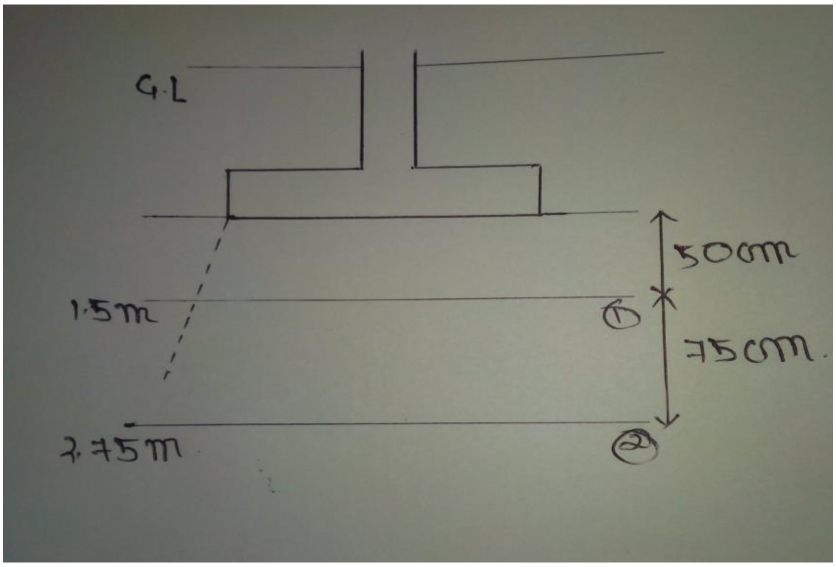


Fig.15: Pressure distribution through soil

Table 3: Settlement calculation

LAYER	DEPTH OF LAYER	THICKNESS OF THE LAYER(m)	σ_v KN/m^2	$\Delta\sigma_v$ KN/m^2	C_c	e_0	SETTLEMENT ΔH (mm)
1	1.5	0.75	32.25	24.02	0.122	0.6	10.70
2	2.75	0.5	44	15.31	0.126	0.645	6.52

TOTAL SETTLEMENT=18mm

Consolidation Settlement in clay layer = 18 mm which is less than allowable settlement in clays that is 50 mm. Hence the design is safe.

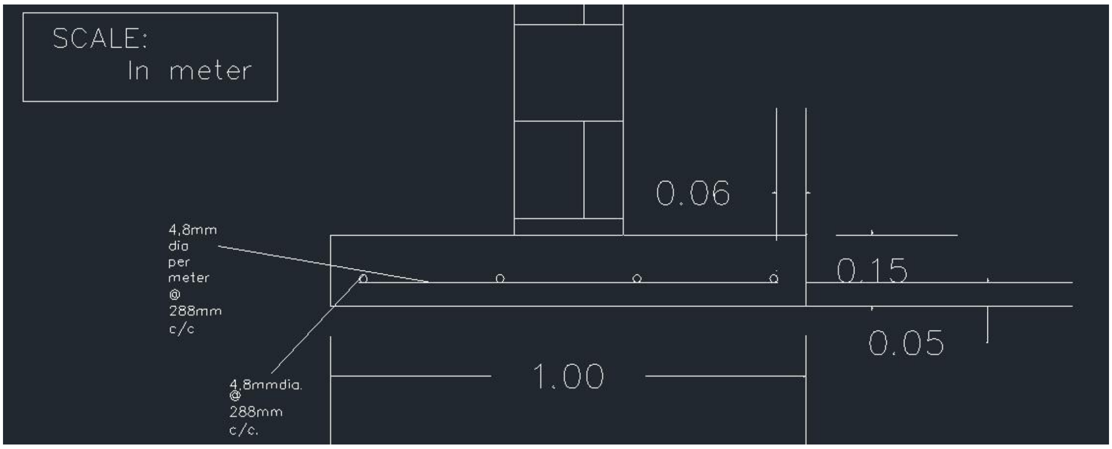


Fig. 16 Cross-section of the foundation

SECTION 5: FOUNDATION DESIGN OF A SHOPPING COMPLEX (Isolated foundation)

5.1 Specifications for Design as per IS 456 : 2000

The important guidelines given in IS 456 : 2000 for the design of isolated footings are as follows:

General

Footings shall be designed to sustain the applied loads, moments and forces and the induced reactions and to ensure that any settlement which may occur shall be as nearly uniform as possible, and the safe bearing capacity of the soil is not exceeded (see IS 1904).

In sloped or stepped footings the effective cross-section in compression shall be limited by the area above the neutral plane, and the angle of slope or depth and location of steps shall be such that the design requirements are satisfied at every section.

Sloped and stepped footings that are designed as a unit shall be constructed to assure action as a unit.

Moments and Forces

In the case of footings on piles, computation for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the centre of the pile.

For the purpose of computing stresses in footings which support a round or octagonal concrete column or pedestal, the face of the column or pedestal shall be taken as the side of a square inscribed within the perimeter of the round or octagonal column or pedestal.

Bending Moment

The bending moment at any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of the said plane.

Tensile Reinforcement

The total tensile reinforcement at any section shall provide a moment of resistance at least equal to the bending moment on the section.

Total tensile reinforcement shall be distributed across the corresponding resisting section as given below:

a) In one-way reinforced footing, the-reinforcement extending in each direction shall be distributed uniformly across the full width of the footing.

b) In two-way reinforced square footing, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing.

5.2 Provided data

- Site location is Pratap vihar in Ghaziabad, Uttar Pradesh.
- Soil properties are determined from Tests performed earlier on the same soil profile
- Isolated foundation has been designed for each column
- R.C.C Design is done for each column according to Indian standard codes.
- Two storeyed shopping complex(8 shops)
- Floor area 4m X 8.5m of each shop.
- Height of each shop is 3m.
- Live load is 4KPa.
- Dead load due to finish, partition etc is 1.5 KPa.
- Fe 415 grade HYSD reinforcement and M25 grade concrete.
- Thickness of slab is 150mm.
- Column size is 300mm x 300mm..
- Beam size is 230mm x 250mm.

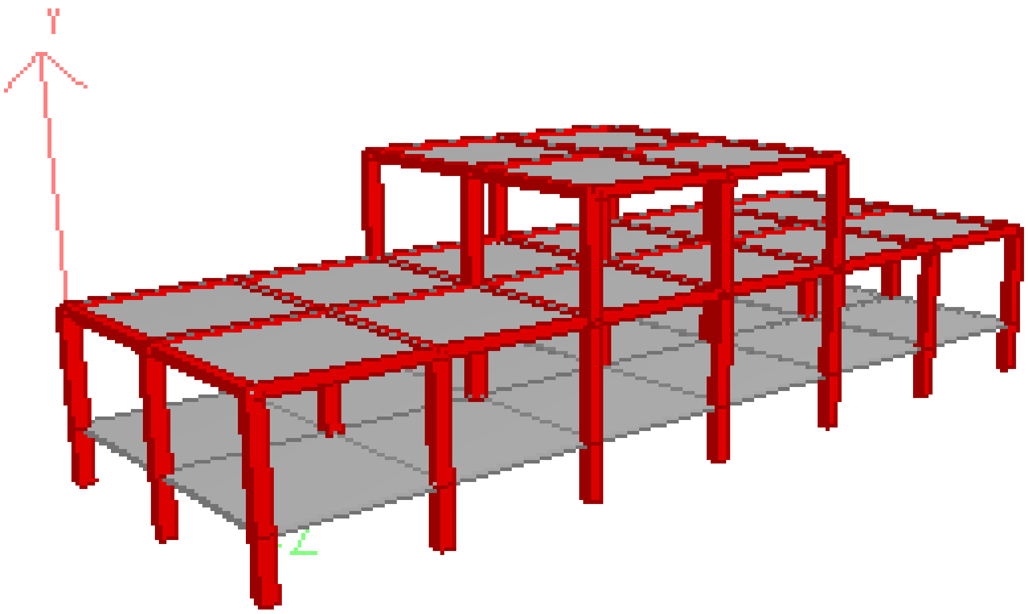


Fig 17:- Proposed structure

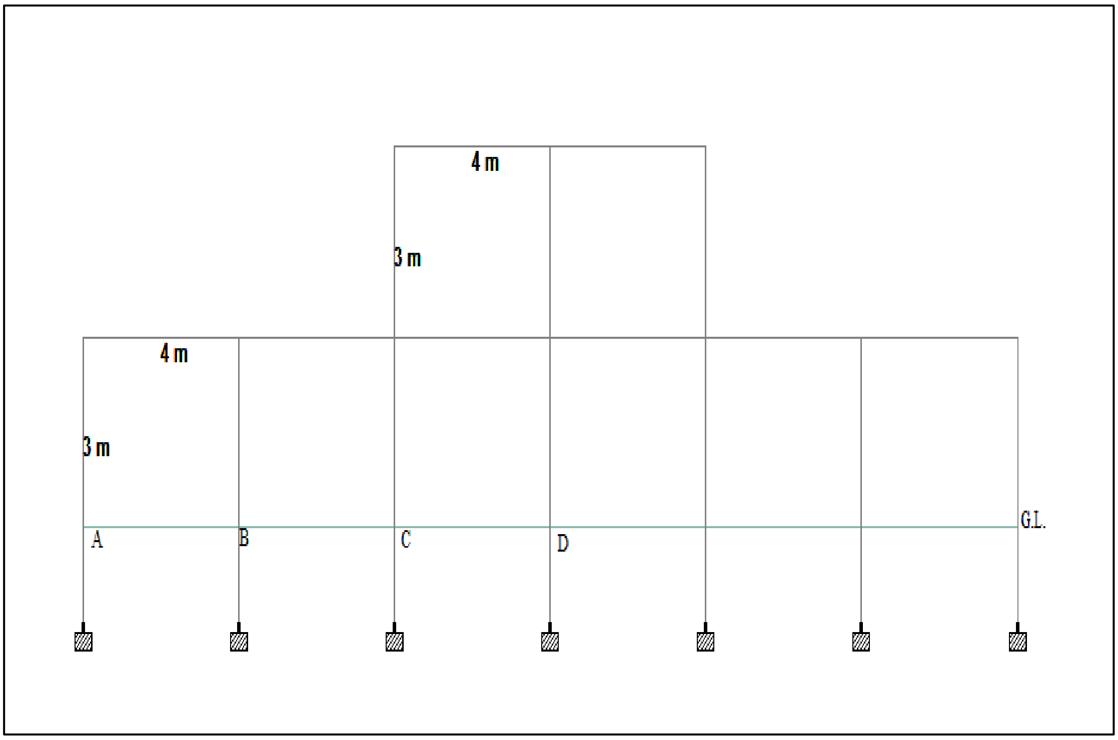


Fig 18:- Front view of shopping complex

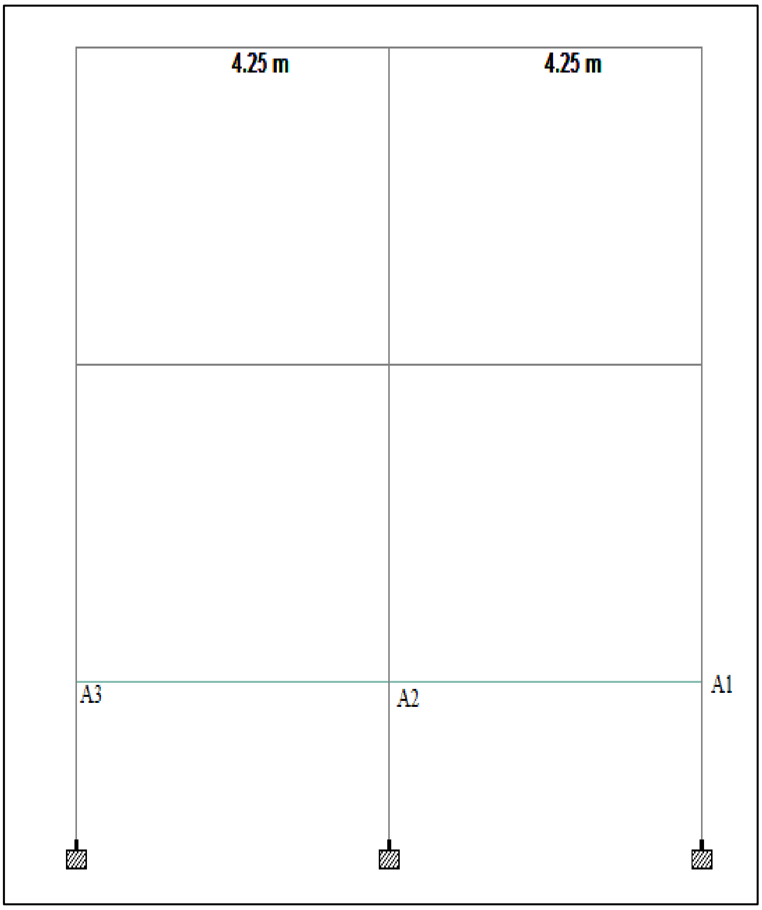


Fig 19:- Side view of shopping complex

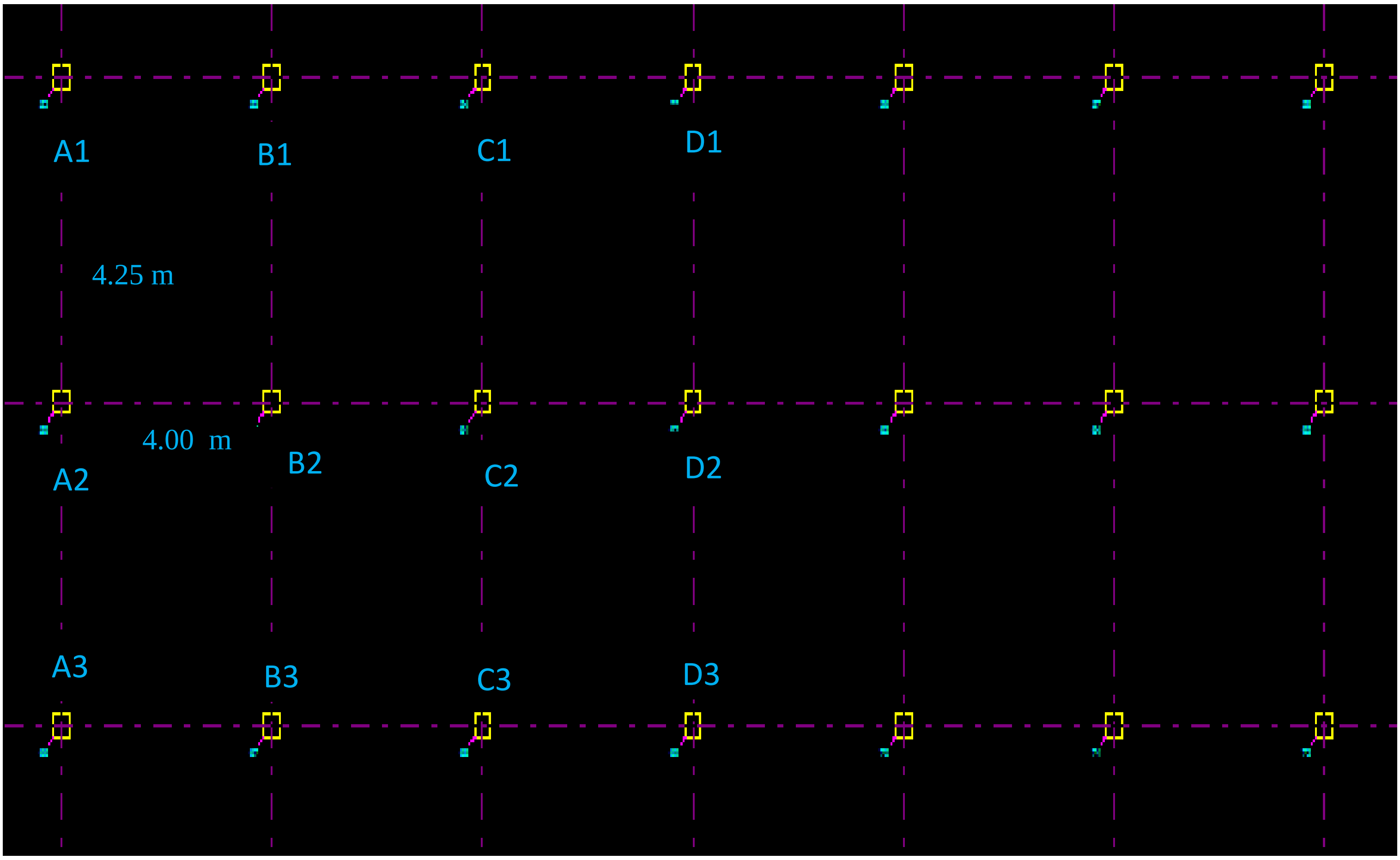


Fig20 – Top view of Foundations

5.3 Foundation Load Calculation

Axial loads on Foundation A1-

Dead weight of slab= (4.25/2 × 2 × .15) × Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN
= 15.9 kN

Dead weight of Beam = 0.23×0.25×(4.25/2 + 2) × 25
= 5.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete)
= 7.2 kN

Dead load due to finish partition = 4.25/2 ×2×1.5
= 6.375 kN

Live load experienced = 4.25/2 ×2×4
= 17 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead
Weight of column + Dead load due to finish + Live load
= 2×15.9 + 5.92 + 7.2 + 6.37 + 17
= 67 kN

Axial loads on Foundation A2-

Dead weight of slab= (4.25/2 × 2 × .15) × 2×Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN
= 31.9 kN

Dead weight of Beam = 0.23×0.25×(4.25 + 2) × 25
= 8.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete)
= 7.2 kN

Dead load due to finish partition = 4.25 ×2×1.5
= 12.75 kN

Live load experienced = 4.25 ×2×4
= 34 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead
Weight of column + Dead load due to finish + Live load
= 2×31.9 + 8.92 + 7.2 + 12.77 + 34
= 130 kN

Axial loads on Foundation A3-

Dead weight of slab= (4.25/2 × 2 × .15) × Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN
= 15.9 kN

Dead weight of Beam = 0.23×0.25×(4.25/2 + 2) × 25
= 5.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete)
= 7.2 kN

Dead load due to finish partition = 4.25/2 ×2×1.5
= 6.375 kN

Live load experienced = 4.25/2 ×2×4
= 17 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×15.9 + 5.92 + 7.2 + 6.37 + 17
= 67 kN

Axial loads on Foundation B1-

Dead weight of slab= (4.25/2 × 2 × .15) × 2×Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN
= 31.9 kN

Dead weight of Beam = 0.23×0.25×(4.25 + 2) × 25
= 8.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete)
= 7.2 kN

Dead load due to finish partition = 4.25 ×2×1.5
= 12.75 kN

Live load experienced = 4.25 ×2×4
= 34 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×31.9 + 8.92 + 7.2 + 12.77 + 34
= 131.7 kN

Axial loads on Foundation B2-

Dead weight of slab= (4.25 × 4 × .15) ×Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN
= 63.9 kN

Dead weight of Beam = 0.23×0.25×(4.25 + 4) × 25
= 11.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete)
= 7.2 kN

Dead load due to finish partition = 4.25 ×4×1.5
= 25.5 kN

Live load experienced = 4.25 ×4×4
= 68 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×63.9 +11.92 + 7.2 + 25.5 + 68
= 240 kN

Axial loads on Foundation B3-

Dead weight of slab= (4.25/2 × 2 × .15) × 2×Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN
= 31.9 kN

Dead weight of Beam = 0.23×0.25×(4.25 + 2) × 25
= 8.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete)
= 7.2 kN

Dead load due to finish partition = 4.25 ×2×1.5
= 12.75 kN

Live load experienced = 4.25 ×2×4
= 34 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×31.9 + 8.92 + 7.2 + 12.77 + 34
= 131.7 kN

Axial loads on Foundation C1-

Dead weight of slab= (4.25 × 2 × .15) ×Unit wt of concrete + (4.25/2 ×2×0.15) × Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN + (4.25/2 ×2×0.15) ×25
= 47.9 kN

Dead weight of Beam = 0.23 × 0.25 × (4.25 + 2) × 25 + 0.23 × 0.25 × (4.25/2 +2) × 25
= 15.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete) ×2
= 14.4 kN

Dead load due to finish partition = 4.25 ×2×1.5 + 4.25/2 × 2 × 1.5
= 19.75 kN

Live load experienced = 4.25 × 2× 4 + 4.25/2 × 2 × 1.5
= 51 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×47.9 + 15.92 + 14.4 + 19.75 + 51
= 185 kN

Axial loads on Foundation C2-

Dead weight of slab= (4.25 × 2 × .15) × 25 kN + (4.25 × 4 ×0.15) ×25
= 95 kN

Dead weight of Beam = 0.23 × 0.25 × (4.25 + 2) × 25 + 0.23 × 0.25 × (4.25 + 4)
= 19.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete) ×2
= 14.4 kN

Dead load due to finish partition = 4.25 ×2×1.5 + 4.25 × 4 × 1.5
= 38.75 kN

Live load experienced = 4.25 × 2× 4 + 4.25 × 4 × 1.5
= 102 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2 × 95 + 19.92 + 14.4 + 38.75 + 102
= 335 kN

Axial loads on Foundation C3-

Dead weight of slab= (4.25 × 2 × .15) ×Unit wt of concrete + (4.25/2 ×2×0.15) × Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN + (4.25/2 ×2×0.15) ×25
= 47.9 kN

Dead weight of Beam = 0.23 × 0.25 × (4.25 + 2) × 25 + 0.23 × 0.25 × (4.25/2 +2) × 25
= 15.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete) ×2
= 14.4 kN

Dead load due to finish partition = 4.25 ×2×1.5 + 4.25/2 × 2 × 1.5
= 19.75 kN

Live load experienced = 4.25 × 2× 4 + 4.25/2 × 2 × 1.5
= 51 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×47.9 + 15.92 + 14.4 + 19.75 + 51
= 185 kN

Axial loads on Foundation D1-

Dead weight of slab= (4.25 × 2 × .15) ×Unit wt of concrete + (4.25 ×2×0.15) × Unit wt of concrete
= (4.25 × 2 × .15) × 25 kN + (4.25 ×2×0.15) ×25
= 127.5 kN

Dead weight of Beam = 0.23× 0.25 × (4.25 + 4) × 25 + 0.23 × 0.25 × (4.25 + 4) × 25
= 23.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete) ×2
= 14.4 kN

Dead load due to finish partition = 4.25 × 4 ×1.5 + 4.25 × 4 × 1.5
= 51 kN

Live load experienced = 4.25 × 4 × 4 + 4.25 × 4 × 1.5
= 136 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead
Weight of column + Dead load due to finish + Live load
= 2× 127.5 + 23.92 + 14.4 + 51 + 136
= 412 kN

Axial loads on Foundation C3-

Dead weight of slab= (4.25 × 2 × .15) ×Unit wt of concrete + (4.25/2 ×2×0.15)
× Unit wt of concrete
= (4.25/2 × 2 × .15) × 25 kN + (4.25/2 ×2×0.15) ×25
= 47.9 kN

Dead weight of Beam = 0.23 × 0.25 × (4.25 + 2) × 25 + 0.23 × 0.25 × (4.25/2
+2) × 25
= 15.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete) ×2
= 14.4 kN

Dead load due to finish partition = 4.25 ×2×1.5 + 4.25/2 × 2 × 1.5
= 19.75 kN

Live load experienced = 4.25 × 2× 4 + 4.25/2 × 2 × 1.5
= 51 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead
Weight of column + Dead load due to finish + Live load
= 2×47.9 + 15.92 + 14.4 + 19.75 + 51
= 185 kN

Axial loads on Foundation D3-

Dead weight of slab= (4.25 × 4 × .15) ×Unit wt of concrete + (4.25 ×4 ×0.15)
× Unit wt of concrete
= (4.25 × 4 × .15) × 25 kN + (4.25 ×4×0.15) ×25
= 63.67 kN

Dead weight of Beam = 0.23× 0.25 × (4.25/2 + 4) × 25 + 0.23 × 0.25 × (4.25/2
+ 4) × 25
= 18.92 kN

Dead weight of Column = 0.3×0.35×2.75(Height of column)×25(unit weight of concrete) ×2
= 14.4 kN

Dead load due to finish partition = 4.25 ×2×1.5 + 4.25 × 2 × 1.5
= 25.375 kN

Live load experienced = 4.25 × 2× 4 + 4.25 × 2 × 1.5
= 68 kN

Total Load on A1= 2×dead weight of slab + Dead weight of Beam +Dead Weight of column + Dead load due to finish + Live load
= 2×63.67 + 18.92 + 14.4 + 25.375 + 68
= 230 kN

Because our structure is symmetric about the D columns, therefore loads on foundation will also be symmetric about D columns:-

Load on E1 = Load on A1 = 67 kN
Load on E2 = Load on A2 = 130 kN
Load on E3 = Load on A3 = 67 kN
Load on E1 = Load on B1 = 132 kN
Load on F2 = Load on B3 = 240 kN
Load on F3 = Load on B3 = 132 kN
Load on G1 = Load on C1 = 185 kN
Load on G2 = Load on C2 = 335 kN
Load on G3 = Load on C3 = 185 kN

5.4 Width Calculation of footings:

Taking depth of footing(D_f) from ground level = 1.5m

Net Safe Bearing capacity(q_{ns}) of soil at depth 1.5m = 140 kN/m^2 (From PLT test)
Surcharge above the footing = $\gamma \times D_f$
= 18 kN/m^2 × 1.5m
= 27 kN/m^2

Assumming Square footing, Area of footing= B^2 (Where is Width of footing)

$$Q_{ns} = \frac{Load\ on\ foundation}{area\ of\ footing} = \frac{Load}{B\times B}$$

$$B^2 = \frac{Load}{Qns}$$

But considering Soil Surcharge, we get empirical formula,

$$B = \sqrt{\frac{Load - 2.83}{113}}$$

Calculating width of Foundations based on the loads calculated:-

For Foundation A1 -

Load = 67 kN

$$B = \sqrt{\frac{Load - 2.83}{113}} = \sqrt{\frac{67 - 2.83}{113}} = 0.75m$$

For Foundation A2 -

Load = 130 kN

$$B = \sqrt{\frac{Load - 2.83}{113}} = \sqrt{\frac{130 - 2.83}{113}} = 0.95m$$

For Foundation A3 -

Load = 67 kN

$$B = \sqrt{\frac{Load - 2.83}{113}} = \sqrt{\frac{67 - 2.83}{113}} = 0.75m$$

For Foundation B1 -

Load = 132 kN

$$B = \sqrt{\frac{Load - 2.83}{113}} = \sqrt{\frac{132 - 2.83}{113}} = 0.96m$$

For Foundation B2 -

Load = 240 kN

$$B = \sqrt{\frac{Load - 2.83}{113}} = \sqrt{\frac{240 - 2.83}{113}} = 1.3m$$

For Foundation B3 -

Load = 132 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{132-2.83}{113}}$ = 0.96m

For Foundation C1 -

Load = 185 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{185-2.83}{113}}$ = 1.18m

For Foundation C2 -

Load = 335 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{335-2.83}{113}}$ = 1.65m , providing 1.7m

For Foundation C3 -

Load = 185 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{185-2.83}{113}}$ = 1.18m

For Foundation D1 -

Load = 230 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{230-2.83}{113}}$ = 1.3m

For Foundation D2 -

Load = 412 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{412-2.83}{113}}$ = 1.85m , providing 2m

For Foundation D3 -

Load = 185 kN

B = $\sqrt{\frac{Load-2.83}{113}}$ = $\sqrt{\frac{185-2.83}{113}}$ = 1.3m

But since we are considering the foundations as shallow foundations, and according to terzaghi analaysis which is valid only for shallow foundations, the width of the footing should be equal to or greater than Depth of the foundation. Therefore minimum width of foundation for our structure will be equal to 1.5m (Depth of foundation).

5.5 Consolidation Settlement -

$\Delta H = \frac{C_c}{1+e_0} H \log_{10} \frac{\boldsymbol{\sigma_v} + \Delta\sigma_v}{\boldsymbol{\sigma_v}}$

C_c= Compression index =.009(w_L-10)

σ_v = Effective initial overburden pressure = $\gamma \times (D_f + C.L_{layer})$

□σ_v=Vertical stress increment due to footing load.= $\frac{Load}{Area\ of\ spread}$

Area of spread = B'× B' , B'=Width of footing at $C.L_{layer}$

H= Thickness of clay layer (m)

Assuming Load dispersion at 30˚ from the vertical.

C_c = (liquid limit-10) × 0.009

$E_0 = \frac{G \times \gamma w}{\frac{\gamma}{1+w}} - 1$, G= Specific gravity = 2.65 , w= water content

γ_w = Saturated unit wt = 10kN/m^3

Table 4 : Calculation of settlement

Foundation	Layer	Depth of layer (m)	Layer thickness (m)	σ_v KN/m^2	$\Delta\sigma_v$ KN/m^2	C_c	e_0	Settlement ΔH (mm)	Total Settlement (mm)
A1	1	2.0	0.50	31.5	17.3	0.122	0.6	6.37	
	2	2.75	0.75	42.75	14.2	0.126	0.645	7.34	13.67
A2	1	2.0	0.50	31.5	30.94	0.122	0.6	9.8	
	2	2.75	0.75	42.75	25.3	0.126	0.645	15.3	25.1
A3	1	2.0	0.50	31.5	17.3	0.122	0.6	6.37	
	2	2.75	0.75	42.75	14.2	0.126	0.645	7.34	13.67
B1	1	2.0	0.50	31.5	31	0.122	0.6	9.9	
	2	2.75	0.75	42.75	26.2	0.126	0.645	15.2	25.2
B2	1	2.0	0.50	31.5	57.4	0.122	0.6	15.3	
	2	2.75	0.75	42.75	46.9	0.126	0.645	23.7	39
B3	1	2.0	0.50	31.5	31	0.122	0.6	9.9	
	2	2.75	0.75	42.75	26.2	0.126	0.645	15.2	25.2
C1	1	2.0	0.50	31.5	48	0.122	0.6	13.63	
	2	2.75	0.75	42.75	39	0.126	0.645	11.8	25.4
C2	1	2.0	0.50	31.5	84	0.122	0.6	19.2	
	2	2.75	0.75	42.75	47.74	0.126	0.645	24	43
C3	1	2.0	0.50	31.5	48	0.122	0.6	13.63	
	2	2.75	0.75	42.75	39	0.126	0.645	11.8	25.4
D1	1	2.0	0.50	31.5	62	0.122	0.6	16.0	
	2	2.75	0.75	42.75	51	0.126	0.645	19.0	35
D2	1	2.0	0.50	31.5	88	0.122	0.6	19.6	
	2	2.75	0.75	42.75	50	0.126	0.645	24.77	44.6
D3	1	2.0	0.50	31.5	62	0.122	0.6	16.0	
	2	2.75	0.75	42.75	51	0.126	0.645	19.0	35

Maximun settlement is coming out for foundation D2 = 44.6mm < 65mm(permissible settlement for clays according to I.S Code 1904(1986), Code of practice for design and construction of foundation on soils.Therefore our Foundations are safe in settlement.

5.6 Depth of footing

Depth of footing is calculated on basis of 3 criteria :-

1. One way shear method
2. Two way shear method
3. Bending moment

Depth is calculated using the above stated 3 criteria's and the maximum of the three depth obtained from each criteria is taken as depth of footing.
Minimum depth of footing according to IS 456:2000 is 150mm

5.6.1 One way shear method

The critical section for one way shear occurs at a distance 'd' from the face of the column.(As shown in fig.)

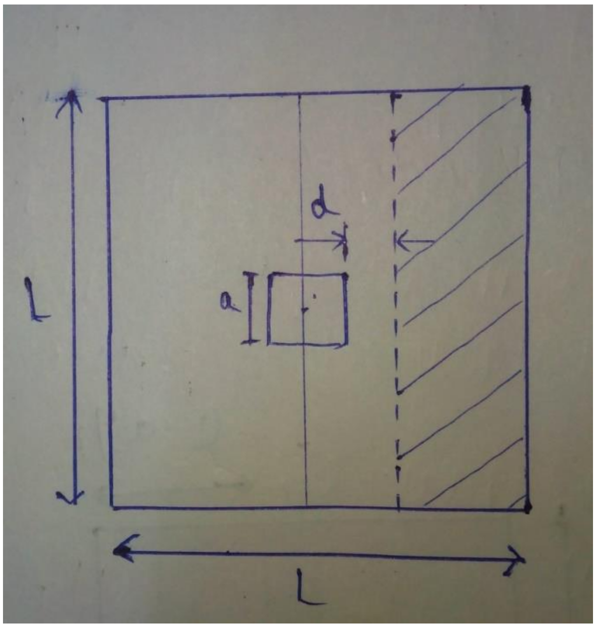


Fig 21 – One way critical section

Shear force V at the critical section at d distance away from the face of the column is

$V = q \times L \times \{(L-a)/2 - d'\}$, where ($q = \frac{Factored\ Load\ P}{L*L}$) , a= column width

$V = P/L^2 \times L \times \{(L-a)/2 - d'\}$ P = 1.5× Design load

$V = \frac{P}{2L}(L - a - 2d')$

Now,

$\tau_c \times L \times d' = V = \frac{P}{2L}(L - a - 2d')$

Where τ_c = maximum shear stress = 3.1 N/mm^2 (IS 456:2000)

Table 5 : Maximum shear Stress

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm^2

(*Clauses* 40.2.3, 40.2.3.1, 40.5.1 *and* 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm^2	2.5	2.8	3.1	3.5	3.7	4.0

On simplifying the above equation we get,

$d' = \frac{P(L-a)}{2(P+\tau \times L \times L}$

5.6.2 Two way shear method

The critical section for the two way shear or punching shear occurs at a distance of d/2 from the face of the column (See Fig.), where a is the side of the column

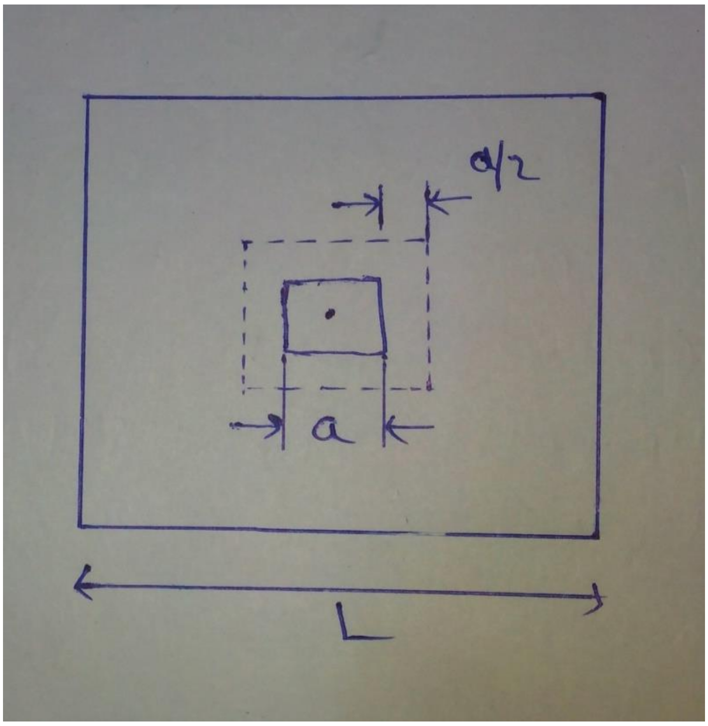


Fig 22 – Two way critical section

punching area of footing = $(a + d)^2$

Perimeter of the critical section = 4 (a+d)

Considering equillibrium of forces,

$$\frac{P}{L2}\ [L^2 - (a+d)^2\ ] = 4\tau_v(a+d)d$$

Where τ_v = Punching shear stress

$= 0.25\sqrt{fck}$

$= 0.25\times\sqrt{25}$ N/mm^2

$= 1.25$ N/mm^2

Value of d will be calculated from the above equation.

5.6.3 Bending moment method(Flexure method)

The critical section for flexure occurs at the face of the column (Fig.).

The projection of footing beyond the column face is treated as a cantilever slab subjected to factored upward pressure of soil.

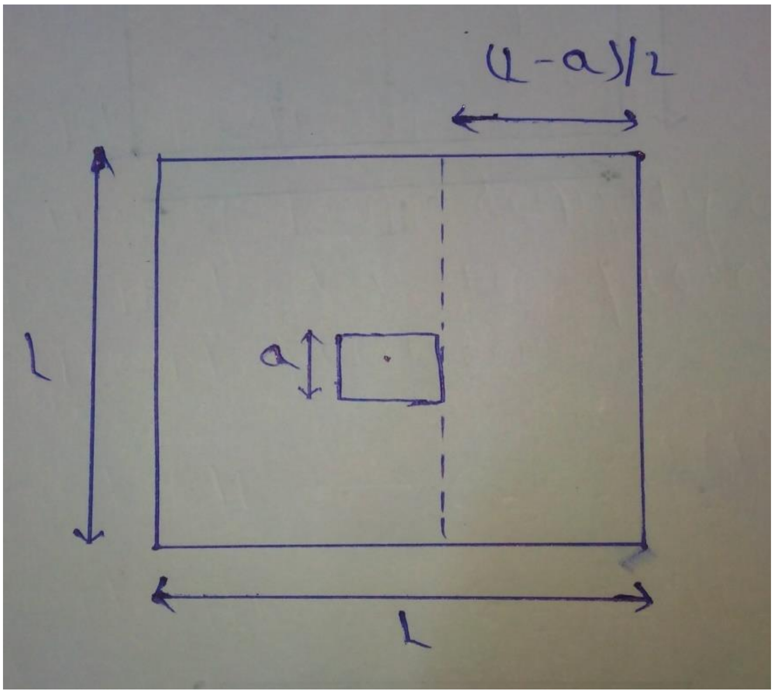


Fig23 – Bending Moment critical section

Moment at face of column

$$M = \frac{P}{L \times L}\ \left[\frac{L(L-a)2}{8}\right]$$

$$d = \sqrt{\frac{M}{0.138 \times fck \times L}}$$

Table 6: Calculation of Bending Moment

FOUNDATION	FOOTING WIDTH(m)	MOMENT AT THE FACE (KN-m)
A1	1.5	12.06
A2	1.5	23.4
A3	1.5	12.06
B1	1.5	24.1
B2	1.5	43.2
B3	1.5	24.1
C1	1.5	33.3
C2	1.7	60.3
C3	1.5	33.3

D1	1.5	41.4
D2	2.0	73.8
D3	1.5	41.4

5.6.4 Calculating depth using above three criteria,

For Foundation A1

One way, $d = \frac{67\times1.5(1500-300)}{2(67\times1.5+310\times1500\times1500} = 72$ mm

Two way, $\frac{1.5\times67}{1.5\times1.5}$ $[1.5^2 - (.3 + d)^2] = 4\times1.25(.3 + d)d$

$d = 52$mm

Bending moment, $d = \sqrt{\frac{12.06\times1000}{.138\times25\times1.5}} = 48$mm

For Foundation A2

One way, $d' = \frac{130\times1.5(1500-300)}{2(130\times1.5+310\times1500\times1500} = 143$ mm

Two way, $\frac{1.5\times130}{1.5\times1.5}$ $[1.5^2 - (.3 + d)^2] = 4\times1.25(.3 + d)d$

$d = 85$mm

Bending moment, $d = \sqrt{\frac{23.06\times1000}{.138\times25\times1.5}} = 67$mm

For Foundation A3

One way, $d' = \frac{67\times1.5(1500-300)}{2(67\times1.5+310\times1500\times1500} = 72$ mm

Two way, $\frac{1.5\times67}{1.5\times1.5}$ $[1.5^2 - (.3 + d)^2] = 4\times1.25(.3 + d)d$

$d = 52$mm

Bending moment, $d = \sqrt{\frac{12.06\times1000}{.138\times25\times1.5}} = 48mm$

For Foundation B1

One way, $d' = \frac{132\times1.5(1500-300)}{2(132\times1.5+310\times1500\times1500} = 145$ mm

Two way, $\frac{1.5\times132}{1.5\times1.5}$ $[1.5^2 - (.3 + d)^2] = 4\times1.25(.3 + d)d$

d = 113mm

Bending moment, $d = \sqrt{\frac{24.1\times1000}{.138\times25\times1.5}} = 68mm$

For Foundation B2

One way, $d' = \frac{240\times1.5(1500-300)}{2(240\times1.5+310\times1500\times1500} = 190$ mm

Two way, $\frac{1.5\times240}{1.5\times1.5}$ $[1.5^2 - (.3 + d)^2] = 4\times1.25(.3 + d)d$

d = 158mm

Bending moment, $d = \sqrt{\frac{43.2\times1000}{.138\times25\times1.5}} = 93mm$

For Foundation B3

One way, $d' = \frac{132\times1.5(1500-300)}{2(130\times1.5+310\times1500\times1500} = 143$ mm

Two way, $\frac{1.5\times132}{1.5\times1.5}$ $[1.5^2 - (.3 + d)^2] = 4\times1.25(.3 + d)d$

d = 113mm

Bending moment, $d = \sqrt{\frac{24.1\times1000}{.138\times25\times1.5}} = 68mm$

For Foundation C1

One way, d' = $\frac{185\times1.5(1500-300)}{2(185\times1.5+310\times1500\times1500}=$ 162 mm

Two way, $\frac{1.5\times185}{1.5\times1.5}$ [1.5^2 - $(.3 + d)^2$] = 4×1.25(.3 + d)d

d = 135mm

Bending moment, d = $\sqrt{\frac{33.3\times1000}{.138\times25\times1.5}}=$ 82mm

For Foundation C2

One way, d' = $\frac{335\times1.5(1700-300)}{2(335\times1.5+310\times1700\times1700}=$ 250 mm

Two way, $\frac{1.5\times335}{1.5\times1.5}$ [1.5^2 - $(.3 + d)^2$] = 4×1.25(.3 + d)d

d = 192mm

Bending moment, d = $\sqrt{\frac{60.3\times1000}{.138\times25\times1.5}}=$ 103mm

For Foundation C3

One way, d' = $\frac{185\times1.5(1500-300)}{2(185\times1.5+310\times1500\times1500}=$ 162 mm

Two way, $\frac{1.5\times185}{1.5\times1.5}$ [1.5^2 - $(.3 + d)^2$] = 4×1.25(.3 + d)d

d = 135mm

Bending moment, d = $\sqrt{\frac{33.3\times1000}{.138\times25\times1.5}}=$ 82mm

For Foundation D1

One way, d' = $\frac{230\times1.5(1500-300)}{2(230\times1.5+310\times1500\times1500}=$ 202 mm

Two way, $\frac{1.5\times230}{1.5\times1.5}$ [1.5^2 - $(.3 + d)^2$] = 4×1.25(.3 + d)d

d = 154mm

Bending moment, d = $\sqrt{\frac{41.4\times1000}{.138\times25\times1.5}} = 84$mm

For Foundation D2

One way, d' = $\frac{412\times1.5(2000-300)}{2(412\times1.5+310\times2000\times2000} = 293$ mm

Two way, $\frac{1.5\times412}{1.5\times1.5}$ [1.5^2 - $(.3 + d)^2$] = 4×1.25(.3 + d)d

d = 236mm

Bending moment, d = $\sqrt{\frac{73.8\times1000}{.138\times25\times1.5}} = 108$mm

For Foundation D3

One way, d' =$\frac{230\times1.5(1500-300)}{2(230\times1.5+310\times1500\times1500} = 202$ mm

Two way, $\frac{1.5\times230}{1.5\times1.5}$ [1.5^2 - $(.3 + d)^2$] = 4×1.25(.3 + d)d

d = 154mm

Bending moment, d = $\sqrt{\frac{41.4\times1000}{.138\times25\times1.5}} = 84$mm

Now we'll take the maximum value of depth obtained from all the criteria for all the foundations.Minimum depth = 150mm (IS 456:2000).

Table 7: Effective depth

Foundation	One way shear depth d_1(mm)	Two way shear depth d_2(mm)	Bending moment depth d_3(mm)	Max(d_1,d_2,d_3) (mm)	Minimun depth (mm)	Total effective depth(d) (mm)
A1	72	52	48	72	150	150
A2	143	85	67	143	150	150
A3	72	52	48	72	150	150
B1	145	113	68	145	150	150
B2	190	158	93	190	150	190
B3	145	113	68	145	150	150
C1	162	135	82	162	150	162
C2	250	192	103	250	150	250
C3	162	135	82	162	150	162
D1	202	154	84	202	150	202
D2	293	236	108	293	150	293
D3	202	154	84	202	150	202

5.7 R.C.C Design of Footings

The distribution of the total tensile reinforcement, calculated in accordance with the moment at critical sections, as specified in part (c) of this section, shall be done as given below for one-way and two-way footing slabs separately.

(i) In one-way reinforced footing slabs like wall footings, the reinforcement shall be distributed uniformly across the full width of the footing i.e., perpendicular to the direction of wall. Nominal distribution reinforcement shall be provided as per cl. 34.5 of IS 456 along the length of the wall to take care of the secondary moment, differential settlement, shrinkage and temperature effects.

(ii) In two-way reinforced square footing slabs, the reinforcement extending in each direction shall be distributed uniformly across the full width/length of the footing.

5.7.1 Transfer of load at the base of column

All forces and moments acting at the base of the column must be transferred to the pedestal, if any, and then from the base of the pedestal to the footing, (or directly from the base of the column to the footing if there is no pedestal) by compression in concrete and steel and tension in steel. Compression forces are transferred through direct bearing while tension forces are transferred through developed reinforcement. The permissible bearing stresses on full area of concrete shall be taken as given below from cl.34.4 of IS 456:

$\sigma_{br} = 0.25f_{ck}$, in working stress method,

The stress of concrete is taken as $0.45f_{ck}$ while designing the column. Since the area of footing is much larger, this bearing stress of concrete in column may be increased considering the dispersion of the concentrated load of column to footing. Accordingly, the permissible bearing stress of concrete in footing is given by (cl.34.4 of IS 456):

$\sigma_{br} = 0.45f_{ck}\,(A_1/A_2)^{1/2}$ with a condition that $(A_1/A_2)^{1/2} \leq 2.0$

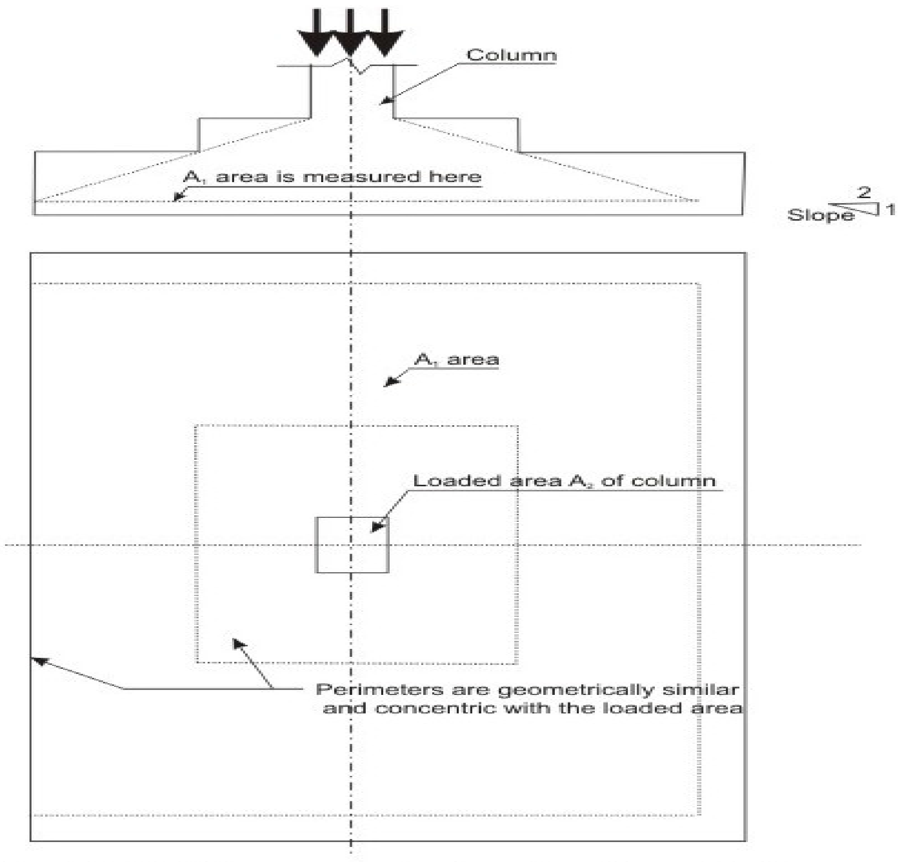


Fig. 11.28.15: Bearing area in sloped or stepped footing

Fig 24 Bearing Area in footing

where A_1 = maximum supporting area of footing for bearing which is geometrically similar to and concentric with the loaded area A_2, as shown in Fig.

A_2 = loaded area at the base of the column.

If the permissible bearing stress on concrete in column or in footing is exceeded, reinforcement shall be provided for developing the excess force (cl.34.4.1 of IS 456), either by extending the longitudinal bars of columns into the footing (cl.34.4.2 of IS 456) or by providing dowels as stipulated in cl.34.4.3 of IS 456

5.7.2 Steel Reinforcement required

Calculating required area of main steel using RCC analysis according to indian standards using IS-456:2000.

$$A_{st} = \frac{M}{.87 * fy * z}\text{, where}$$

M= limiting moment

$$= \frac{P}{\mathrm{L} \times \mathrm{L}} \left[\frac{\mathrm{L}(\mathrm{L}-\mathrm{a})2}{8} \right]$$

$$z = \text{lever arm} = d\left(1\text{-}0.42\,\frac{x}{d}\right)$$

$$\frac{x}{d} = 1.2 - \left[1.44 - \frac{6.6 * M}{fck * L * \mathrm{d}2}\right]^{1/2}$$

Steel required is calculated in table

Table 8: Calculation of Reinforcement

Foundation	Moment (kN-m)	Effective depth (mm)	$\frac{x}{d}$	Lever arm (mm)	A_{st} (mm^2)
A1	12.06	150	.037	147.6	226.2
A2	23.4	150	.074	145.4	4465.9
A3	12.06	150	.037	147.6	226.2
B1	24.1	150	.076	145.2	459.6
B2	43.2	190	.085	183.2	653.14
B3	24.1	150	.076	145.2	459.8
C1	33.3	162	.091	155.8	591.86

C2	60.3	250	.068	242.8	687.77
C3	33.3	162	.091	155.8	591.86
D1	41.4	202	.072	195.9	585.32
D2	73.8	293	.061	285.5	715.85
D3	41.4	202	.072	195.9	585.32

Now According to IS 456:2000, Minimum % of steel provided P_t = 0.12% for temperature and shrinkage

Therefore for all the footings, Minimum Required steel is calculated in Table

Table 9: Reinforcement Description

Foundation	**Calculated A_{st} (mm^2)**	**Minimum A_{st} (mm^2)**	**Actual A_{st} (mm^2)**	**Description**	**Provided A_{st} (mm^2)**
A1	326.2	270	330	7 bars- 8mm ϕ	350
A2	465.9	270	470	6 bars- 10mm ϕ	472
A3	326.2	270	330	7 bars- 8mm ϕ	350
B1	459.6	270	460	6 bars- 10mm ϕ	472
B2	653.14	342	655	6 bars- 12mm ϕ	680
B3	459.8	270	460	6 bars- 10mm ϕ	472
C1	591.86	292	595	6 bars- 12mm ϕ	680
C2	687.77	510	690	7 bars- 12mm ϕ	750
C3	591.86	292	600	6 bars- 12mm ϕ	680
D1	585.32	370	590	6 bars 12mm ϕ	680
D2	715.85	702	720	4 bars 16mm ϕ	800
D3	585.32	370	590	6 bars 12mm ϕ	680

5.8 Check for bearing stress

According to the Clause 34.4 of IS 456:2000,the permissible stress at the base of the column is

The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to

$\sqrt{\frac{A_1}{A_2}}$ but not greater than 2;

where

A_1 = supporting area for bearing of footing, which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal; and

A_2 = loaded area at the column base.

Fig 25 – Permissible bearing stress

Permissible bearing stress $\sigma_{br} = 0.45 \times f_{ck} \times \sqrt{(A_1/A_2)}$

Such that $\sqrt{(A_1/A_2)} \leq 2.0$ always.

For Minimum Footing dimension 1500mm × 1500mm and Column Dimension = 300mm ×300mm

$$\sqrt{(A_1/A_2)} = \sqrt{(1500 \times 1500/300 \times 300)}$$

$$= 5 > 2$$

Therefore for all the Footings, We will take $\sqrt{(A_1/A_2)} = 2$

Now to compute whether the transfer of load at the base of the column is done properly and the design is safe, we will have to calculate the bearing strength at the end of the column and bearing strength at the base of the footing.

Permissible Bearing Stress = $0.45 \times f_{ck} \times \sqrt{(A_1/A_2)}$
$= 0.45 \times 25 \times 2$
$= 22.5\ N/mm^2$

Actual bearing stress = $\dfrac{Factored\ load}{Area\ at\ Columnn\ base}$

Area at the column base = $300 \times 300\ mm^2 = 90000\ mm^2$

Table 10: Permissible bearing stress

Foundation	Facored Load (1.5×P) (kN)	Allowable Bearing stress (N/mm^2)	Permissible bearing stress (N/mm^2)	Design Consideration
A1	101	1.12	22.5	Safe
A2	195	2.20	22.5	Safe
A3	101	1.12	22.5	Safe
B1	198	2.30	22.5	Safe
B2	360	4.00	22.5	Safe
B3	198	2.30	22.5	Safe
C1	278	3.11	22.5	Safe
C2	503	5.66	22.5	Safe
C3	278	3.11	22.5	Safe
D1	345	3.88	22.5	Safe
D2	620	6.88	22.5	Safe
D3	345	3.88	22.5	Safe

5.9 Check for Development Length(L_d)

Sufficient development length should be available for the reinforcement from the critical section.

Here, the critical section considered for Ld is that of flexure.

$L_d = \frac{\phi \times \sigma s}{4 \times \tau b}$

Where τ_b = Design bond stress

ϕ = Nominal diameter of bars

26.2.1.1 Design bond stress in limit state method for plain bars in tension shall be as below:

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
Design bond stress, τ_{bd}, N/mm^2	1.2	1.4	1.5	1.7	1.9

Fig 26- Design bond stress(IS 456)

$L_d = 47 \times \phi$

For 8mm Diameter bars, $L_d = 47 \times 8 = 376$mm

Providing 60 mm side cover, the total length available from the critical section is $0.5(L - a) - 60 + dmin = 0.5(1500 - 300) - 60 + 150 = 690 > L_d$. Hence ok

For 10mm Diameter bars, $L_d = 47 \times 10 = 470$mm

Providing 60 mm side cover, the total length available from the critical section is $0.5(L - a) - 60 + dmin = 0.5(1500 - 300) - 60 + 150 = 690 > L_d$. Hence ok

For 12mm Diameter bars, $L_d = 47 \times 12 = 564$mm

Providing 60 mm side cover, the total length available from the critical section is $0.5(L - a) - 60 + dmin = 0.5(1500 - 300) - 60 + 150 = 690 > L_d$. Hence ok

For 16mm Diameter bars, $L_d = 47 \times 12 = 752$mm

Providing 60 mm side cover, the total length available from the critical section is $0.5(L - a) - 60 + dmin = 0.5(2000 - 300) - 60 + 150 = 940 > L_d$. Hence ok

5.10 STAAD.Pro Analysis

Before the availability of computers and specialized analysis and design programs, towers were often designed by graphical methods. Today's analysis tools allow engineers to refine designs to an unprecedented degree, and as a result, many utilities feel testing is not warranted. However, while great strides have been made in the analysis and design of latticed steel transmission towers, differences between analysis results and full-scale tests still occur.

STAAD.Pro features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and dynamic analysis capabilities. From model generation, analysis and design to visualization and result verification, STAAD.Pro is the professional's choice for steel, concrete, timber, aluminum and cold-formed steel design of low and high-rise buildings, culverts, petrochemical plants, tunnels, bridges, piles and much more. The following key STAAD.Pro tools help simplify ordinarily tedious tasks:

- The STAAD.Pro Graphical User Interface incorporates Research Engineers' innovative tabbed page layout. By selecting tabs, starting from the top of the screen and heading down, you input all the necessary data for creating, analyzing and designing a model. Utilizing tabs minimizes the learning curve and helps insure you never miss a step.
- The STAAD.Pro Structure Wizard contains a library of trusses and frames. Use the Structure Wizard to quickly generate models by specifying height, width, breadth and number of bays in each direction. Create any customizable parametric structures for repeated use. Ideal for skyscrapers, bridges and roof structures.
- The STAAD.Pro Foundation is useful for analysis and design of the structures and it is possible to import the structure directly from STAAD structure wizard, thus simplifying the design process

Features of STAAD.Pro

- "Concurrent Engineering" based user environment for model development, analysis, design, visualization and verification
- Full range of analysis including static, P-delta, pushover, response spectrum, time history, cable (linear and non-linear), buckling and steel, concrete and timber design included with no extra charge
- Object-oriented intuitive 2D/3D graphical model generation
- Pull down menus, floating tool bars, tool tip help
- Quick data input through property sheets and spreadsheets

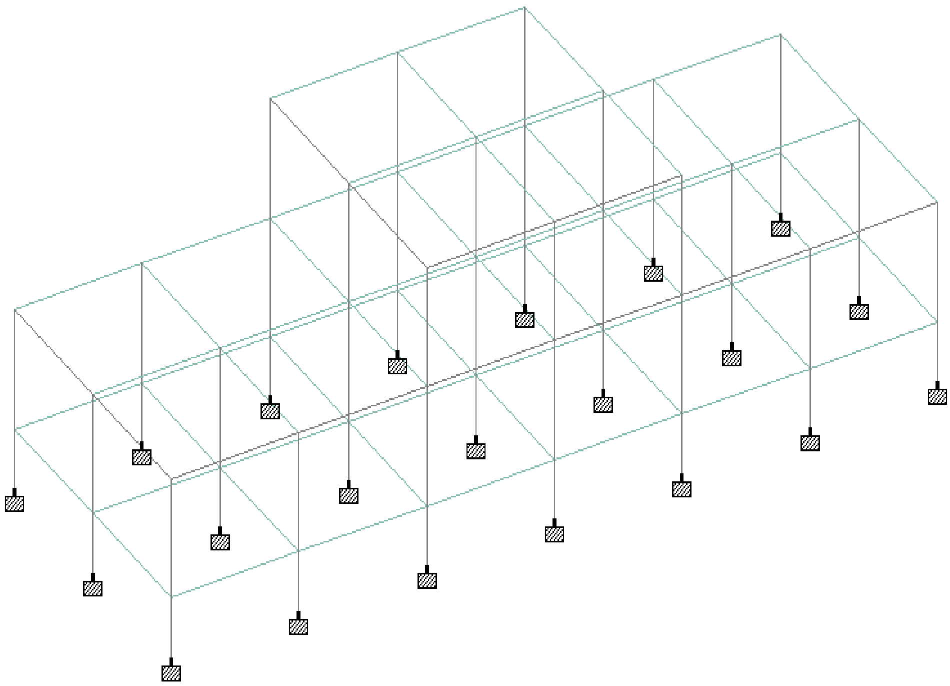


Fig 27 Structure in STAAD

5.9.1 Load Calculation on STAAD.Pro

Table 11 – Load Axial from STAAD.pro

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
52	1:DL	-0.089	52.964	-0.129	-0.054	0.000	0.026
	2:LL	0.059	17.096	0.054	0.026	0.000	-0.028
53	1:DL	0.177	92.666	-0.067	-0.028	-0.001	-0.092
	2:LL	0.058	34.249	0.080	0.038	-0.000	-0.028
54	1:DL	0.130	127.412	0.167	0.080	-0.000	-0.065
	2:LL	0.059	51.715	0.138	0.065	-0.000	-0.028
55	1:DL	0.000	151.567	0.146	0.069	0.000	-0.000
	2:LL	0.000	68.328	0.125	0.059	-0.000	-0.000
56	1:DL	-0.130	127.412	0.167	0.080	0.000	0.065
	2:LL	-0.059	51.715	0.138	0.065	0.000	0.028
57	1:DL	-0.177	92.666	-0.067	-0.028	0.001	0.092
	2:LL	-0.058	34.249	0.080	0.038	0.000	0.028
58	1:DL	0.089	52.964	-0.129	-0.054	-0.000	-0.026
	2:LL	-0.059	17.096	0.054	0.026	-0.000	0.028
59	1:DL	-0.155	92.729	-0.000	-0.000	-0.000	0.057
	2:LL	0.059	34.002	-0.000	-0.000	-0.000	-0.028
60	1:DL	0.343	168.266	-0.000	-0.000	0.000	-0.170
	2:LL	0.108	67.819	-0.000	-0.000	-0.000	-0.051
61	1:DL	0.202	225.318	0.000	-0.000	-0.000	-0.099
	2:LL	0.099	101.335	0.000	-0.000	-0.000	-0.047
62	1:DL	0.000	268.174	0.000	-0.000	0.000	-0.000
	2:LL	0.000	132.798	0.000	-0.000	-0.000	0.000
63	1:DL	-0.202	225.318	0.000	-0.000	0.000	0.099
	2:LL	-0.099	101.335	0.000	-0.000	0.000	0.047

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Reactions Cont...

Node	L/C	Horizontal FX (kN)	Vertical FY (kN)	Horizontal FZ (kN)	Moment MX (kNm)	Moment MY (kNm)	Moment MZ (kNm)
64	1:DL	-0.343	168.266	-0.000	-0.000	-0.000	0.170
	2:LL	-0.108	67.819	-0.000	-0.000	0.000	0.051
65	1:DL	0.155	92.729	-0.000	-0.000	0.000	-0.057
	2:LL	-0.059	34.002	-0.000	-0.000	0.000	0.028
66	1:DL	-0.089	52.964	0.129	0.054	-0.000	0.026
	2:LL	0.059	17.096	-0.054	-0.026	-0.000	-0.028
67	1:DL	0.177	92.666	0.067	0.028	0.001	-0.092
	2:LL	0.058	34.249	-0.080	-0.038	0.000	-0.028
68	1:DL	0.130	127.412	-0.167	-0.080	0.000	-0.065
	2:LL	0.059	51.715	-0.138	-0.065	0.000	-0.028
69	1:DL	0.000	151.567	-0.146	-0.069	-0.000	-0.000
	2:LL	0.000	68.328	-0.125	-0.059	0.000	-0.000
70	1:DL	-0.130	127.412	-0.167	-0.080	-0.000	0.065
	2:LL	-0.059	51.715	-0.138	-0.065	-0.000	0.028
71	1:DL	-0.177	92.666	0.067	0.028	-0.001	0.092
	2:LL	-0.058	34.249	-0.080	-0.038	-0.000	0.028
72	1:DL	0.089	52.964	0.129	0.054	0.000	-0.026
	2:LL	-0.059	17.096	-0.054	-0.026	0.000	0.028

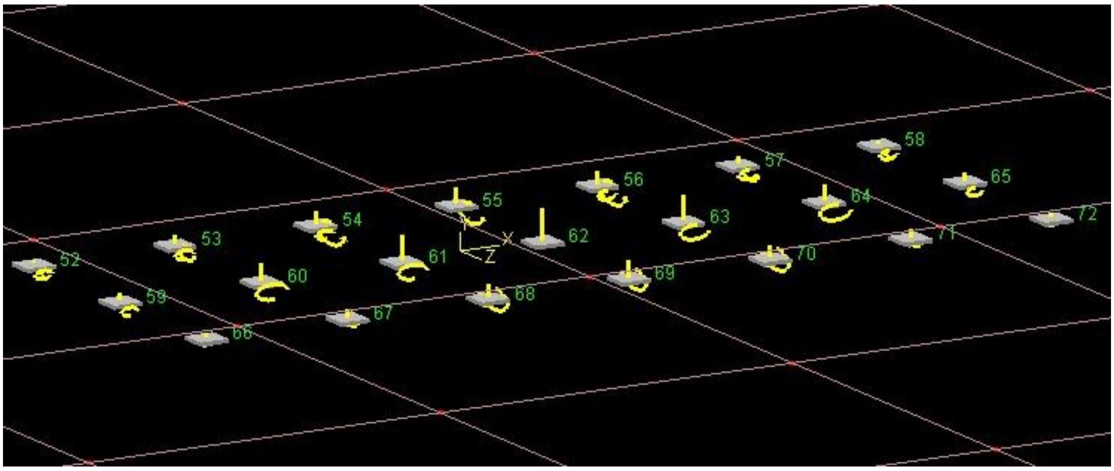


Fig 28- Layout of footings on STAAD.Foundation

5.11 Reinforcement detailing

Actual depth of Foundations = effective depth + nominal cover +1.5 × bar diameter

Nominal cover provided should be greater than 50 mm according to IS 456.
But if footing is to laid directly on the ground ,then cover provided should be atleast 75 mm.Therefore we are providing cover = 75mm

Table 12 – Actual depth calculation

Foundation	**Effective depth(mm)**	**Nominal cover(mm)**	**Bar Diameter (mm)**	**Actual depth (mm)**
A1	150	75	8	250
A2	150	75	10	250
A3	150	75	8	250
B1	150	75	10	250
B2	190	75	12	285
B3	150	75	10	250
C1	162	75	12	260
C2	250	75	12	350
C3	162	75	12	260
D1	202	75	12	300
D2	293	75	16	400
D3	202	75	12	300

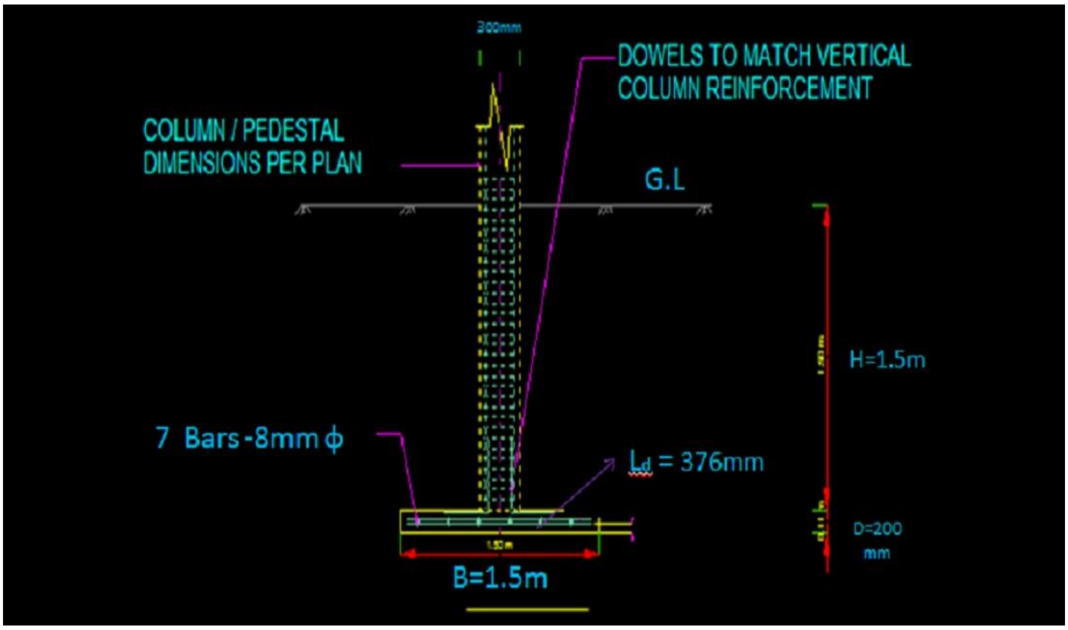


Fig 29 – Detailing for Foundation A1

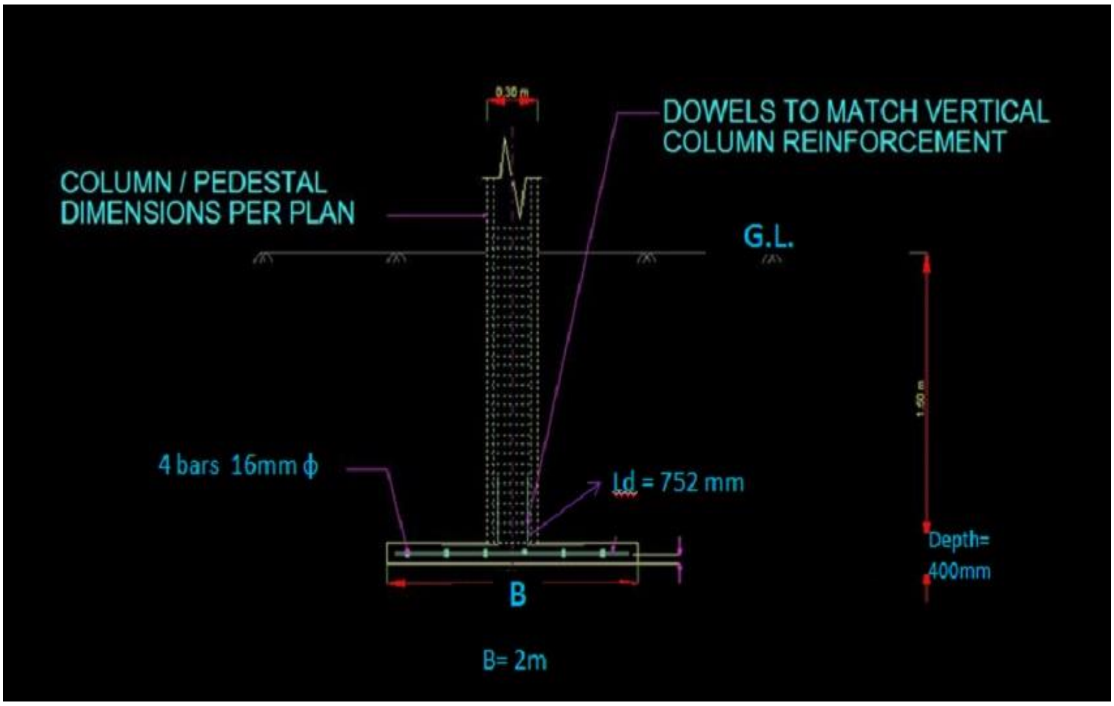
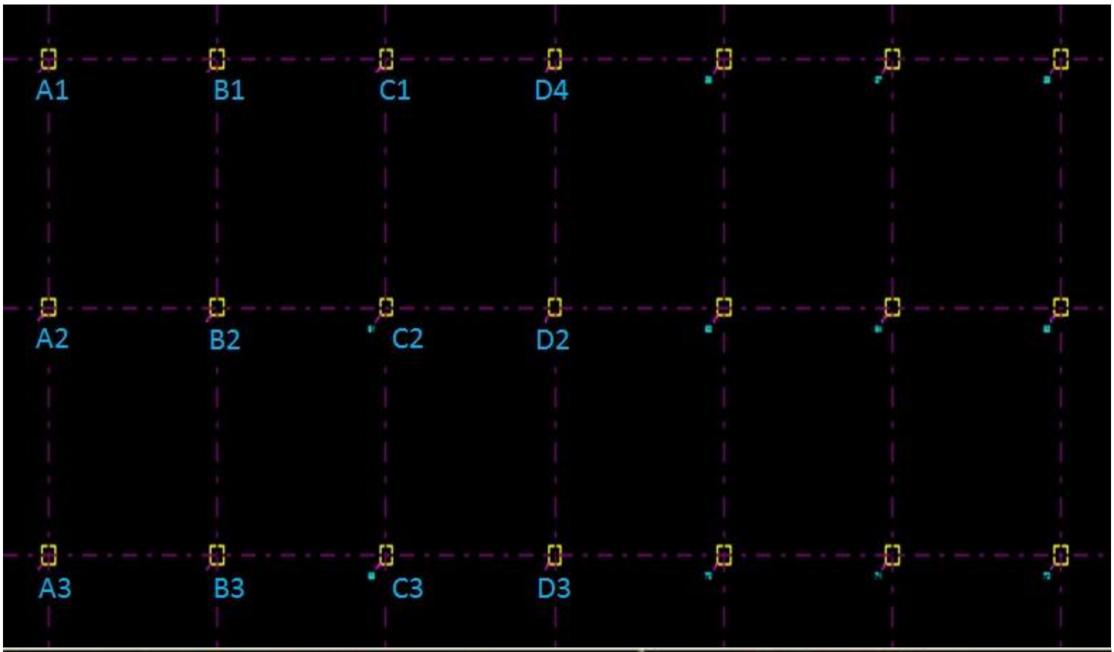


Fig 30 – Detailing for Foundation D2



Footing Size	Column Nos.
1.500 X 1.500 X 250	A1
1.500 X 1.500 X 250	A2
1.500 X 1.500 X 250	A3
1.500 X 1.500 X 250	B1
1.500 X 1.500 X 285	B2
1.500 X 1.500 X 250	B3
1.500 X 1.500 X 260	C1
1.700 X 1.700 X 350	C2
1.500 X 1.500 X 260	C3
1.500 X 1.500 X 300	D1
2.00 X 2.000 X 400	D2
1.500 X 1.500 X 300	D3

Fig 31- General Detailing of Footings

5.12 Design charts

The allowable bearing pressure method is sufficient for small to medium sized structures.However, larger structures especially with greater column loads, warrant a more easy and precise method for calculating the dimensions of footings.

Design charts are mainly graphs between the axial loads P and width of the footing B Procedure to develop Design charts is:

- Determine the footing shape.Each shape has its own Design chart
- Select the depth of foundation
- Set some value of the footing width B, then conduct a bearing capcity analysis and compute the column load .Then select a new series of values of B and compute the corresponding P and plot the points (B,P) on a graph.Continue the process until a perfect graph is obtained
- Developing the settlement curve
- Select a settlement value for a curve(eg. 20mm)
- Select a footing width B arbitrarily and corresponding load P
- By trial error, adjust the value of P until the computed settlement matches the assumed settlement.Plot this (B,P_a)
- Repeat for different settlement values

Design charts are like ready to implement design methods which are designed by a geotechnical engineer and handed over to a structural engineer, who can easily calculate the width of footing with the help of design loads, thus making the work of an engineer easier.

SECTION 6 : RESULTS AND CONCLUSIONS

At depth = 1.5m soil profile is low compressible silty clay.Bearing Capacity of the soil according to 3 performed tests that is Plate Load test, Standars penetration test and Direct cone penetration test are

Bearing capacity from different tests at D=1.5m

Test	**Depth**	**Nabp**
PLT	1.5m	14 t/m^2
DCPT	1.5m	21 t/m^2
SPT	1.5m	26 t/m^2

From the graph obtained in PLT test, it can be said our soil may be somewhere between partially cohesive soil and cohesive soil but most part of the graph is represented closely by behavior of cohesive soil.

Minimum net allowable bearing pressure is from PLT.

From the calculated N value which is 18, value of unconfined compressive strength (20 t/m^2) and unconfined shear strength (10 t/m^2) is known. Also it can be said that our soil is stiff clay.

PLT can be considered as the most effective and easy to perform test. Effective in the sense that it gives the direct relationship between allowable pressure and settlement at the depth of the footing also this test does not give the ultimate settlement particularly in case of cohesive soil as it is a short duration test.

There is a great variation in bearing capacity due to variation in ground water level and every method deals with this effect differently.

All the calculation related to soil analysis has been done, like bearing capacity and allowed settlement from various tests is known also the soil layer at which footing has to be designed is clay of low compressibility.

Design of isolated Footing for shopping complex at Pratap vihar, Ghaziabad, U.P.

Taking Safe Bearing Capacity = 140 kN/mm^2.

Table 13- Final results (a)

Foundation	**Axial Load (kN)**	**Dimensions (m × m)**	**Settlement (mm)**	**Effective footing depth (mm)**
A1	67	1.5m ×1.5m	13.67	150
A2	130	1.5m ×1.5m	25.1	150
A3	67	1.5m ×1.5m	13.67	150
B1	132	1.5m ×1.5m	25.2	150
B2	240	1.5m ×1.5m	39	190
B3	132	1.5m ×1.5m	25.2	150
C1	185	1.5m ×1.5m	25.4	162
C2	335	1.7m ×1.7m	43	250
C3	185	1.5m ×1.5m	25.4	162
D1	232	1.5m ×1.5m	35	202
D2	412	2.0m ×2.0m	44.6	293
D3	232	1.5m ×1.5m	35	202

Table 14 – Final result (b)

Foundation	**Actual depth (mm)**	**Provided A_{st} (mm^2)**	**Description**	**Development Length(L_d)**
A1	250	350	7 bars- 8mm ϕ	376
A2	250	472	6 bars- 10mm ϕ	470
A3	250	350	7 bars- 8mm ϕ	376
B1	250	472	6 bars- 10mm ϕ	470
B2	285	680	6 bars- 12mm ϕ	564
B3	250	472	6 bars- 10mm ϕ	470
C1	260	680	6 bars- 12mm ϕ	564
C2	350	750	7 bars- 12mm ϕ	564
C3	260	680	6 bars- 12mm ϕ	564
D1	300	680	6 bars 12mm ϕ	564
D2	400	800	4 bars 16mm ϕ	752
D3	300	680	6 bars 12mm ϕ	564

- This Work attempts to study, analyse and calculate the Bearing capacity of the soil of a real life location, Pratap vihar in Ghaziabad, Uttar Pradesh
- Test data of three tests namely- PLT, SPT, DCPT conducted by technicians from IIT-Roorkee is provided and safe bearing capacity of the soil at different depths is calculated on basis of the test data
- Safe bearing capacity(SBC) for depth = 1.5 m is coming out to be 140 kN/m^2
- Based on the calculation of SBC, Isolated(shallow) Foundation is designed for a proposed shopping complex on the same soil profile at depth = 1.5m
- Axial Loads on each foundation is calculated by considering every Load combination(Table
- Dimensions of footings are also calculated for Safe bearing Capacity.
- Settlement for each footing is calculated by Indian Standards and settlements are within permissible limits

- Footing depth is calculated by considering all the criteria stated in Indian Codes
- R.C.C Design for all the foundations is done and steel required is calculated.All the checks for tensile reinforcement is performed as stated in Indian design code IS 456:2000.

SECTION 7 : FUTURE SCOPE

After completely analyzing the soil through all the field test performed and data collected from them, I have successfully designed Isolated footing for a shopping complex . Now, as we have completely analyzed the soil,so now safe bearing capacity at different depths can be calculated using the same process and the calculated bearing capacities can be used to design foundations at different depths.For bigger structures having more than 3 floors, different foundations can be designed.If the loads on foundation is coming out to be very high and the safe bearing capacity of soil at desired depth is less, then Mat foundation can be designed for such structures and if the safe bearing capacity is more then Combined foundations or even deep foundation design can also be taken into the account.

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2. IS 456 (4^{th} revision)- 2000: "*Code of practice for Plane and Reinforced Concrete structures*", Bureau of Indian Standards code, India.
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6. Dr B.C. Punmia, Ashok Kumar Jain, Arun Kumar Jain et al, " *Soil Mechanics and Foundations*" , Laxmi Publications Limited, 16^{th} Edition: 2005

APPENDIX A- STAAD.Pro CODE

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 06-Apr-15

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

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MEMBER INCIDENCES

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ELEMENT INCIDENCES SHELL

75 8 9 23 22; 76 9 10 24 23; 78 10 11 25 24; 80 11 12 26 25; 81 12 13 27 26;

82 13 14 28 27; 83 22 23 37 36; 84 23 24 38 37; 86 24 25 39 38; 88 25 26 40 39;

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101 50 51 48 47; 102 48 45 44 47; 103 1 2 16 15; 104 2 3 17 16; 105 3 4 18 17;

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111 17 18 32 31; 112 18 19 33 32; 113 19 20 34 33; 114 20 21 35 34;

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103 TO 114 THICKNESS 0.15

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POISSON 0.17

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ALPHA 1e-005

DAMP 0.05

TYPE CONCRETE

STRENGTH FCU 27579

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CONSTANTS

MATERIAL CONCRETE ALL

SUPPORTS

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ELEMENT LOAD

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SELFWEIGHT Y -1

LOAD 2 LOADTYPE Live REDUCIBLE TITLE LL

ELEMENT LOAD

78 80 86 88 103 TO 114 PR GY -4

PERFORM ANALYSIS PRINT ALL

PERFORM ANALYSIS PRINT ALL

PERFORM ANALYSIS PRINT ALL

PERFORM ANALYSIS