Design of Bridge Sub Structure

Dissertation submitted to

Visvesvaraya National Institute of Technology, Nagpur

in partial fulfilment of the requirements for the award of the degree

Master of Technology
in
Structural Engineering

by

Yada Keshava (MT19STR026)

Under the guidance of

Dr. R. S. Sonparote

Department of Applied Mechanics

Visvesvaraya National Institute of Technology
Nagpur 440 010 (India)

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Declaration

I, Yada Keshava, hereby declare that this dissertation titled **"Design of Bridge Sub-Structure"** is carried out by me in the Department of Applied Mechanics of Visvesvaraya National Institute of Technology, Nagpur. The work is original and has not been submitted earlier whole or in part for the award of any degree/diploma at this or any other Institution / University.

Yada Keshava

(MT19STR026)

Date:

Certificate

This to certify that the dissertation titled **"Analysis of Bridge Sub-Structure"** is submitted by **Yada Keshava** in partial fulfillment of the requirements for the award of the degree of **Master of Technology in Structural Engineering**, VNIT Nagpur. The work is comprehensive, complete and fit for final evaluation.

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ABSTRACT

The analysis and design of Substructure of the simplest bridge type is laborious and cumbersome work, this may involves lots of human errors in calculations while designing different components of substructure. So in order to avoid these human errors and to make the analysis and design handy an attempt is made to study the design of the Substructure components and develop the spread sheets for design of pier, pile foundation, well foundation and spread foundations. The obtained results from spread sheets are compared with the values obtained from models developed in the SAP2000, which observes 10%-20% variation in the design moments. In this thesis further attempt is carried out to develop P-M interaction curve using VBA in excel, these curves are developed based on the guidelines given in IRC-112-2011 .The developed P-M interaction curve is compared with IS-456-2000 code curves and observed that there is no significant change in curves but the developed P-M interaction curve based on IRC 112-2011 lies higher side of the curves based on IS-456-2000.The effect of use of different mesh sizes in isolated footings in SAP2000 is also studied and found that the mesh size which is much lesser than the column size gives accurate results.

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CHAPTER 1

1 INTRODUCTION

1.1 General

Bridge is a structure which provides passage over an obstacle without disturbing the way beneath. The passage may be for a railway, a road, a canal, pedestrians or a pipeline and the obstacle to be crossed may be a river, a road, railways or a valley. The portion below the level of the bearing is generally referred to as the substructure. The design of bridge substructure is an important part of the overall design for a bridge and affects to a considerable extent the safety, the aesthetics and the economy of the bridge. Bridge substructure is a very important part as it safely transfers the loads from the superstructure to the soil in such a manner that the stresses on the soil are not high & the resulting deformations are within the acceptable limits. The selection of the pier depends on many factors like super structure size, orientation etc. The selection of the foundation system for a particular site depends on many considerations like the nature of subsoil, location of the bridge i.e., over a river, road, or a valley, etc. & the scour depth. A bridge may have the following types of foundations.

1.1.1 Open foundations

This type of foundations can be considered as shallow foundations, these foundations are also called as spread foundation. Open foundations are used when the suitable soil available in moderate heights which is firm to support the bridge structure. Usually this are constructed when hard soil is available within 1.5 to 3. Open foundation shown in Figure 1.1.

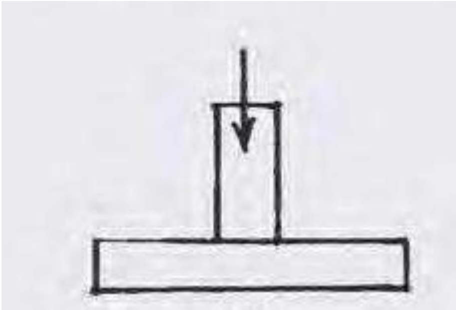


Figure 1. 1 Open or Spread foundation (Varghese 2010)

1.1.2 Well foundation

Well foundation is a deep foundation. It is the type of foundation which is commonly used for both road & railway bridges. This can carry very heavy vertical and lateral loads and can be sunk to great depths. One of the types of well foundations shown in Figure 1.2 can also be installed in a boulder stratum. It is a relatively rigid and massive structure.

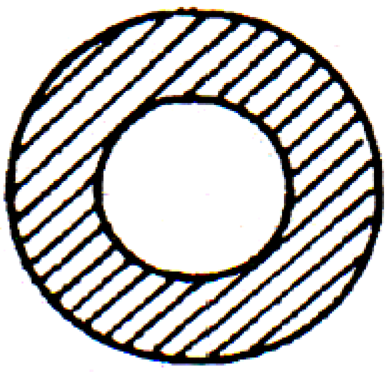


Figure 1. 2 Well foundation (Jagadeesh and Jayaram 2009)

1.1.3 Pile foundation

Pile foundation is also another type of deep foundation, the long and slender members called piles shown in Figure 1.3 which transfer loads through weak soil or rock strata or water to deeper soil having a high bearing capacity. In elevated road ways also, this can be used under normal conditions.

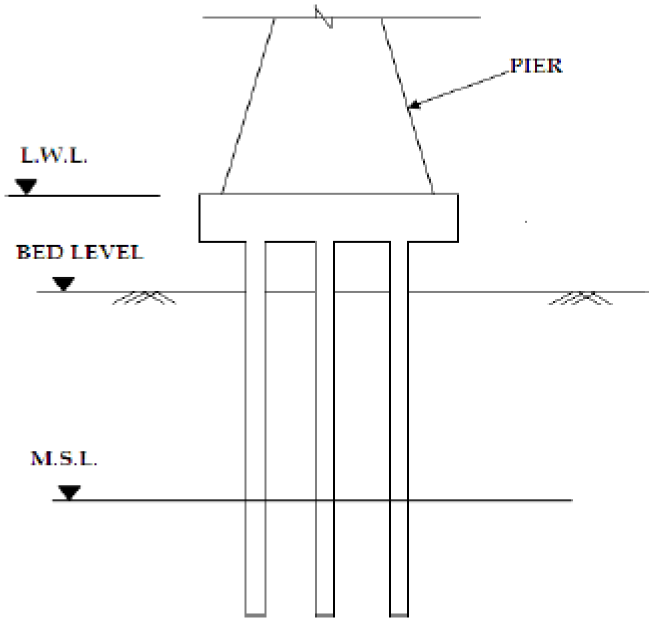


Figure 1. 3 Pile foundation (Jagadeesh and Jayaram 2009)

1.2 Objective of study

The commenced thesis is to study design and analysis of substructure of bridge

Following are the objectives defined for study.

- To develop the spread sheets in excel for design of pier, well foundation, pile foundation and Spread foundations.
- Comparison of spread sheet values with results obtained from the sap2000 modelling.
- To evaluate the optimum size of mesh and spring stiffness to be used in modelling of spread foundations.
- To develop the P-M interaction curves for piers refereeing IRC 112-2011.
- Comparison of P-M interactions curves of IS 456-2000 & IRC 112-2011

1.3 Organization of thesis

The work splits in to following seven chapters and described as follows

Chapter1: Introduction includes about the discussion on bridge substructure and its components and discussion on the objective of the thesis and the organization of thesis.

Chapter2: Literature review includes the overview of the various literature surveyed.

Chapter3: This chapter includes the study on analysis and design of bridge piers and discussion on the development of the P-M interaction curves as per IRC 112-2011 and discussion on the spread sheet developed for the design of pier.it also includes the discussion on the comparison of P-M curves between IRC 112-2011 and IS 456-2000.

Chapter4: This chapter includes the discussion on analysis and design well foundation .It also includes the discussion on the developed spread sheets for design and stability analysis. Comparison of the stresses and moments in well steining obtained from spreadsheets with the SAP2000 model values also covered.

Chapter5: This chapter includes the discussion on the analysis and design of pile foundations .It also includes the discussion on the developed spread sheets and comparison of design moments obtained with the SAP2000 models.

Chapter6: This chapter includes the discussion on design of spread foundation and developed spread sheets. It also includes study on the effect of different mesh size and soil stiffness in spread foundation in SAP2000 and comparison of design moments with SAP2000 results.

Chapter7: This chapter includes discussion on all the conclusions obtained from the study of the previous chapters.

CHAPTER 2

2 LITERATURE REVIEW

Several literature on the pile stiffness, sub grade reaction of soil, effect of mesh sizes in modelling and study on P-M curves are surveyed and briefly explained in this chapter. This literature studied includes papers and books.

Novak and Sharnouby (1983) presented the stiffness constants for single vertical piles in the form of tables and charts. The soil considered having the shear modulus either constant or with parabolic variation along depth, the butt considered is either fixed or pinned. The author observed that pin headed has lesser stiffness and damping than the fixed head and homogeneous soils have more stiffness and damping than the soil with parabolic variation of shear modulus.

Stavridis (2002) studied the interaction of the arbitrary structure with its foundation soil which is treated by representing the stratified soil with the end of a linear elastic half space model. In this method the author solved examples and compared with those with winkler's assumption of modulus of sub-grade reaction. The author observed some advantage over winkler's method where modulus of sub grade reaction does not represent a soil property resulting from geotechnical tests. Author concluded that in case of flexible beams loaded with limited concentrated loads and resting on stiff soil, the bending response are practically insignificant.

Tabsh and Shawa (2005) developed a relative stiffness factor Kr' which determines whether a footing can be considered as rigid for the analysis and design. The factor developed is the modified version of an expression proposed by meyerhof in 1953 which takes the size of column into the account. In this author studied by modelling different square and rectangle footings with plate elements and soil with spring elements by finite element analysis. Author concluded that if the value of relative stiffness factor is greater than 1 then it can be treated as rigid footing with reasonable accuracy . Author stated that maximum shear forces within a spread footing are less sensitive to changes in the stiffness of a footing than bending moments.

Chow and small (2005) presented a finite layer method used for the analysis of the pile rafts with piles of different lengths and diameters. The soil is divided with different material properties into horizontal layers and it is analysed for only vertical loads on the raft, interaction of raft-soil-pile are computed. Author concluded that the behaviour of piled raft is governed by the soil structure interaction and when non uniform load applied on raft the use of long piles under heavy loads will minimize the risk of tilting and differential settlements. Also concluded that finite layer method has more advantages than finite element in the analysis of piles of different length.

Daloglu and Girija (2012) developed the method to evaluate and equivalent modulus subgrade reaction K to be used in winkler model using non dimensional parameters. In this author used poisons ratio 0.25 .The graphs are developed such the k value can be computed using complete geometry and properties of overall system and the comparisons is done with the k suggested by biot and vesic. Author concluded that if constant value of subgrade reaction is used for uniformly loaded slab the displacements will be uniform and there will be no bending moment in slab. To get realistic results one should take higher values at the edges of slab and k value will be depended on soil layer depth.

Novak et al. (2013) presented the characteristics of vertical single pile like the stiffness constants and constants of equivalent viscous damping in the form of tables and charts. The shear modulus of soil is either changing parabolic or constant with the depth, the piles considered are both pinned and end bearing and the butt is either fixed or pinned. By considering the above data author compared and concluded that the fixed-headed piles have more stiffness than the pin headed piles and homogeneous soil will have high stiffness and damping than the parabolic variation of soil. End bearing piles generate less damping in vertical response compare to the floating piles.

More and Bindhu (2015) studied mesh size issue in finite element analysis. For this authors had considered the structure made up of steel and studied the effects of mesh size in numerical analysis results. Authors has done static and buckling analysis using the Femap and NX-Nastran. Author concluded that for the static analysis which is steady loading, the model should be discretized in to elements of 40 mm. For buckling analysis, the model should be discretized in to elements between 30-50 mm for getting the accurate and satisfied results.

Dutt (2015) studied on the effect of mesh size in the finite element analysis of cantilever beam using Creo 2.0 software. In this author considered the mesh sizes of 2mm, 3mm, 4mm, 5mm and 6mm and the dimensions of cantilever beam is 10 cm x 10 cm. Author analysed the models for the Vonmises stresses, deflections and deviation percentage with the 2 mm model with each model respectively and observed that percentage of error for deflection is far less as compared to the stresses due to the concentration at the edge of beams, the stresses and deflections are decreased as the mesh size increased.

Akamadzic et al. (2018) presented the influence of subgrade reaction value of the foundation which is used for soil structure interaction on the 2D & 3D frame analysis. For this purpose author had used different sub grade reaction value like Vesic, Biot, mehrof etc. After calculating the coefficients author modelled the 3-d frame with each coefficient respectively and analysis was done. Author observed the moment values and displacement for symmetrical and unsymmetrical frames and stated deviation of 1 % in moments and 16% in displacements compare to the average one. Higher the subgrade reaction lesser the moments, lesser the displacement and higher stresses, author also concluded that using average value of subgrade reaction gives the optimum results.

Bhatia and Dewangnan(2018) compared the variations in design capacity of axially loaded and flexure elements due to the difference in the design curves in the IRC 112 and IS 456-200. For the study the author selected a bridge deck slab section as flexure member and bridge pier as axially loaded with uniaxial bending. The interaction curves are developed using the MATLAB by refereeing the respective codes. Author concluded that the design capacity of flexure member is more as compared to the older concrete and steel curves, but the variation in P-M interaction curves is not significant.

Solanki and Londhe (2018) studied the effect of confinement on the P-M interaction curves. For this author evaluated RC column strength as per IS 456 2000 and ductility ensured as per IS 13920:2016.The stress strain curve for confined concrete remains above the stress strain curve in IS 456. The strength of RC column under static loading condition in terms of Pu-Mu interaction curve is evaluated for (i) IS 456-2000 model and (ii) Mander stress strain model for confined concrete. A generalized spreadsheet was prepared for the strength evaluation of RC column. It is observed that the strength obtained in terms of P-M curve for confined column of different sizes is 5%-20 % more than that obtained by IS 456-2000.

Wayne (1992) presented the graphs and tables for finding the maximum pressures in the footings when load is eccentric and lift off is occurred. In this book author divided the biaxial case into 4 types according to the amount of eccentricity for each type the formulas and graph is presented for finding the maximum pressures.

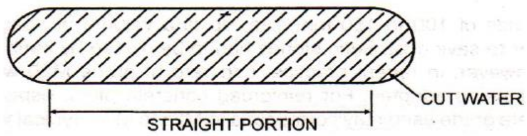
CHAPTER 3

3 PIERS

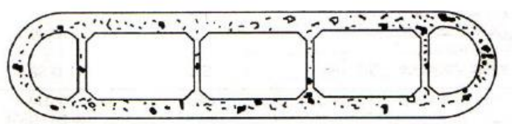
In this chapter the general introduction of piers and procedures for analysis of different loads on pier is discussed. The design procedure for pier according to IRC 112-2011 also covered. Spread sheets for the load calculation and design of pier is developed and discussed. Study on comparison of the P-M interaction curves of IS 456 2000 and IRC 112 2011 is carried out and concluded.

3.1 General

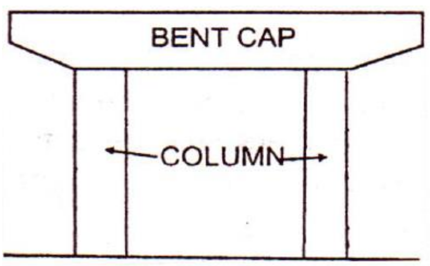
Pier is one of the components of substructure which usually locates at the end of the span or the intermediate point between two abutments. The main function of pier is to transfer the vertical loads from superstructure to the foundation and also to resist all lateral or transfer forces acting on it. Pier cap will be located on the pier which acts as seating to the super structure, this pier cap is provided with some overhang at the edges outside the cross section of pier. There are different types of cross sections for pier shown in Figure 3.1 like circular, rectangle and combine and this section may be solid or cellular type.



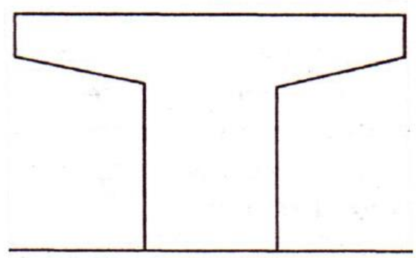
a) Solid pier



b) cellular type pier



c) Trestle RC Pier



d) Hammer-Head Type Pier

Figure 3. 1 Typical shapes of piers

In this study rectangular section of pier is considered and study is carried on it. The different types of piers are used for different conditions like height and span of the bridges.

Minimum width of the pier at the top level is kept at least 600 mm more than the dimension of bearing plate measured in the longitudinal direction of super structure. The length of the pier should be at least 1200 mm more than the dimension of bearing plate measure perpendicular to the longitudinal axis of super structure. The cross section can be constant or can be increased as going down up to 1 in 25 slope such that restricts the net stresses within the permissible limit.

3.2 Analysis of pier

For analysis of pier different types of load should be consider and they are mentioned below. For analysing pier with following loads a spread sheet was developed and shown in.

i. Dead load

This load includes the total weight of the bridge substructure and super structure above the base of pier. It can be calculated as per section 203 from IRC 6-2016.

ii. Live load

This load includes the live load of traffic passing over the bridge and effect of eccentric loading due to live load. It can be calculated as per section 204 from IRC 6-2016.

iii. Buoyancy load

This occurs when the pier portion is submerged in the water. This influence the decrease of the weight of pier, the reduction is done by reducing the density of the pier material by the density of the water. It can be calculated as per section 213 from IRC 6-2016.

iv. Wind load

The wind pressure acting on the pier mainly depends on the location, height, shape, dimension and surroundings of pier, it is designed as per section 209 IRC 6-2016.

The basic wind speed at the location of the bridge site is obtained using the figure shown in clause 209.2 IRC6, using the same clause the Hourly wind speed (V_z) and the wind intensity (P_z) at different heights (H) can be calculated.

The wind force applied on the pier can be calculated as Eq. (1)

$$F_T = P_Z A_1 C_D G \quad (1)$$

Where,

F_T = Force in newton (N) acted on pier

P_Z = Wind intensity $\frac{N}{m^2}$

C_D = Drag coefficient

G = Gust factor

A_1 = Area of plain perpendicular to wind direction.

v. Water current

The portion of the pier which is submerged experiences the water current force on it, and should be designed for this force, these are calculates as per clause 210 IRC 6-2016

The intensity of pressure is shown in Eq. (2)

$$P = 0.52 * KV^2 \quad (2)$$

Where,

P = Intensity of pressure in $\frac{kN}{m^2}$

K= constant depends upon the shape of the cross section of pier clause 210.2 IRC 6-2016

V= velocity of current where pressure is calculated $\frac{m}{sec}$

Water can be hit the pier in any direction that is inclined to the pier usually 20^0 of deviation is taken from the angle of water current hitting the pier provided.

In such case the force resolved in to two directions and applied on the pier

vi. Earthquake load

The earthquake force in horizontal and longitudinal can be calculated per clause 219 IRC 6-2016. The earthquake load need not be checked for following cases.

1. In all seismic zones for minor bridges and culverts up to 10 m span
2. Bridges not exceeding both total length 60 meters and span 15 meters in seismic zone 2 and 3

Special investigation should be carried for bridges having span 150 m and taller than 30 meter in zone 4 and 5, arch bridges, bridges having any innovative arrangements and bridge located at near field regions (10 km radius from known active fault).

The seismic force is calculated as Eq. (3)

$$F = A_h(dead\ load + appropriate\ live\ load) \qquad (3)$$

Where,

F = seismic force resisted

A_h = seismic coefficient

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} \qquad (4)$$

Z= zone factor

I= importance factor

$\frac{S_a}{g}$ = average response acceleration for 5% damping for particular natural time period (T) clause 219.5.1 IRC 6-2016

Illustration:

Force calculation in pier:

Dimensions of pier

b=1m

h=9m

Length = 10 m

Grade of concrete= M30

Grade of steel= Fe500

Assuming weight of vehicle = 700 kN

Basic wind speed at location=44 $\frac{m}{s}$

Angle of water current (θ) =20^0

Maximum mean velocity = 2.2 m/sec

Table 3. 1 Load calculation details

Sr.No	Description	Formula	Value	Unit	Reference
1	Effective length	1.4*height	14	m	IRC 112 Table-11.1
2	Dead load	Weight of pier+ superstructure +pier cap	8000	kN	IRC 6-2016. Section 203
3	Breaking force	20% of weight of vehicle	0.2*700 = 140	kN	IRC 6-2000. Section 214.2
4	Moment due to breaking force	Force X height of vehicle from the base of pier.	140*13.2 = 1848	kN-m	
5	Buoyancy	Density of water X volume of pier submerged	10* (1*9*7)= 630	kN	IRC 6-2016. Section 213
6	Pz @ 10 m for 33 m/s	-	463	$\frac{N}{m^2}$	IRC 6-2016. Section 209
7	Pz @ 10 m for 44 m/s (for given location)	$463*\frac{44^2}{33^2}$	823	$\frac{N}{m^2}$	
8	Wind load	$F_T = P_Z A_1 C_D G$	823.1*10*2*0.9 =14816	N	IRC 6-2016. Section 209
9	Maximum velocity of water(V)	(Maximum mean velocity)*$2^{0.5}$	$2.2*2^{0.5}$ =3.12	m/sec	IRC 6-2016. Section 210
10	Pressure of water current	$P = 0.52 * K\, V^2$	$0.52*1.5*3.12^2$=7.55	$\frac{kN}{M^2}$	IRC 6-2016. Section 210
11	Pressure of water current(+20^o deviation) parallel to depth	$7.55\cos(20+20)^{\ 2}$	4.43	$\frac{kN}{M^2}$	IRC 6-2016. Section 210
12	Pressure of water current(+20^o deviation) perpendicular to depth	$7.55\sin(20+20)^2$	3.11	$\frac{kN}{M^2}$	IRC 6-2016. Section 210

13	Pressure of water current(-20^o deviation) parallel to depth	$7.55\cos(20-20)\ ^2$	7.55	$\frac{kN}{M^2}$	IRC 6-2016. Section 210
14	Pressure of water current(-20^o deviation) parallel to depth	$7.55\sin(20-20)^2$	0	$\frac{kN}{M^2}$	IRC 6-2016. Section 210
15	seismic coefficient longitudinal	$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$	$\frac{016}{2}\frac{1.2}{3}0.71$ = 0.02272		IRC 6-2016. Section 219
16	seismic coefficient transverse	$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$	$\frac{016}{2}\frac{1.2}{3}0.67$ = 0.02272		IRC 6-2016. Section 219
17	Force longitudinal	$A_h\begin{pmatrix} dead\ load + \\ live\ load \end{pmatrix}$	148	kN	IRC 6-2016. Section 219
18	Force transverse	$A_h\begin{pmatrix} dead\ load + \\ live\ load \end{pmatrix}$	151	kN	IRC 6-2016. Section 219

Spread sheet:

For detail force calculation a spread sheet is developed and shown in Figure 3.2, Figure 3.3 ,Figure 3.4 and Figure 3.5.In the spread sheet the yellow cells are for input and the blue cells gives the output values. For each load calculation the force acting on the pier calculated and displayed in spread sheet.

FORCE CALCULATIONS:			
DIMENSIONS:			
CROSS SECTION TYPE:	rectangle		
WIDTH(b) =	1	m	
DEPTH(d) =	9	m	
LENGTH =	10	m	
EFFECTIVE LENGTH	14	m	(IRC 112 TABLE 11.1)
GRADE OF CONCRETE	30	N/mm^2	
GRADE OF STEEL	500	N/mm^2	
Ecm	31	Gpa	
DEAD LOAD:			
PIER =	2250	kN	
SUPERSTRUCTURE=	5000	kN	
PIER CAP =	750	kN	
TOTAL =	8000	kN	
BREAKING FORCE:			
Force= 20% of the weight of design vehicle			(IRC-6 2000 ,214.2)
weight of vehicle	700	kN	
Breaking force	140	kN	
height of road top level from base	12	m	
FORCE acts @ 1.2 m from road base			
Moment at base of pier	1848	kN m	
BUYONCY:			
Area of pier:	9	m^2	
Depth of submerged part of pier:	7	m	
volume of submerged part of pier:	63	m^3	
Net upward force due to buyoncy:	-630	kN	

Figure 3. 2 Spread sheet for force calculations on pier

WIND LOAD

$F_T = P_Z A_1 C_D G$

HEIGHT OF PIER = 10 m

H (m)	Bridge Situated in			
	Plain Terrain		Terrain with Obstructions	
	V_z (m/s)	P_z (N/m²)	V_z (m/s)	P_z (N/m²)
Up to 10 m	27.80	463.70	17.80	190.50
15	29.20	512.50	19.60	230.50
20	30.30	550.60	21.00	265.30
30	31.40	590.20	22.80	312.20
50	33.10	659.20	24.90	373.40
60	33.60	676.30	25.60	392.90
70	34.00	693.60	26.20	412.80
80	34.40	711.20	26.90	433.30
90	34.90	729.00	27.50	454.20
100	35.30	747.00	28.20	475.60

HOURLY MEAN SPEED OF WIND	44	m/s
Pz at height 10 m	463	N/m^2
Pz for 44 m/s	823.111111	N/m^2
A1	10	
G	2	
t/b	9	
H/b	10	
Cd	0.9	

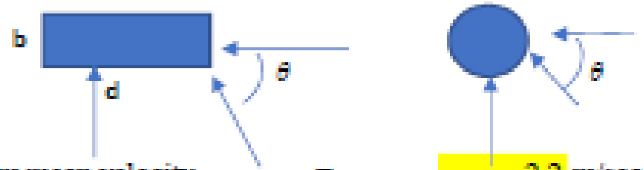
PLAN SHAPE	$\frac{t}{b}$	C_D FOR PIER $\frac{\text{HEIGHT}}{\text{BREADTH}}$ RATIOS OF						
		1	2	4	6	10	20	40
WIND	$\le \frac{1}{4}$	1.3	1.4	1.5	1.6	1.7	1.9	2.1
	$\frac{1}{3}$ $\frac{1}{2}$	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	$\frac{2}{3}$	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	1	1.2	1.3	1.4	1.5	1.6	1.8	2.0
	$1\frac{1}{2}$	1.0	1.1	1.2	1.3	1.4	1.5	1.7
	2	0.8	0.9	1.0	1.1	1.2	1.3	1.4
	3	0.8	0.8	0.8	0.9	0.9	1.0	1.2
	≥ 4	0.8	0.8	0.8	0.9	0.9	0.9	1.1
CIRCLE WITH SMOOTH SURFACE WHERE $tV_z \ge 6$ m²/s		0.5	0.5	0.5	0.5	0.5	0.6	0.6
CIRCLE WITH SURFACE WHERE $tV_z < 6$ m²/s CIRCLE WITH ROUGH SURFACE OR WITH PROJECTION		0.7	0.7	0.8	0.8	0.9	1.0	1.2

F_T = 14816 N 14.816 kN

Figure 3. 3 Spread sheet for wind force calculations on pier

Water current forces:

angle of watter current(θ)		20	^{0}
solving for deviation of+20 degrees with provided angle			
Depth of submerged part of pier:		7	m
exposed area:(parallel to d)		7	m^2
exposed area:(perpendicular)		63	m^2



Maximum mean velocity	=	2.2	m/sec
Maximum velocity	=	3.11127	m/sec

$P = 0.52 * K\, V^2$

Pressure intensity P	=	7.5504	kN/m^2
Pressure parallel to depth	=	4.43339	kN/m^2
Pressure perpendicularto depth	=	3.11701	kN/m^2
Force parallel to depth	=	15.5169	kN
Force perpendicular to depth	=	98.1859	kN

solving for deviation of-20 degrees with provided angle

Depth of submerged part of pier:		7	m
permeter of exposed area:(parallel to d)		7	
permeter of exposed area:(perpendicular)		63	
Maximum mean velocity	=	2.2	m/sec
Maximum velocity	=	3.11127	m/sec
Pressure intensity P	=	7.5504	kN/m^2
Pressure parallel to depth	=	7.5504	kN/m^2
Pressure perpendicularto depth	=	0	kN/m^2
Force parallel to depth	=	26.4264	kN
Force perpendicular to depth	=	0	kN

FORCE ACT @2.34 FROM BASE

Figure 3. 4 Spread sheet for water current force calculations on pier

EARTH QUAKE				
Dead load		6500	kN(DL+(DL OF PIER/3)	
20 % live load max reaction(excl impact)			500	kN
		TRANSVER:	LONGITUDINAL	
I(MOI)	=	60.75	0.75	m^4
CRACKED I	=	45.5625	0.5625	m^4
mass in longitudinal	=		662589	kg
mass in transverse	=		713558	kg
K longitudinal	=	52312.5	N/mm	
K transverse	=	4237312.5	N/mm	
	TIME PERIC	sa/g		
Longitudinal	0.70677158	0.71		
transverse	0.08149462	0.67		
R		3		
IMPORTANCE FACTOR		1.2		
Z		0.16		

$$F = A_h\,(dead\ load + appropriate\ live\ load)$$

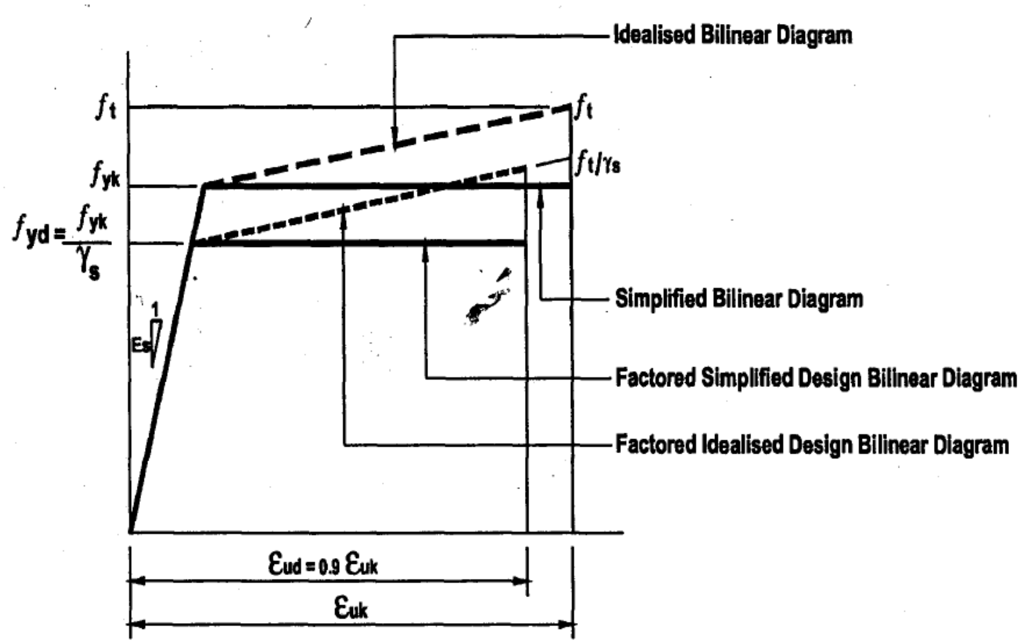
$$A_h = \frac{Z}{2}\,\frac{I}{R}\,\frac{S_a}{g}$$

Fh longitudinal	=	147.68	kN
Fh transverse	=	150.08	kN
MOMENT(ML)	=	1500.8	kN m
MOMENT(MT)	=	1476.8	kN m

Figure 3. 5 Spread sheet for earthquake force calculations on pier

3.3 Design of pier

The Pier is design as column referring IRC 112 -2011, the stress strain curve consider for the steel is bilinear stress strain as shown in Figure 3.6.



Where,

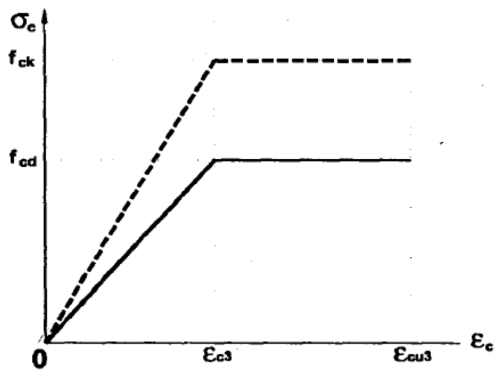
$f_{yk}\ = Yeild\ stress\ of\ steel$

$\varepsilon_{uk} = strain\ in\ steel$

$\gamma_s = factor\ of\ safety$ 1.15

Figure 3. 6 Bilinear stress strain diagram for reinforcement steel

The stress strain curve used for concrete is considered by referring the IRC 112-2011 which considers it as bilinear stress strain relation as shown in Figure 3.7. And the stress block for the compression is considered as rectangular block as shown in Figure 3.8.



Where,

$f_{ck} \; = Strength \; of \; concrete$

$\varepsilon_{cu3} = Ultimate \; strain \; in \; concrete$

$f_{cd} \; = \frac{f_{ck}}{1.5}$

Figure 3. 7 Bilinear Stress Strain curve for concrete

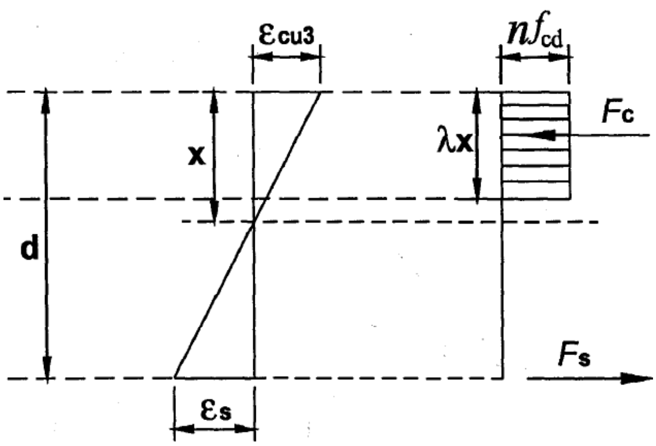


Figure 3. 8 Rectangular stress block for concrete

Where,

$\lambda = 0.8 \, , \eta = 1 \; for \; f_{ck} \leq 60 \; MPa$

3.3.1 Construction of M-N curves for uniaxial bending:

The construction of the M-N curves is done referring Bhatt et al (2014) which considered design as the IRC 112-2011 only difference is, it is done with respective to the cylindrical strength and IRC 112 2011 considers the cube strength of the concrete, the design procedure is shown below.

N = Axial force

M = Moment

X = Neutral axis depth

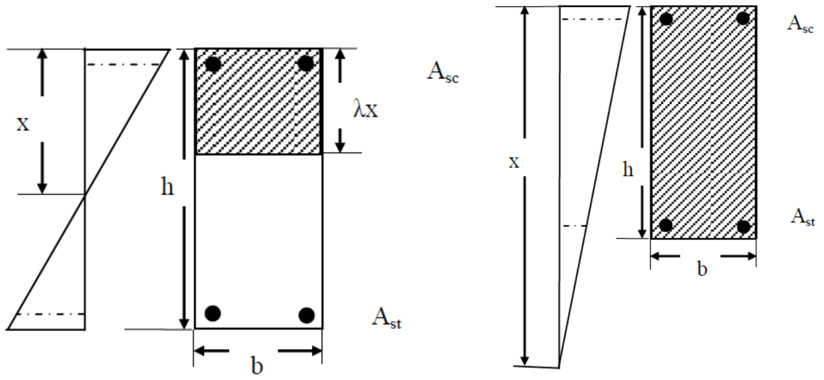
b= Width

h = Depth

A_{st} = Reinforcement in tension zone

A_{sc} = Reinforcement in compression zone

Depending on the values of the M & N the neutral axis may or may not lie within depth of section



Case 1: partially in compression Case 2: fully compression

Figure 3. 9 Cross section with position of neutral axis

Case 1: $\boldsymbol{\lambda x < h}$

The neutral axis lies within the cross section the compression force (Cc) and moment (Mc) at centre due to the concrete in compression zone can be calculated as Eq. (4)

$$\frac{C_C}{bh} = \alpha_{cc}\eta\lambda f_{cd}\frac{x}{h} \quad (4)$$

$$\frac{M_c}{bh^2} = 0.5\alpha_{cc}\eta\lambda f_{cd}\frac{x}{h}\left(1-\lambda\frac{x}{h}\right) \tag{5}$$

Where,

$f_{cd} = \frac{fck}{1.5}$

$\alpha_{cc} = 1$

Case 2: $\boldsymbol{\lambda x > h}$

The neutral axis lies outside the cross section the compression force and moment at centre can be commutated as Eq. (6)

$$\frac{C_C}{bh} = \alpha_{cc}\eta f_{cd} \tag{6}$$

$$\frac{M_c}{bh^2} = 0 \tag{7}$$

Where,

$f_{cd} = \frac{fck}{1.5}$

Generating curve for the cross-section shown in Figure 3.10 having reinforcement along the perimeter of the cross section.

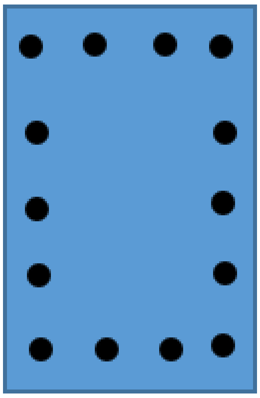


Figure 3. 10 Cross section example

Stresses and strains at different layers of the steel:

Let the distance of each steel layer from top fibre be d' respectively, the ultimate strain in concrete $\varepsilon_{uc3} = 0.0035$

$$\text{strain}(\varepsilon_s) = 0.0035\left(\frac{\text{x}-\text{d}'}{\text{x}}\right) \tag{8}$$

$$\text{stress} = 2 * 10^5 * \text{strain} \ \frac{\boldsymbol{N}}{\boldsymbol{mm^2}}$$

Stress in the steel is limited to the $\frac{f_y}{1.15}$

Force and moment for respective layer of steel respectively is given in Eq. (9a) &Eq. (9b)

Force = stress * area of steel (9a)

Moment = Force (0.5*h – d’) (9b)

Conditions for steel whether they are in compression or tension & negative or positive moment:

1. If the location steel is above x then it has compression force and below x it has tension
2. If the compression steel is below the 0.5*h then it gives negative moment and tension steel lies above 0.5* h it gives negative moment.

Total axial force = Compression force of concrete and steel −Tension of steel

Total moment at mid depth = Moment due to concrete compression + Moment due to Steel (Compression and tension)

The forces for each layer of steel should be calculated for each respective value of $\frac{x}{h}$

The whole process is iterated for each value of $\frac{x}{h}$

The sum of internal forces is:

$$\frac{N}{bh} = \frac{Cc}{bh} + \sum \frac{A_S}{bh} f_s \tag{10}$$

Where,

A_S & f_S are the area of steel and stress in the steel at layer respectively.

$$\frac{M}{bh^2} = \frac{M_C}{bh^2} + \frac{A_S}{bh^2}\,\text{K} \tag{11}$$

Where,

K= distance of respective layer from the centre of the cross section.

Using above equations, one cannot design a section directly to carry load and moment so therefore one should assume a trial section and % of reinforcement should be checked whether it is safe or not with the constructed charts.

ILLUSTRATION:

Taking a cross section which has symmetric reinforcement consisting of each layer on either side of mid-point.

% of steel = 4%

Each layer = 2%

Grade of concrete = 30

Grade of steel = 500

As the equations developed with respective cylindrical strength take f_{ck} = 0.8*30 = 24

$\frac{d'}{h}$ = 0.05 for first layer

$\frac{d'}{h}$ = 0.95 for second layer

Table 3. 2 Calculations in developing P-M curves

Sr.No	Description	Formula	Value	Unit
1	compression force of concrete	$\frac{C_C}{bh} = \alpha_{cc}\eta\lambda f_{cd}\frac{x}{h}$ For $\frac{\lambda x}{h} \leq 1$	$1 * 1 * 16 * \frac{\lambda x}{h}$ $=16\frac{\lambda x}{h}$	$\frac{N}{mm^2}$
2	compression force of concrete	$\frac{C_C}{bh} = \alpha_{cc}\eta\lambda f_{cd}\frac{x}{h}$ For $\frac{\lambda x}{h}$ >1	$1 * 1 * 16$ =16	$\frac{N}{mm^2}$
3	Moment due to Concrete	$\frac{M_c}{bh^2} = 0.5\alpha_{cc}\eta\lambda f_{cd}\frac{x}{h}\left(1-\lambda\frac{x}{h}\right)$ for $\frac{\lambda x}{h} \leq 1$	$=0.5 * 1 * 1 * 16 * \frac{\lambda x}{h}\left(1-\lambda\frac{x}{h}\right)$ $=8 * \frac{\lambda x}{h}\left(1-\lambda\frac{x}{h}\right)$	$\frac{N}{mm^2}$
4	Moment due to Concrete	$\frac{M_c}{bh^2} = 0$ for $\frac{\lambda x}{h}$ >1	0	$\frac{N}{mm^2}$
5	Stress in steel in first layer	E* Strain	$2*10^5*0.0035*\left(1-\frac{h}{h}\frac{d\prime}{x}\right)$	$\frac{N}{mm^2}$
6	Stress in steel in second layer	E* Strain	$2*10^5*0.0035*\left(\frac{h}{x}\frac{d\prime}{h}-1\right)$	$\frac{N}{mm^2}$

From above steps we can see that the all forces and moments totally depends on $\frac{x}{h}$ values

Therefore, by keeping the values of $\frac{x}{h}$ from 0 to 3 we get the $\frac{N}{bh}$ and $\frac{M}{bh^2}$ respectively from that values we can plot the graph $\frac{N}{bh}$ Vs. $\frac{M}{bh^2}$

Taking a cross section of 300 X 450 mm column with axial load 1000 kN and moment 200 kN m and with above mentioned criteria we get the P-M curve as shown in Figure 3.11. The point shown in the graph is located inside the curve thus it shows that the column with given load is acceptable and safe.

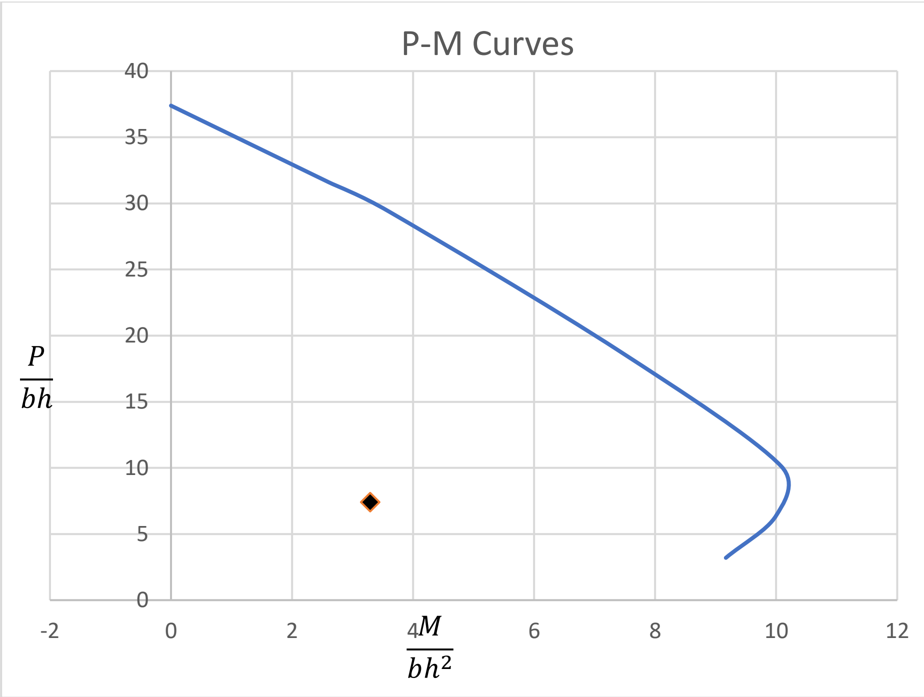


Figure 3. 11 P-M Interaction curve

3.3.2 Biaxial bending

1. When a section should be designed for axial and biaxial bending care must be taken in selecting the critical combination for getting the moments.
2. First step for biaxial bending is to do separate design in each principal direction, and when there is an unfavourable effect imperfection should be taken care.
3. No further check is necessary if the section satisfies the following conditions.

$$\frac{\lambda_y}{\lambda_z} \le 2 \quad \& \quad \frac{\lambda_y}{\lambda_z} \le 2 \tag{12}$$

And

$$\frac{e_y}{d}\Big/\frac{e_z}{b} \le 0.2 \ \ OR \ \ \frac{e_z}{b}\Big/\frac{e_y}{d} \le 0.2 \tag{13}$$

Where,

Z, Y = Two principal actions of cross section.

$\lambda_{z,}\lambda_y$ = slenderness ratios of respective axis direction

e_z, e_y = Eccentricities for axial load in respective moment directions.

4. If the above conditions stated is not fulfilled then bi axial bending should be taken account, the following simplified criteria can be used for checking bi axial bending.

$$\left(\frac{M_{EDY}}{M_{RDY}}\right)^{\propto} + \left(\frac{M_{EDZ}}{M_{RDZ}}\right)^{\propto} \leq 1 \qquad (14)$$

Where,

M_{EDY}, M_{EDZ} = Design moments in respective axis.

M_{RDY}, M_{RDZ} = Moment resistance in respective axis.

$\propto = 1$ For circular and elliptical sections

For rectangular section:

N_{ED}/N_{RD}	0.1	0.7	1
$\propto$	1	1.5	2

With linear interpolation for intermediate values

Where

N_{ED} = design value of axial force.

$$N_{RD} = A_C f_{cd} + A_s f_{yd} \qquad (15)$$

A_C = Gross area of concrete section.

A_s = Area of longitudinal reinforcement.

3.4 Illustration

A Pier of 1 X 9 metre which is subjected to axial load and biaxial moments in each axis respectively values are shown below

Axial force= 8000 kN

Moment about width = 2500 kN-m

Moment about depth = 1000 kN m

F_{ck} = 30 MPa

F_y =500 MPa

As preliminary step 0.5% steel is assumed and arranged in a symmetric and along the perimeter of the cross section

$A_{st} = \frac{0.5}{100} * 1 * 9 * 1000 * 1000 = 45000\ mm^2$

Assuming 92 number of bars of 25 mm diameter are arranged such that spacing along width direction is 95 mm and spacing along depth direction is 232 mm.

Spread sheet:

A spread sheet is developed using VBA code shown in Annexure A for extracting the P-M curve for the given input and also for the uniaxial and biaxial checks. For the above example values, the calculations and checks that are done in excel sheet is shown in Figure 3.13 .From the graphs developed in excel sheet one can know the given pier with respective loads is safe or not for uniaxial bending using position of point.

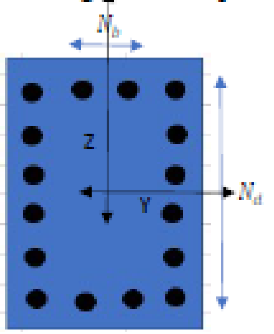
If the given pier not satisfies the checks for biaxial bending a message displays saying to revise the dimension of pier. It also gives message whether biaxial check is required or not for the section assumed.

The Figure 3.14 and Figure 3.15 shows the uniaxial capacity curves with the point which shows the applied combination in each axis respectively. It can be seen that the point is within curve that states that the cross section is safe in both axis for uniaxial bending, the further checks in biaxial are shown in Figure 3.13.

Checks for Pier

AXIAL FORCE	=	8000	kN
MOMENT ABOUT Z	=	1000	kN m
MOMENT ABOUT Y	=	2500	kN m
WIDTH	=	1	m
DEPTH	=	9	m
Assume % of steel	=	0.5	%
Ast	=	45000	mm^2
Dia of bar provided	=	25	mm
bars required	=	91.71974522	
bars provided	=	92	

bars arrangement example:



N_b	=	8	
N_d	=	38	
cover(mm)	=	75	
spacing (mm)	=	95	(between layers in y direction)
spacing (mm)	=	232	(between layers in z direction)

slenderness ratios

λ_y	=	16.16580754		e_y	=	0.125	
λ_z	=	48.49742261		e_z	=	0.3125	
$\frac{\lambda_y}{\lambda_z}$		0.333333	OK	$\frac{e_y}{d} / \frac{e_z}{b}$	=	0.044444	OK
$\frac{\lambda_z}{\lambda_y}$		3	NOT OK	$\frac{e_z}{b} / \frac{e_y}{d}$	=	22.5	

Need to biaxial check

CHECK FOR UNIAXIAL

refer sheet Y for uniaxial about y axis

refer sheet Z for uniaxial about z axis

CHECK FOR BIAXIAL

N_{RD}	=	163535	kN
M_{RDY}	=	43645.01552	kN m
M_{RDZ}	=	7554.236223	kN m
N_{ED} / N_{RD}	=	0.048919192	
α	=	1	
$\left(\frac{M_{EDY}}{M_{RDY}}\right)^{\alpha} + \left(\frac{M_{EDZ}}{M_{RDZ}}\right)^{\alpha}$	=	0.189656366	

BIAXIAL CHECK IS SATISFIED

Figure 3. 12 Spread sheet for uniaxial and biaxial checks

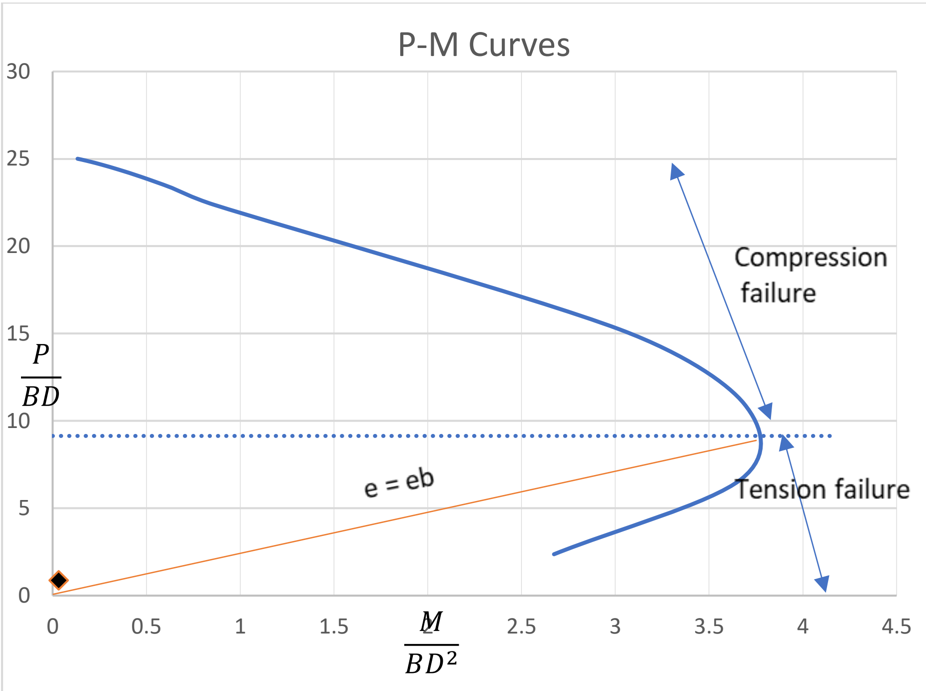


Figure 3. 13 P-M Curve of uniaxial bending about depth

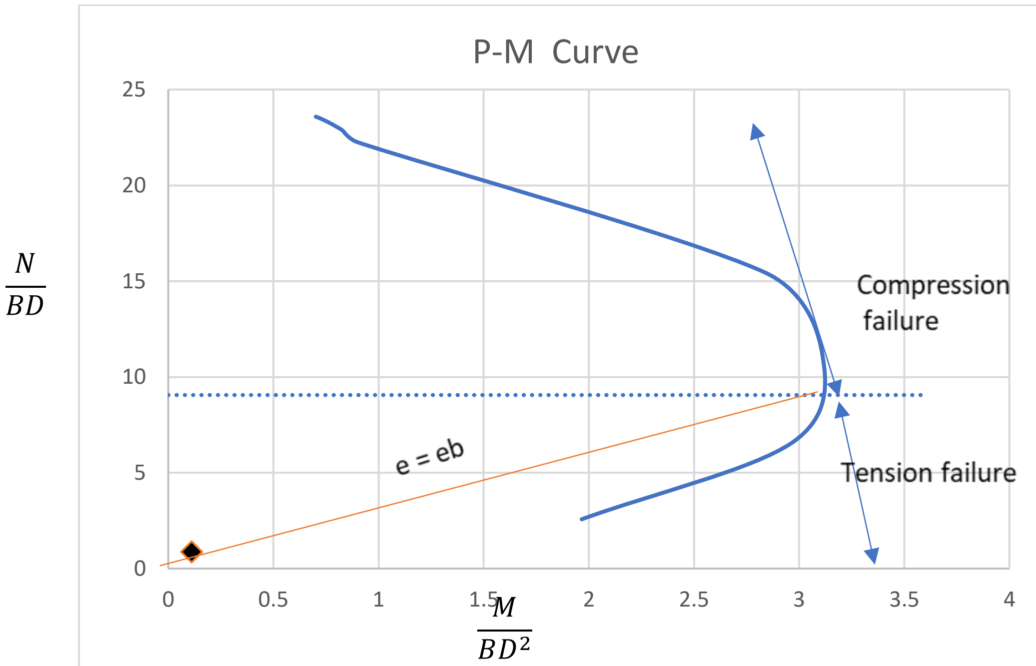


Figure 3. 14 P-M Curve of uniaxial bending about width

In the both the P-M curves in each direction respectively shows that the point is inside the curve such that it states the pier with respective cross section is safe for applied bending in both directions, the orange line shows the point of balanced section ($e=e_b$) if point lies below it states $e > e_b$ and if point is above orange line it states $e < e_b$.

3.5 Comparison of P-M curves

Referring SP-16 charts and IRC 112-2011 code using prepared excels the following example was taken and the respective curves was built and compared.

In SP-16 the charts given are with respective to the F_y and $\frac{d\prime}{D}$ ratio and each graph the curves are plotted with $\frac{P}{F_{ck}BD}$, $\frac{M}{F_{ck}BD^2}$ and $\frac{\rho}{F_{ck}}$ respectively, but graphs developed using IRC 112 with excel VBA was plotted using $\frac{P}{BD}$ and $\frac{M}{BD^2}$ for respective F_{ck}, F_y and percentage of steel%, so the values in the SP-16 are converted to the values respective to IRC 112 for following example and converted.

Example:

Taking a cross section with $\frac{d\prime}{D} = 0.05$, Percentage of steel% = 4% of cross section area,

F_{ck} =30 MPa (cylinder), 37.5 MPa (cube), grade of steel Fe 500.

With the above data Chart 35 compression with bending form SP16 is selected.

$\frac{\rho}{F_{ck}} = \frac{4}{37.5}$ approximately equal to = 0.11

For respective $\frac{\rho}{F_{ck}}$ =0.11 the values are interpolated in between 0.10 and 0.12

The values $\frac{P}{F_{ck}BD}$ and $\frac{M}{F_{ck}BD^2}$ are converted to $\frac{P}{BD}$ and $\frac{M}{BD^2}$ at selected points manually and $\frac{P}{BD}$ values for each $\frac{M}{BD^2}$ respectively of IRC 112 are extracted from excel prepared, the graph showing two curves shown in below Figure 3.12.

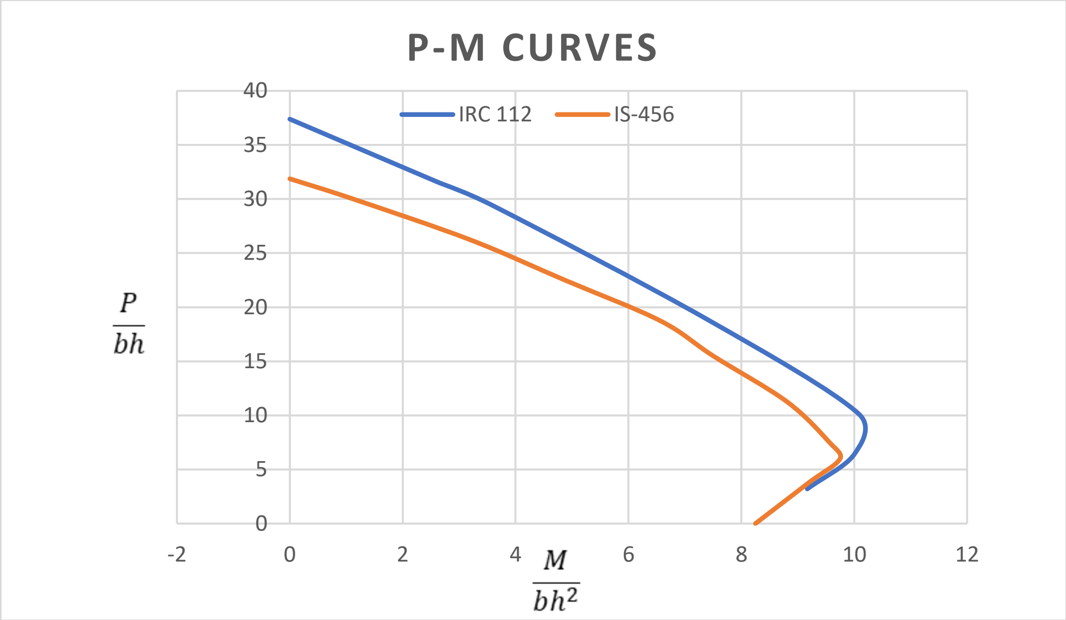


Figure 3. 15 P-M Curve of IS 456-2000 and IRC 112-2011

From the comparison curve shown in the Figure 3.12 it can be seen that the curve of IS 456-2000 and IRC 112 2011 are almost similar and can be noticed no significant changes. But the curve developed based on IRC 112-2011 is on the higher side of the IS-456-2000 curve.

CHAPTER 4

4 WELL FOUNDATION

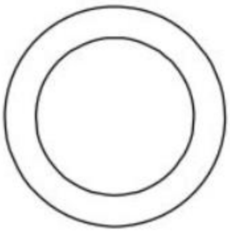
4.1 General

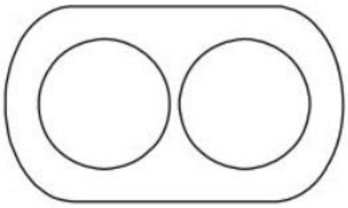
Well foundations is used as deep foundations in many buildings and bridges that are built hundreds of years ago in india. These were used for bridges across the major rivers during the mughal period. Well foundation is used in one of the famous monument taj mahal. It is a massive and solid structure. Well foundation will have very high section modulus for a given cross sectional area. It is relatively rigid in its structural behaviour and monolithic.

4.2 Types of well foundations

The shape and size of well foundation mainly depends on the shape and size of pier resting on it. The cost, ease to sunk, forces to be resisted, tilts and shifts are the other considerations for selection of shape of well foundation. The different types of well foundations are shown in Figure 4.

Circular well foundations shown in Figure 4.1a are most commonly used because of the ease of construction, ease of sinking and due to its high strength. But only disadvantage is it offers less lateral resistance against tilting for given cross section compared to other sections. For supporting the two piers a tied well can be used. Most commonly used tied well is double-D shape shown in Figure 4.1d. The other tied well foundation are dumb bell and double circular shown in Figure 4.1 b and 4.1c respectively. Rectangular wells are used when bridge foundation having hallow depths.





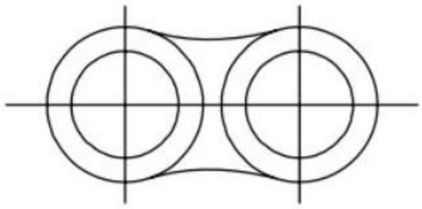
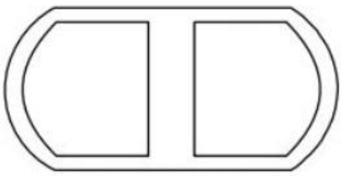
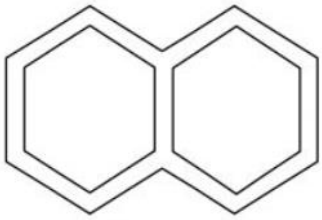


Figure 4.1a circular Figure 4.1b Double circular Figure 4.1c Dumb-bell





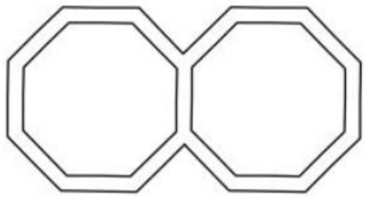


Figure 4.1d Double -D Figure 4.1e Twin Hexagonal Figure 4.1f Twin Octagonal

Figure 4. 1 Types of well foundations (Jagadeesh and Jayaram 2009)

4.3 Elements of Well foundation

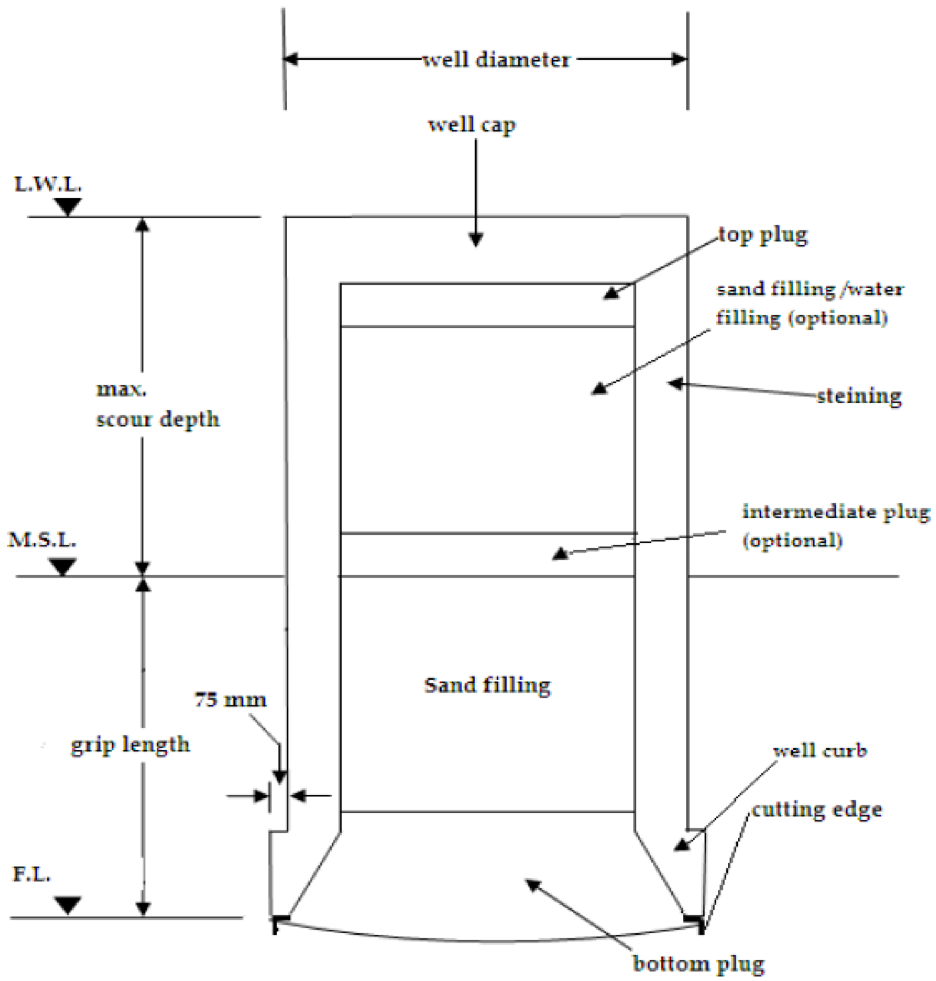


Figure 4. 2 Elements of well foundation (Jagadeesh and Jayaram 2009)

4.4 Analysis and design of well components

4.4.1 Maximum scour depth:

The formula for calculating the mean depth of scour according to IRC 78-2014 below high flood level (HFL) for natural channels flowing over scorable bed is as Eq. (16).

$$d_{sm} = 1.34\left(\frac{{D_b}^2}{K_{sf}}\right)^{\frac{1}{3}} \tag{16}$$

Where,

D_b = Design discharge per meter width, $\frac{m^3}{ms}$

$D_b = \frac{Q}{L}$, Q is the design discharge in $\frac{m^3}{m}$ and $L = 4.76 * Q^{\frac{1}{2}}$ is the linear waterway, m

K_{sf} =Silt factor

= 1.76, d_m is the median size of the bed sediments in mm.

The normal scour depth for natural streams in alluvial beds can also be calculated using Lacey's formula Eq. (17):

$$d = 0.473\left(\frac{Q}{f}\right)^{\frac{1}{3}} \tag{17}$$

Where,

d =Normal depth of scour, m

Q =Designed discharge, $\frac{m^3}{s}$

f =Lacey's silt factor of the bed material.

The mean scour depth for design of foundation d_{sm} is the maximum of Eq. (16) and Eq. (17) as of IRC- 78 – 2014, at the noses of piers, the maximum depth of scour d_{max} is taken as twice of mean scour depth d_{sm} Eq. (18).

$$d_{max} = 2 * d_{sm} \tag{18}$$

The depth of well foundation should be taken such that it is safe against scour. Other factors to be considered for depth are settlement stability, suitability of strata and bearing capacity. In all cases the depth of well foundation should satisfy the grip length. Usually the grip length below the anticipated maximum scour level shall not be less than $1/3^{rd}$ the maximum anticipated depth of scour below H.F.L.

4.4.2 Loads for design

The following loads are considered for the analysis and design of well foundation:

1. Dead load
2. Live load
3. Buoyancy
4. Wind load
5. Horizontal force due to water current
6. Centrifugal forces
7. Longitudinal forces
8. Seismic forces
9. Horizontal shear forces at bearings due to longitudinal forces and seismic forces
10. Forces due to tilt and shift.

4.4.3 Design of well steining

The Stresses in the steining are checked at the level of maximum scour as per Eq. (19a) & Eq. (19b) before designing it.

$$\sigma_1 = \frac{W}{A} + \frac{M}{Z} \tag{19a}$$

$$\sigma_2 = \frac{W}{A} - \frac{M}{Z} \tag{19b}$$

Where,

$W =$ Total vertical load acting up to the maximum scour depth,

$A =$ Area of cross-section of well steining,

$M =$Resultant moment due to various loads as considered during analysis of well at maximum scour level

$Z =$ Section modulus of well steining.

The stresses should be within the permissible limits. If the stresses exceed the permissible limits, the thickness of the well steining as per Eq. (20) has to be increased.

$$t = KD\sqrt{L} \tag{20}$$

Where,

$t =$Minimum thickness of concrete steining, m,

$D =$External diameter of circular well

$L =$Depth of well in m, below L.W.L. or top of well cap whichever is greater,

$K =$a constant depending on the nature of subsoil and steining material (taken as 0.30 for circular well for concrete steining in sandy strata and 10% more than the corresponding value in the case of clayey soil).

After performing the checks for stresses and thickness of steining, the reinforcements in the steining are calculated. The vertical reinforcements in the steining should not be less than 0.12 percent of the gross sectional area of the actual thickness provided for the steining. The vertical reinforcement should be equally distributed on both the faces of the steining. The vertical reinforcement should be tied up with hoop steel not less than 0.04 percent of the volume per unit length of the steining.

4.4.4 Design of well curb

The curb cuts through the soil under the action of the dead weight of the steining including kentledge when the well is dredged during the process of sinking. In this process hoop tension develops in the well curb. To resist the hoop tension reinforcement should be provided in form of rings along the perimeter in the well curb.

$$T = 0.75N \left(\frac{\sin\theta - \mu\cos\theta}{\mu\sin\theta + \cos\theta}\right) d \qquad (21)$$

Where,

N =Running load of the well steining on the curb,

d =Mean diameter of well steining,

θ =Angle of bevelled edge of well curb with horizontal, and,

μ =Coefficient of friction between soil and concrete of curb.

A minimum reinforcement of 72 $\frac{Kg}{m^3}$ is provided in the well curb.

4.4.5 Design of bottom plug

The minimum thickness should be provided for bottom plug is shown in Eq. (22) and Eq. (23).

$$t^2 = 1.18 * r^2 \frac{q}{f_c} \quad \text{For circular wells} \qquad (22)$$

$$t^2 = \frac{3qb^2}{4f_c(1+1.61\alpha)} \quad \text{For rectangular wells} \qquad (23)$$

Where,

r =Radius of well at the base

q =Unit bearing pressure against the base of the well

f_c =Flexural strength of concrete

b =Short dimension of well

α = Ratio of Short side and long side of well.

4.4.6 Design of well cap

A well cap transfer the loads and moments from the pier to the well. The shape of the well cap is taken same as of the well with a minimum overhang of 150 mm. The well cap is designed as a two-way reinforced concrete slab resting over the top of well. The support conditions are taken partially restrained.

The design of the well cap is carried out by assuming that the load from the pier acts on an imaginary circle having an area equal to the area of dispersion of the loads transferred from the pier to the well cap. Since the well-cap is assumed to be partially restrained by the steining, the moments in the well-cap are calculated for circular patch loading and for U.D.L. (self-weight of well cap) for the following two conditions.

1. Freely supported well cap on steining.
2. Fully clamped well cap on steining.

Well cap freely supported on steining:

Take,

ϑ =Poisson's ratio of concrete,

w =Weight of well cap per unit area

V =Vertical load acting on the well cap

h =Effective diameter of well cap,

M_t and M_r are the tangential and radial moments in well-cap, respectively.

In the first instance, the moments in the well cap due to vertical loads transferred from the pier and the self-weight of the well cap are determined as per following steps.

- Moments beneath loaded area due to circular patch loading

$$M_r = \frac{V}{4\pi}\left[1 + (1+\vartheta)\ ln\left(\frac{h}{d}\right)\right] \quad (24)$$

$$M_t = \frac{V}{4\pi}\left[1 + (1+\vartheta)\ ln\left(\frac{h}{d}\right)\right] \quad (25)$$

$d =$ Diameter of equivalent circular patch loading

- Moments beneath unloaded area due to circular patch loading

$$M_r = -\frac{V}{4\pi}[(1-\vartheta)\ ln(\xi)] \quad (26)$$

$$M_r = -\frac{V}{4\pi}[(1-\vartheta)\ -(1+\vartheta)\ ln(\xi)] \quad (27)$$

At support, $d =$ h; $\xi = \frac{d}{h} = 1$

The radial and tangential moments in the well cap due to U.D.L. are given by

$$M_t = \frac{wh^2}{64}(3+\vartheta)\ -(1+3\vartheta)\ \xi \quad (28)$$

$$M_r = \frac{wh^2}{64}(3+\vartheta)\left[1-\left(\frac{\xi}{h}\right)^2\right] \quad (29)$$

At centre, $d = 0$; $\xi = \frac{d}{h} = 0$

At support, d =h $\xi = \frac{d}{h} = 1$

Well cap fully clamped at support

- Moments beneath loaded area due to circular patch loading

$$M_r = \frac{V}{4\pi}\left[(1+\vartheta)\ ln\left(\frac{h}{d}\right)\right] \quad (30)$$

$$M_t = \frac{V}{4\pi}\left[(1+\vartheta)\ ln\left(\frac{h}{d}\right)\right] \quad (31)$$

d = diameter of equivalent circular patch loading

- Moments beneath unloaded area due to circular patch loading

$$M_r = \frac{V}{4\pi}\left[\left(\frac{d}{2\xi h}\right)^2(1-\vartheta) - (1+\vartheta)\ ln(\xi) - 1\right] \quad (32)$$

$$M_t = \frac{V}{4\pi}\left[\left(\frac{d}{2\xi h}\right)^2(1-\vartheta) - (1+\vartheta)\ ln(\xi) - 1\right] \quad (33)$$

At support, d = h; ξ= d/h = 1

The radial and tangential moments in the well cap due to U.D.L. are given by

$$M_r = \frac{wh^2}{64}[(1+\vartheta) - (3+\vartheta)\ \xi^2] \quad (34)$$

$$M_r = \frac{wh^2}{64}[(1+\vartheta) - (3+\vartheta)\ \xi^2] \quad (35)$$

At centre, d = 0; ξ= d/h = 0

At support, d = h; ξ= d/h = 1

If M_1 is the resultant moment per meter length of the pier, then maximum reactive moment at the support =$\pm\frac{M1}{4} * 0.5 = \pm\frac{M1}{8}$

Hence, the maximum moment at the centre of the well cap due to moments Transferred form pier =$\pm\frac{5M1}{8}$

The resultant moments for the design of the well-cap section at mid-span and at Supports can be found out as follows.

Centre = Mean radial moment due to patch loads beneath the loaded area

+ Mean radial moment due to U.D.L. at the center of well-cap

+ Moment at the center of well cap due to moments transferred from pier

M_{edge} = Mean radial moment due to patch loads beneath unloaded area

+ Mean radial moment due to U.D.L. at the support of well-cap

+ Moment at the edges of well cap due to moments transferred from pier

The reinforcement at the center of the well-cap is calculated for the moment M_{centre} and the reinforcement at the edges of well-cap is calculated for the moment M_{edge}.

The well-cap is finally checked for punching shear as per IS 456-2000

4.5 Stability analysis

IRC 45-1972 recommends to check the surrounding soil resistance of well using:

1. Elastic theory
2. Ultimate resistance (plastic theory)

Elastic theory:

This theory includes the following steps:

1. Determine the Vertical load (W), Lateral load (H) and External moment (M) including self-weight and tilts and shifts for respective calculations.

2. Calculation of Moment of inertia I, I_B, I_V .
   Where,

   I_B = Moment of inertia of base normal to horizontal force

   I_V = Moment of inertia projected area of elevation for soil resistance

   $I = I_B + m\, I_V(1+2\mu\alpha)$

   m= ratio of lateral to vertical sub grade reaction of soil = $\frac{K_h}{K_v}$

μ = coefficient of friction between wall and foundation

$\alpha = \frac{B}{2D}$ for rectangle

$= \frac{diameter}{2D}$ for circular

D = depth of well below scour level.

3. Check for following:

$$H > \frac{M}{r}(1 + \mu\mu') - \mu W \tag{36}$$

$$H < \frac{M}{r}(1 - \mu\mu') + \mu W \tag{37}$$

Where

$r = \frac{D}{2}\frac{1}{mI_v}$

μ' = coefficient of friction between wall and foundation

4. Check for Elastic state

$$\frac{mM}{I} < \gamma(K_P - K_A) \tag{38}$$

γ = density of soil

K_P, K_A = Passive and Active earth pressures.

5. Calculate σ_1, σ_2

$$\sigma_{1,2} = \frac{w - \mu' p}{A} \pm \frac{MB}{2I} \tag{39}$$

$\sigma_{1,2}$ = Maximum and Minimum base pressures respectively,

A = area of base of well

B= width of base of well in direction of forces and moment

$P = \frac{M}{r}$

Check, σ_1 < safe bearing capacity of soil

$\sigma_2 > 0$ (no tension)

Ultimate resistance (plastic theory):

1. Check:

$$\frac{W}{A} < \frac{\sigma_u}{2} \tag{40}$$

σ_u= ultimate bearing capacity of soil

W = total vertical load for different load combinations

2. Calculation of base resisting moment M_b

$$M_b = QWB \tan \emptyset \tag{41}$$

B = width in the direction of forces

$\emptyset$ = internal friction of soil

3. Calculation of ultimate moment resistance of well sides due to passive resistance of soil M_s

$$M_s = 0.10\gamma D^3(K_P - K_A)L \tag{42}$$

L = projected width for soil resistance

4. Calculate ultimate moment resistance due to friction M_f

For rectangle:

$$M_f = 0.18\gamma(K_P - K_A)LBD^2 \sin \delta \tag{43}$$

For circular:

$$M_f = 0.11\gamma(K_P - K_A)B^2D^2 \sin \delta \tag{44}$$

5. Ultimate moment resistance M_t

$$M_t = 0.7\left(M_b + M_s + M_f\right) \tag{45}$$

Check $M_t > M$

M = total applied external moment about the plane of rotation of the well taking appropriate load combinations

4.6 Illustration

A Well foundation is to be designed for an abutment of 7m X 5 m base dimensions. The well is founded on a sandy soil. The data available are as follows:

Height of bearing above the max scour level: 16 m

Permissible horizontal displacement of the bearing level: 50 mm

Height of the abutment: 6 m

Total vertical load at the scour level = 20000 kN

Total horizontal load at the scour level = 20000 kN

Submerged unit weight of soil: 9.5 kN/m^3

Solution:

Table 4. 1 Calculations in design of well foundation

Sr.No	Description	Formula	Value	Unit	Reference
1	Diameter of well(D)	Take according to the abutment size	7.5	metre	
2	Grip length	$\frac{\underline{\textit{Height of well abve scour level}}}{3}$	$\frac{10}{3} = 3.33$ Assuming 8	metre	Jagadesh and

					jayaram(2009)
3	Total length(L)	Grip length+ Height of well above scour level	=10+8 = 18	metre	
4	Check for stresses in steining	$\frac{P}{A}+\frac{M}{I}Y$	= 0.68 <0.7*$\sqrt{30}$	$\frac{N}{mm^2}$	
5	Thickness of steining (h)	$KD\sqrt{L}$	=0.03*10* $\sqrt{30}$ = 1.1	m	IRC 78-2014 Section 708.2
6	Vertical reinforcement	0.15% of cross section	71 no bars of 20 mm dia@ 420 mm c/c		IRC 78-2014 Section 708.2
7	Hoop reinforcement	0.05% of 1 m volume of steining	5 loops of 12 mm dia@ each face per meter.		
8	Bottom plug thickness	$t^2 = 1.18 * r^2 \frac{q}{fc}$	$=1.18 * 3.75^2 \frac{20000}{66.72*3834}$ = 1.71	m	IRC 78-2014 Section 708.8
9	Reinforcement in well cap for sagging (d=1200 mm) Mu = 1243 kN-m	$p_t = 50\frac{\left(1-\sqrt{1-\frac{4.6*M_u}{f_{ck}bd^2}}\right)}{\frac{fy}{fck}}=$ 0.35 (percentage of steel)	Provide 25 mm dia bars @ 122 mm spacing at bottom		IRC 78-2014 Section 708.11
10	Reinforcement in well cap for hogging (d=1250 mm) Mu = 1546 kN -m	$p_t = 50\frac{\left(1-\sqrt{1-\frac{4.6*M_u}{f_{ck}bd^2}}\right)}{\frac{fy}{fck}}=$ 0.44 (percentage of steel)	Provide 25 mm dia bars @ 100 mm spacing at bottom		

11	Well curb hoop tension (depth =2.75 m)	$T = 0.75N\left(\frac{\sin\theta - \mu\cos\theta}{\mu\sin\theta + \cos\theta}\right)$	2108	kN	
12	Well curb hoop reinforcement	Provide to resist hoop tension (Minimum 72 kg/m^3)	Provide 40 no bars of 25 mm dia along perimeter		IRC 78-2014 Section 708.7

Spread sheet:

For detail calculations for design of well foundation a spread sheet is developed and shown in Figure. The design of each component of the well foundation is done in spread sheet. If the stresses in well steining is above the permissible limits it give error message and asks to revise the diameter of the well steining. If the dispersion dimensions of pier doesn't fit into the provided diameter of well it gives message asking to revise the dimension of well.

Separate spread sheet is developed for the stability analysis and shown from Figure 4.3 If the required checks doesn't satisfies with the given conditions it gives error message showing that it is failed in particular check.

Sap modelling results of well steining are shown in Figure 4.9 & Figure 4.10.

INPUT				
PLAN DIMENSION OF ABUTMENT(M) :	7	X	5	
HEIGHT OF BEARING FROM MAX SCOUR LEVEL:	16	m		
HEIGHT OF ABUTMENT=	6	m		
SUBMERGED UNIT WEIGHT OF SOIL=	9.5	kN/m^3		
Total vertical load (includin dead weight and buyonce)	20000	kN		
Total lateral load at scour level=	400	kN		
GRADE 0F CONCRETE	30	N/mm^2		
GRIP LENGTH(m)=	3.33333	=	8	
DEPTH OF WELL (L)=	15.25	m		
DESIGN OF WELL COMPONENTS:				
1) DIMENSION OF WELL				
consider the DIAMETER of well with respective to abutment size				
INNER DIAMETER(di)	7.5	m		
2) WELL STEINING				
THICKNESS(h) = $Kd(L)^{1/2}$				
K VALUES	d is outer dia considering MIN 500mm thickness			
1)SANDY STRATA=0.03				
2)CLAYEY STRATA=0.033				
K =	0.03			
h=	0.99581	=	1	
outer diameter(d)=	9.5	m		
REINFORCEMENT:				
Grade of steel	=	500	N/mm^2	
VERTICAL:				
Provide greater than 0.12% of cross section	0.15	%		
Ast=	0.04004	m^2		
	40035	mm^2		
DIA of steelprovided=	20	mm		
NO OF BARS(for each face)=	63.75	=	64	
(provide 50 mm cover)spacing=	461.188	=	420	mm
i.e provide 20mm dia bars @420c/c at each face				
HOOP REINFORCEMENT:				
Provide not less than 0.04% volume/unit Length of steining			0.05	%
Volume of steel =	0.01335	m^3/m		
DIA of steelprovided=	12	mm		
Length of hoop=	29.516	m		
weight of steel per unit length=	104.758	kg		
weight of single hoop=	26.1914			
No of hoops=	3.99971	=	4	
i.e provide 4 hoops per 1 meter height at each face				
3) BOTTOM PLUG				
THICKNESS OF BOTTON PLUG(t)				

$$t^2 = 1.18r^2\frac{q}{fc}$$

where
q= bearing pressure at the base in KN/m^2
r= radius of the well in m
fc= fleural strength of concrete in KN/m^2

flexure strength of concrete M30	3834	kN/m^2		
THICKNESS(t)=	1.80089	=	1.9	m

Figure 4. 3 Spread sheet for design of well foundation

CHECK FOR STRESS IN STEINING						
Maximum stress occurs at x distane from scour level(Max Bending moment point)						
φ of soil	=	30	degrees			
δ	=	20	degrees			
factor of safety		1				
cosφ	=	0.86616				
cosδ	=	0.93975				
sin(φ+δ)	=	0.76576				
sin(φ)	=	0.49977				
Ka	=	0.29749				
Kp	=	6.09719				
x	=	1.23628				
MAX moment	=	494.514	kN m			
AREA OF WELL	=	26.69		M^2		
MOMENT OF INERTIA OF WELL	=	244.3803125		M^4		
MAX STRESS IN STEINING						
	758.956	kN/m^2				
	0.75896	<	3.8340579			
	SAFE					
4) DESIGN OF WELL CAP						
OVERALL DEPTH OF WELL CAP	=	1200	mm			
ASSUME DIA OF VERICAL REINFORCEMENT	=	28	mm			
EFFECTIVE DEPTH OF WELL CAP	=	1136	mm			
LOADING DETAILS@ BASE OF PIER						
VERTICAL LOAD	=	7325	kN			
MOMENT ABOUT TRANSVERSE AXIS	=	8768	kN m			
MOMENT ABOUT LONGITUDINAL AXIS	=	1309	kN m			
PIER DIMENSIONS:						
WIDTH	=	5000	mm			
LENGTH	=	7000	mm			
DISPERSION DIMENSIONS:;						
WIDTH	=	7272	mm			
LENGTH	=	9272	mm			
		ok				
DIA OF EQUIVALENT CIRCLE OF PIER	=	9267.85	mm			
support condition:						
freely supported on steining	=	1				
fully clamped on steining	=	2				
support condition:	2					
M_r =radial moment						
M_t = tangential moment		Mr	Mt			
Moment beneath loaded area	=	16.5932	16.5931688		698.5	698.457
Moment beneath unloaded area	=	-495.317	-74.297499		0.0	-495.721
Moment due to UDL@CENTRE	=	48.6504	48.6503906		133.3	133.26
Moment due to UDL@support	=	-84.6094	-12.691406		0.0	71.918
Max moment @centre from pier	=	791.533	(without sign)			
Max moment @edge from pier	=	1266.45	(without sign)			
centre due to patch load	=	357.525				
centre due to udl		90.9551				
centre from pier		791.533				
TOTAL SAGGING @CENTRE		1240.01	kN m	TOTAL HOGGIN		-791.533

Figure 4. 4 Spread sheet for design of well cap and check for stresses in steining

edge due to patch load	=	-247.658	
edge due to udl load		-42.3047	
edge from pier		-1266.45	
TOTAL HOGGING @ EDGE		-1556.42	kN m

$\frac{M_u}{bd^2}$	sagging @ centre	=	1.44132	
	hogging @centre	=	1.80909	

$$p_t = 50 \frac{\left(1-\sqrt{1-\frac{4.6 * M_u}{f_{ck} bd^2}}\right)}{\frac{fy}{fck}}$$

=	0.35218	%
=	0.44981	%

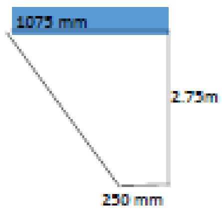
Ast@bottom	=	4000.71	mm^2
Ast@top	=	5109.87	mm^2
DIA of bars provided at bottom	=	25	mm
DIA of bars provided at top	=	25	mm
spacing bottom	=	123	mm
spacing top	=	97	mm

provide 25 dia bars @ 123 c/c at bottom

provide 25 dia bars @ 97 c/c at top

5] Design of Well curb

$$T = 0.75N\left(\frac{\sin\theta - \mu\cos\theta}{\mu\sin\theta + \cos\theta}\right)d$$



total running load of well steining on curb(N)	351.25		
mean diameter of well foundation(d)	8.5		
angle of beveled edge(degrees)	60	1.04667	
co eff of friction between soil and curb	0.3		
Hoop tension (T)	2107.95	kN	
DIA OF BARS(mm)	25		
depth of well curb=	2.75	m	
volume of curb=	57.8005	m^3	
no of bars required=	9		
Minimum reuirement (72 kg/m^3)=	40		

Provide 40 no's of 25 mm dia bars along perimeter

Figure 4. 5 Spread sheet for design of well curb

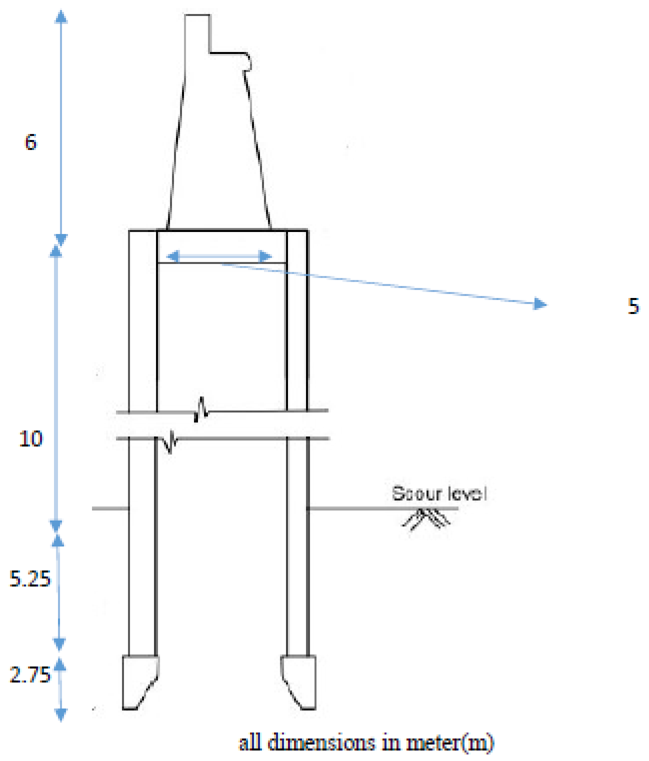


Figure 4. 6 Dimensions of well foundation

STABILITY ANALYSIS
ELASTIC THEORY

Loads under combination of load cases

W	=	10000	kN	VERTICAL LOAD@ BASE OF WELL
H	=	400	kN	HORIZONTAL LOAD @ SCOUR LEVEL
M	=	1000	kN-m	MOMENT@ BASE OF WELL
σ_S	=	400	kN/m^2	SAFE BEARING CAPACITY OF SOIL
L	=	8.55	m	WIDTH OF SOIL MASS PROVIDING RESISTANCE
D	=	8	m	DEPTH OF SCOUR LEVEL
σ_s	=			
I_B	=	244.3803	m^4	
I_V	=	364.8	m^4	
m	=	K_H/K	=	1
μ'	=	0.36377	μ=	0.576996
α	=	0.378185		
I	=	709.553		
r	=	7.780186		

$$\frac{M(1+\mu\mu')}{r}+\mu M = 732.506 \quad \text{OK}$$

$$\frac{M(1+\mu\mu\ )}{r}-\mu M = -421.4868 \quad \text{OK}$$

CHECK FOR ELASTIC STATE

$$\frac{mM}{I} \ngtr \gamma(K_p - K_A) \quad \text{OK}$$

$$\sigma_{1,}, \sigma_2 = \frac{W-\mu' P}{A} \pm \frac{MB}{2I}$$

σ1	=	378.9453	OK
σ2	=	366.8954	OK

Figure 4. 7 Spread sheet for stability checks of well foundation

ULTIMATE RESISTANCE

M	=	1000	kN m	APPLIED EXTERNAL MOMENT FOR A LOAD COMBINATION
σ_u	=	700	kN/m^2	
W	=	10000	kN	TOTAL VERTICAL LOAD CONSIDERING COMBINATIONS
$\frac{W}{A} > \frac{\sigma_u}{2}$	=	374.6722	NOT OK	

BASE RESISTING MOMENT(Mb)

M_b = $QWB \tan \phi$

= 32888.79

ULTIMATE MOMENT RESISTANCE OF WELL SIDE DUE TO PASIVE RESISTANCE(MS)

M_S = $0.10D^3(K_P - K_A)L$

= 2538.877

ULTIMATE MOMENT RESISTANCE OF WELL SIDE DUE TO FRICTION(MF)

M_f = $0.11\gamma(K_P - K_A)B^2D^2 \sin\delta$

= 11967.12

M_t $= 0.7(M_b + M_s + M_f)$

33176.35 OK

Figure 4. 8 Spread sheet for stability checks of well foundation

SAP MODELS:

Figure 4. 9 Stress Diagram (kN/m^2) for well steining

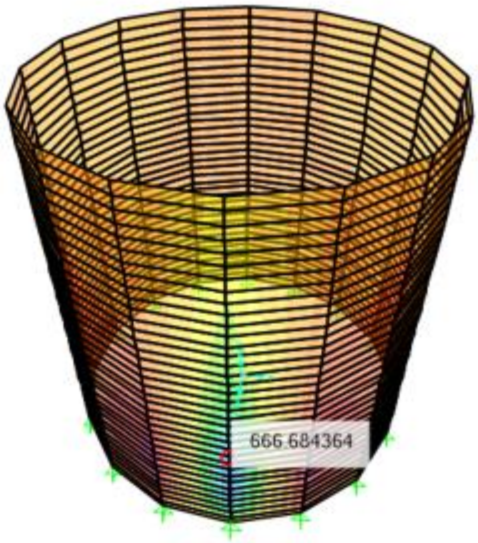


Figure 4. 10 Moment (kN-m) in the steining

Table 4. 2 Comparison of results

Sr.no	Description	Manual calculation	Sap results
1	Stress (kN/m^2) in Well Steining	760	713
2	Moment (kN-m) in the Steining	550	667

The variation of 10% -20% is seen for moment and stresses compared between the sap models and manual calculations.

CHAPTER 5

5 PILE FOUNDATION

In this chapter discussion on design of pile foundation is covered. The general introduction and the procedures for finding capacity of piles is covered. A spread sheet for the calculation of capacities and design of pile foundation is shown and discussed in this chapter.

5.1 General

Pile cap is a component of pile foundation which ties a group of piles together. The factors like spacing between piles, number of piles and type of arrangement of piles decides the plan dimension of pile cap. The arrangement of piles should be such that the centroid of group should coincide with the line of action of force. As per IS-2911-2010 clear overhang of 100 mm to 150 mm beyond the edge of the outermost pile is given to the pile cap. The thickness of pile cap depends on the factors like development length, shear developed and moment applied. Pile caps is designed using the truss analogy or bending theory. A clear cover of 50 mm is provided to the reinforcement in pile cap. Generally, the spacing between piles is not taken less than 3 times the diameter of pile. The load applied and the capacity of individual pile decides the number of piles should be considered in the group, minimum 3 piles should be taken in the pile group.

5.2 Capacity of piles

The following procedure is to find the capacity of the pile in different soil conditions using static analysis, the considered piles are bored piles and the following procedure is taken referring IS-2911-201.

5.2.1 Piles in cohesion-less soil

The pile capacity is the sum of the point bearing and skin friction resistance.

The ultimate capacity of pile is given in Eq. (46)

$$Q = Q_s + Q_p \quad (46)$$

$$Q = f_s A_s + q_p A_p \quad (47)$$

Where,

Q_s = Total skin friction resistance

Q_p = Total end bearing resistance

f_s = Unit skin friction resistance

q_p = Unit end bearing resistance

A_s =Surface area of pile

A_p = Area of pile at tip.

End bearing resistance:

$$f_p = \rho_v \left(N_q - 1\right) + \lambda_s \gamma B N_\gamma \quad (48)$$

Where,

ρ_v = effective overburden stress at the level of the pile tip

N_q = bearing capacity factors

λ_s = shape factor,

= 0.4 for square or rectangular piles

= 0.3 for circular piles

γ = density of the soil

B = diameter or width of pile

N_γ = bearing capacity factors

The bearing factors depends upon the internal friction of the soil it can by calculated using the value internal friction of soil from following Figure 5.1& Table 2 respectively which is reproduced from the IS 2911-2010

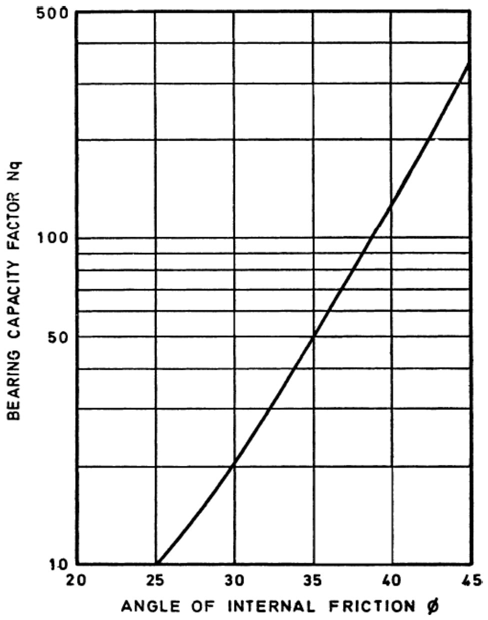


Figure 5. 1 Bearing factor N_q

Angle of internal friction of soil	N_γ
0	0.00
5	0.45
10	1.22
15	2.65
20	5.39
25	10.88
30	22.40
35	48.03
45	271.76

50	762.89

Table 5. 1 Bearing factor N_γ

Skin friction resistance:

$$f_s = K_s \rho_v \tan\delta \quad (49)$$

Where,

K_s = coefficient of horizontal stress

ρ_v = the effective overburden stress at the depth considered

δ = angle of wall friction of the material of the pile

= $\frac{2}{3}$ of the angle of friction of soil(ϕ)

$K_0 = 1 - \sin\phi$

Table 5. 2 Value of coefficient of horizontal stress (K_s)

Installation Method	K_s/K_o
Driven piles, large displacement	1 to 2
Driven piles, small displacement	0.75 to 1.25

Bored and cast-in-situ piles	0.70 to 1
Jetted piles	0.50 to 0.70

5.2.2 Piles cohesive soils:

End bearing resistance:

$$q_p = C_u N_c \quad (50)$$

Where,

C_u = Undrained cohesion at the pile tip

N_c = Bearing capacity factor, generally taken as 9

Skin friction resistance:

$$f_s = \alpha C_u \quad (51)$$

Where,

α = adhesive factor, depends on the C_u values

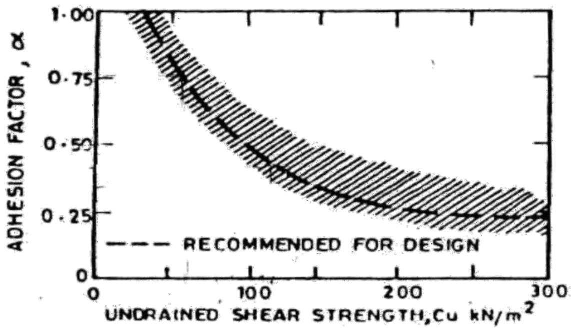


Figure 5. 2 Adhesive factors for cohesion soils

Safe bearing capacity can be obtained by dividing ultimate resistance with factor of safety 2.5.

The ultimate bearing capacity of pile group is calculated same as the individual pile the only difference is the area of pile replaced with the area of group and in case of skin friction adhesive factor is taken as 1 for all cases. For calculating the safe bearing, the ultimate bearing capacity is divided with the factor of safety of 3.

5.2.3 Lateral load capacity

A pile is classified as a long pile when its length is 10 times more than diameter and the response of the pile to lateral loads governed by flexural behaviour of pile.

There are 3 type of boundary conditions for piles

a) Free head pile
b) Fixed head pile
c) Partially fixed head

In a pile group if the number of piles less than 3 then free head is considered and greater than 3 it is considered as fixed head.

The following procedure is followed for calculation of lateral load capacity:

- The relative stiffness factor T or R

$$T = \sqrt[5]{\frac{EI}{\eta h}} \text{ For sand and normally loaded clay} \tag{52}$$

$$R = \sqrt[5]{\frac{EI}{K}} \text{ For pre-loaded clays} \tag{53}$$

Where,

E = Young's modulus of concrete (MPa)

I = second moment of inertia

$\eta h, K$ = constants

Table 5. 3 Values of constant ηh

SOIL TYPE	*Value of ηh, (kN/m^3)*	
	Dry	*Submerged*
Loose sand	2600	1460
Medium sand	7750	5260
Dense sand	20760	12450
Very loose sand under repeated loading	-	410
Very soft organic soil	-	110-270
For normally loaded clays		
Static loads	-	450
Repeated loads	-	270

Table 5. 4 Values of constant *K*

Unconfined Compression Strength, in kN/m^2	Range of values of *K*, in kN/m^2	Probable value of *K*, in kN/m^2
20 – 40	700 – 4200	775

100 - 200	3200 – 6500	4880
200 - 400	6500 – 13000	9770
> 400	-	19546

- The length of Fixity (L_f)

It depends on the stiffness factor values it can be calculated from following Figure 5.3.

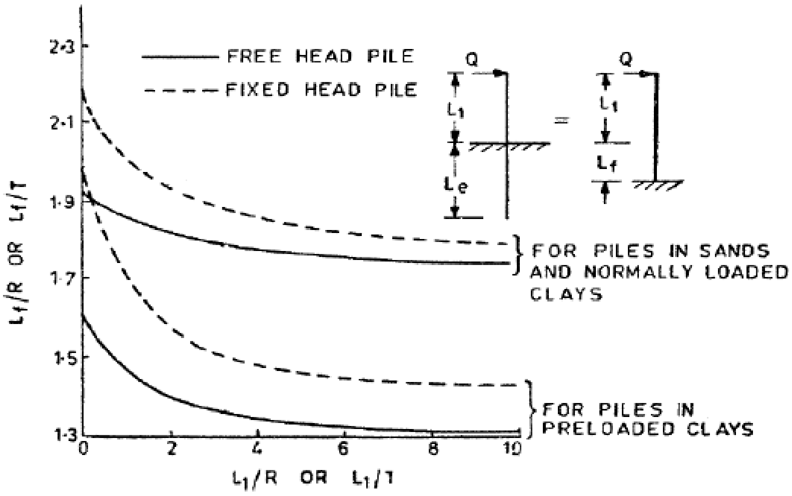


Figure 5. 3 Depth of fixity of piles

Where**,**

L_1 = unsupported length of pile.

Total length of pile = L_1+L_f (54)

- The capacity of piles is given as

$Q = \frac{3EIY}{(L1+LF)^3}$ For free head pile (55)

$Q = \frac{12EIY}{(L1+LF)^3}$ For fixed head pile (56)

Where,

Y = limited deflection (5 mm)

Q = lateral load capacity

5.3 Design of pile cap

5.3.1 Critical sections as per IS 456-2000

The pile cap is also checked for one-way shear & two-way shear. The critical Section for one-way shear in the pile cap is located at a distance equal to effective depth d away from the face of the pier. For two-way shear, the critical section is located at a distance of half of the effective depth d from the face of pier. The critical section for moment is at the face of the pier or pedestal.

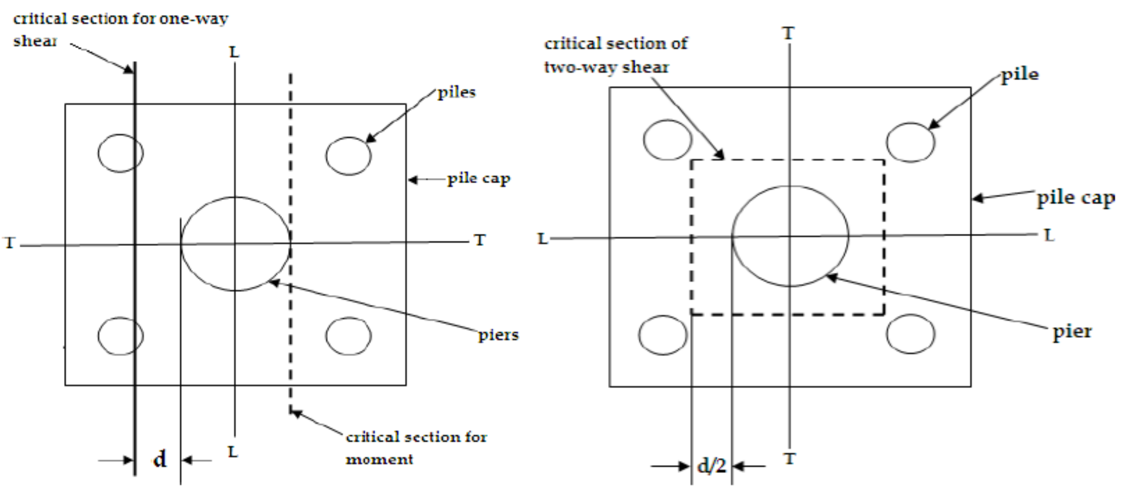


Figure 5. 4 Critical sections for moment, one-way and two-way shears (Varghese 2010)

5.3.2 Development length (L_d)

As per IS 456-2000 the bond stress (τ_{bd})for grade of concrete is shown in Figure 5.5

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

Figure 5. 5 Bond stress for plain bars in tension

- Bond stress is increased 60% for deformed bars.
- Bond stress for bars in compression increased by 25 % of the bond stress in tension.

Development length is given by

$$L_d = \frac{\phi\tau_{bd}}{4*0.87*f_y} \tag{57}$$

Where,

ϕ = diameter of bar

τ_{bd} = bond stress

f_y = yield stress of bars

As per IRC 112 the development is calculated by following procedure

$$L_d = \frac{\phi\tau_{bd}}{4*0.87*f_y} \tag{58}$$

$$L_d = K\phi \tag{59}$$

The value of k is taken by following Figure 5.6

Concrete Grade MPa ➡ Re-Bar Grade ⬇	M 20	M 25	M 30	M 35	M 40	M 45	M 50	M 55	M 60 AND HIGHER
Plain Bars (Fe 240)	52	47	43	40	37	36	35	33	31
HYSD Bars Fe 415 & Fe 415D	45	39	33	30	28	27	24	23	21
HYSD Bars Fe 500 & Fe 500D $\phi \leq 32$mm	54	47	40	36	34	32	29	27	25
HYSD Bars Fe 550 & Fe 550D	60	52	44	40	37	35	32	30	28
HYSD Bars (Fe 600)	65	57	48	43	41	38	35	33	30

Figure 5. 6 Value of k for different grades of concrete and steel

- For unfavourable bond conditions as per IRC 112-2011 the value of k should be multiplied with factor of 1.43
- For $\phi > 32mm$ the length should be increased by a factor of $\frac{100}{132-\phi}$

5.3.3 Depth of pile cap

The depth of pile cap depends upon the following criteria:

- The depth should be accommodating the reinforcement of the pier or column and individual pile for the development length which is calculated as section 5.3.2.

- The depth should be such that it should be safe in the one-way shear and two-way shear in the pile cap with respective to the pier and corner pile.
- The calculated depth should carry the moment which should be not less the design moment.
- It should be designed such that the load carried by the column should disperse at 45 degrees from top of cap to the mid depth of cap.

5.3.4 Distribution of load in vertical piles

By elastic theory the load on each individual pile is calculated and it is shown below

$$Q_i = \frac{Q}{n} \pm \frac{M_{xx} y_i}{\sum y^2} \pm \frac{M_{yy} x_i}{\sum x^2} \qquad (60)$$

Where,

Q = Vertical load on the pile cap from pier

Q_i = Load on the respective i^{th} pile

n= Number of piles

M_{xx} = Moment about x axis

M_{yy} = Moment about y axis

x_i = Distance of i^{th} pile from the centroid of the pile cap along x axis

y_i = Distance of i^{th} pile from the centroid of the pile cap along y axis

$\sum x^2$ = Sum of squares of all piles distance from the centroid of the pile cap along x axis

$\sum y^2$ = Sum of squares of all piles distance from the centroid of the pile cap along y axis.

If the load on any of the individual pile is more than the capacity, pile should be designed for it or the spacing's of piles can be altered.

5.4 Illustration

Design a pile cap for the pier of size 0.6 X 0.6 m using the piles of 0.67 diameter(D) and length 10 m having load capacity of 1000 KN, Grade of concrete = M30, Fy= 500 N/mm^2

Loads from pier to cap

Pz = 5000 KN

Mx =500 KN m

My =500 KN m

Table 5. 5 Calculations in design of pile foundation

Sr.No	Description	Formula	Value	Unit
1	Eccentricity (ey)	$\frac{M_X}{P_Z}$	500/5000 = 0.1	m
2	Eccentricity (ex)	$\frac{M_y}{P_Z}$	500/5000 = 0.1	m
3	Spacing of piles	Greater than 3*D	=3*0.67 =2.01	m
4	Edge distance	Greater than 0.15+0.5*D	=0.15+0.5*0.67 =0.485	m
5	Number of piles	$\frac{1.15 * P * (1+ex)(1+ey)}{load\ capacity}$	$\frac{1.15 * 5000 * (1+0.1)(1+0.1}{1000}$ =6.95 assumed 8 piles	
6	Force in i th pile	$\frac{P}{N} + \frac{Mx}{\Sigma y^\wedge 2} y + \frac{My}{\Sigma x^\wedge 2} x$	calculated in each pile with respective of its co-ordinates	kN
7	Check for capacity	Forces in each pile should be less than in capacities	-	-
8	Moment @ face of the pier	Pile forces outside the critical section * distance of pile	4000	kN m
9	IS-456 Area of steel (0.12% minimum)	$0.5 * \frac{Fck}{Fy} * b * d \left(1 - \sqrt{1 - \frac{4.6 * Mu}{Fck * b * d^\wedge 2}}\right)$	Provide 30 mm dia bars @ 170 c/c in both directions	
10	IRC 112 Area of steel (0.12% minimum)	$A_{st} = \frac{M}{0.87Fyz}$	Provide 30 mm dia bars @ 170 c/c in both directions	

Solution:

Arranging 3 piles @ first row and last row and @ one in middle row we get

Length = 2*2.01+2*0.485 = 5 m

Breadth = 2*2.01+2*0.485 = 5 m

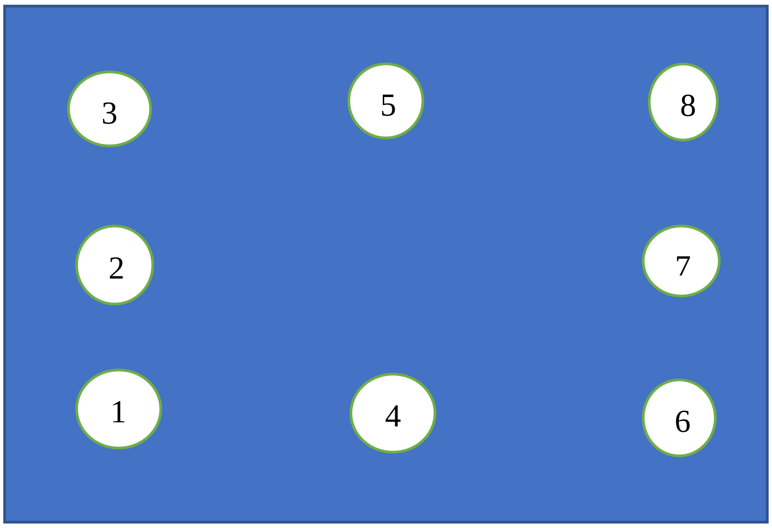


Figure 5. 7 Piles arrangement

Force on ith pile is = $\frac{P}{N} + \frac{Mx}{\Sigma y^{\wedge}2} y + \frac{My}{\Sigma x^{\wedge}2} x$

Pile 1 = $\frac{5000}{8} - \frac{500*2}{24} - \frac{500*2}{24}$ = 541.67 kN

Pile 2 =$\frac{5000}{8} - \frac{500*0}{24} - \frac{500*2}{24}$ =583.33 kN

Pile 3 =$\frac{5000}{8} + \frac{500*2}{24} - \frac{500*2}{24}$ = 625 kN

Pile 4 = $\frac{5000}{8} - \frac{500*2}{24} - \frac{500*0}{24}$ =583.33kN

Pile 5 =$\frac{5000}{8} + \frac{500*2}{24} - \frac{500*0}{24}$ =666.67 kN

Pile 6 =$\frac{5000}{8} - \frac{500*2}{24} + \frac{500*2}{24}$ =625 kN

Pile 7= $\frac{5000}{8} - \frac{500*0}{24} + \frac{500*2}{24}$ =666.67kN

Pile 8 =$\frac{5000}{8} + \frac{500*2}{24} + \frac{500*2}{24}$ =708.33 kN

All piles are safe in compression as forces are < 1000 kN

Assuming depth of cap (d)= 1550 mm

Moment:

Critical section for moment lies at the face of the centre

The pile 6,7,8 lies outside the critical section

The distance of each pile to the critical section is = 2.01

Therefore

Moment = Force of 6,7,8 * distance

$= (625 + 666.67 + 708.33) * (1.71)$

= 4000 KN M

Moment per unit width is = 4000/5 = 800 kN-m/m

Reinforcement as per is 456:

Minimum area of reinforcement = 0.12*b*d/100

= 0.12*5000*1550

= 9300 mm^2

$$A_{st}= 0.5 * \frac{Fck}{Fy} * b * d \left(1 - \sqrt{1 - \frac{4.6*Mu}{Fck*b*d^2}}\right) \quad (61)$$

$$= 0.5 * \frac{30}{500} * 5000 * 1550 * \left(1 - \sqrt{1 - \frac{4.6*3420*10^6}{20*5000*1550*1550}}\right)$$

= 6013mm^2<9300 mm^2

Provide 30 mm dia bars @ 170 c/c in both directions (minimum provided)

Reinforcement as per IRC 112:

$$z = d\left(0.5 * \sqrt{0.25 - k/1.134}\right) \quad (62)$$

Assuming k=0.13 <0.133 for single reinforcement

$$z = 1550\left(0.5 * \sqrt{0.25 - 0.13/1.134}\right)$$

z=1345 mm

$A_{st} = \frac{M}{0.87Fyz}$

=5468 mm^2<9300 mm^2

Check for one way

Critical section @ d from face i.e., 1550 mm from face

Case 1: If pile center is exact at critical section then no effect of pile force

Case 2: If pile center is D/2 away from critical section then full force of pile is included

Case 3: if pile center is in between force is interpolated

Critical section is @ 450 mm from the pile center of piles 6,7,8

Force is = 625+666.67+708.33

= 2000 kN

Stress = $\frac{2000}{5*1.550}$ = 258.54 KN / M^2 = 0.258N/mm^2 < 0.28(for M20 @ 0.121% steel)

Therefore SAFE

Check for two way

Critical section lies @ d/2 from the face i.e., 600 mm

Case 1: If pile center is exact at critical section then no effect of pile force

Case 2: If pile center is D/2 away from critical section then full force of pile is included

Case 3: if pile center is in between force is interpolated

Pile outside the critical sections is 1, 2,3,4,5,6,7,8

Force = 5000 KN

Stress = $\frac{5000}{4(0.6+1.550)1.550}$ = 375.09KN/M^2 = 0.37 N/mm^2 < 0.25*$\sqrt{20}$

Therefore SAFE

Corner pile shear check:

Critical section for corner pile will be at d distance inwards from the corner pile and perpendicular to the line joining centre of corner pile and corner of the pile.

As the pile cap is square the line will make 45 degrees to the horizontal

The distance pile and centre is = $\sqrt{2^2 + 2^2} = 2.828$

The distance of the point on the critical section will be = $2.828 - 1.550 - 0.5 * 0.67$

= 0.943

The x and y coordinate will be equal, x and y are taken from centroid

Therefore $\sqrt{x^2 + y^2}$ = 0.943

X = 0.66+2.5 = 3.16 from left bottom corner as origin

Y = 0.66+2.5 = 3.16

The line equation is $y = mx + c$

m=tan 135= -1

Substituting x , y and m we get c= 6.32

The line touches the edges at (1.32,5) and (5,1.32)

So distance will be equal to 5.2

As the width of critical section is more than the width of pile cap it will be safe for one way shear

Spread sheet:

For detail calculations, checks, design of pile foundation and capacity of piles spread sheet are developed it is provided from Figure 5.8 to Figure 5.20 .In this sheets yellow colour cells are inputs and blue colour cells gives output.

If the forces in piles greater than the capacities it displays the error message showing the pile is not ok. If the pile cap thickness not satisfies the moment and development length it displays the message to revise the assumed depth of cap. If the pile cap not satisfy the shear check after design it displays message to revise the depth of pile cap. To display the pile positions and critical sections in pile cap a graph is generated.

PILE CAPACITY

LENGTH OF PILE	=	4	m
DIAMETER OF PILE	=	0.2	m
NO OF SOIL LAYERS(n)	=	2	<10

TYPE OF SOIL

SANDY	=	1
CLAY	=	2
Ap	=	0.03141593 mm^2
As	=	0.62831853 mm

NOTE:
1. Take separate layer of soil from water table level
2. End the particular soil layer at depth equal to crtical depth

SOIL DETAILS

h	=	height of soil layer(m)
ϒ	=	unit weight of soil(KN/m^2)(effective)
ø	=	internal friction(radians)
δ	=	2/3*ø
Nq	=	bearing capcity factor
Nc	=	bearing capcity factor(9)
Cu	=	average undrained cohession at I th layer of pile
α	=	adhesion factor
P	=	overburden pressure at h
fs	=	skin friction
qp	=	end bearing
zc	=	3.2 m (critical depth 16*Dia of pile)

LAYER NUMBER(n)	SOIL TYPE	h	ϒ	Ø	δ	Nq	Nc	Cu	α	P	qp	fs	Qs
1	2	2	17	0.52	0.346667		9	120	0.415	34		49.8	62.58053
2	1	1.2	17	0.52	0.346667	60	9		0.9	54.4		12.28269	9.26093
3	1	0.8	17	0.52	0.346667	60	9		0.9	54.4	3209.6	12.28269	6.173953
4					0						0	0	0
5					0						0	0	0
6					0						0	0	0
7					0						0	0	0
8					0						0	0	0
9					0						0	0	0
10					0						0	0	0
											3209.6		78.01541

pile capacity	=	178.847967	kN
safe capacity	=	59.6159889	kN

Figure 5. 8 Spread sheet for the calculation of pile capacity

PILE GROUP CAPACITY

TOTAL NO OF PILES	=	25
No of piles in X direction	=	5
No of piles in Y direction	=	5
spacing provided	=	0.5
LENGTH OF GROUP	=	2.2
WIDTH OF GROUP	=	2.2
Ap	=	4.84 m^2
As	=	8.8 m

LAYER NUMBER(n)	SOIL TYPE	h	γ	ø	δ	Nq	Nc	Cu	α	P	qp	fs	Qs
1	2	2	17	0.52	0.346667		9	120	1	34		120	2112
2	1	2	17	0.52	0.346667	60	9		1	68	4012	24.33388	428.2762
3					0						0	0	0
4					0						0	0	0
5					0						0	0	0
6					0						0	0	0
7					0						0	0	0
8					0						0	0	0
9					0						0	0	0
10					0						0	0	0
											4012		2540.276

Pile group capacity	=	21958.3562 kN
Safe group capacity	=	7319.45208 kN
Safe capacity(by individual piles)		1490.39972 kN

Figure 5. 9 Spread sheet for the calculation of pile group capacity

LATERAL LOAD CAPACITY OF PILE

PILE PROPERTIES

Diameter of pile	=	0.9	m
grade of concrete	=	25	Mpa
E	=	25000	N/mm^2
I	=	0.032206	m^4
noof piles	=	3	
consider pile head as		free-head piles	
unsupported length(L1)=		0	m

Relative stiffness:

for sand and normal loaded clays

$$T = \sqrt[5]{\frac{EI}{\eta h}}$$

for pre loaded clays

$$R = \sqrt[5]{\frac{EI}{K}}$$

ηh	=	450
k	=	
T or R	=	4.472326
L1/T or L1/R=		0

SOIL TYPE	Value of η_h, (kN/m³)	
	Dry	Submerged
Loose sand	2600	1460
Medium sand	7750	5260
Dense sand	20760	12450
Very loose sand under repeated loading	-	410
Very soft organic soil	-	110-270
For normally loaded clays • Static loads • Repeated loads	- -	450 270

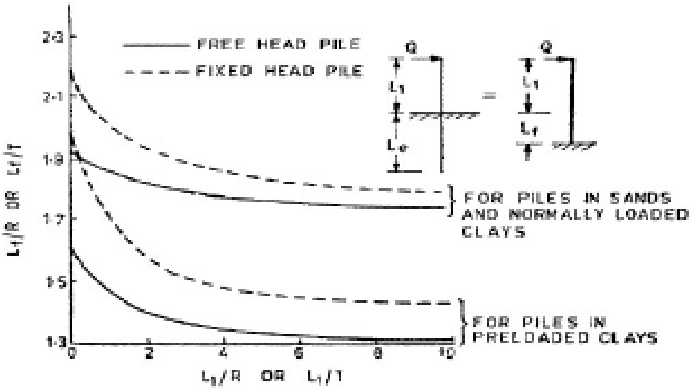
Values of ηh

Unconfined Compression Strength, in kN/m²	Range of values of K, in kN/m²	Probable value of K, in kN/m²
20 – 40	700 – 4200	775
100 - 200	3200 – 6500	4880
200 - 400	6500 – 13000	9770
> 400	-	19546

values of k

Figure 5. 10 Spread sheet for the calculation of pile lateral load capacity

CALCULATION OF Lf



Lf/T or Lf/R=	2.2
Lf=	9.83912

TOTAL LENGTH(L1+Lf)= 9.83912

LATERAL LOAD CAPACITY(Q)

$$Q = \frac{3EIY}{(L1 + LF)^3}$$ FREE HEAD

$$Q = \frac{12EIY}{(L1 + LF)^3}$$ FIXED HEAD

Y = limited lateral deflection(5 mm)

Q = 12.6795 kN

Fixed end moment

$$Mf = Q(L1 + Lf)$$ FREE HEAD

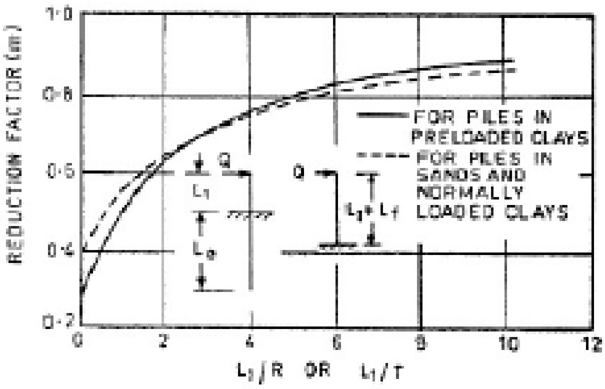
$$Mf = 0.5Q(L1 + Lf)$$ FIXED HEAD

M_f = 124.755 kN m

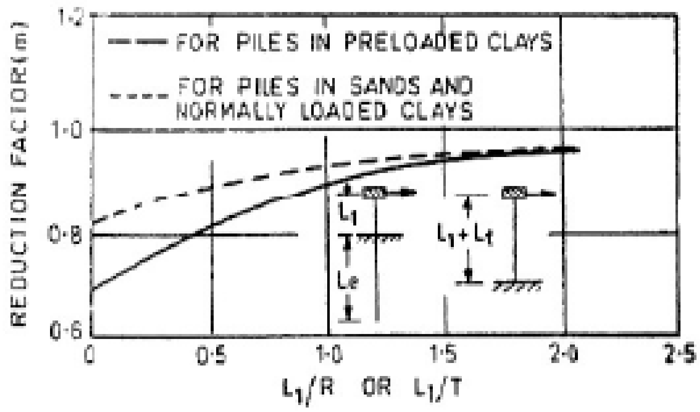
Figure 5. 11 Spread sheet for the calculation of pile lateral load capacity

DESIGN MOMENT (M)

$$M = mM_f$$



Reduction factor(m) for free head piles



Reduction factor(m) for fied head piles

m	=	0.3	
M	=	37.42658	kN-m

Figure 5. 12 Spread sheet for the calculation of moment in pile

INPUT

1)COLUMN DATA:

B	=	0.5	m
D	=	0.5	m
DIA OF BARS	=	20	mm

2) MATERIAL PROPERTIES

REBAR	fy	=	500	N/mm2
CONCRETE	f_{ck}	=	30	N/mm2

E_S	=	200000	N/mm2
E_C	=	27386.13	N/mm2

3) loading data

P_X	=	0	kN				
P_Y	=	0	kN				
P_Z	=	5000	kN				
M_X	=	500	kN-m	e_x	=	0.1	m
M_Y	=	500	kN-m	e_y	=	0.1	m

4)PILE DATA

PILE DIMENSIONS:

D(DIA)	=	0.67	m
L(length)	=	10	m

CAPACITIES:

1) PILE LOAD CAPACITY	=	1000	kN
2) UPLIFT CAPACITY	=	200	kN
3) LATERAL LOAD CAPACITY	=	160	kN
NO OF PILES REQUIRED	=	6.9575	
ENTER NO OF PILES	=	8	
Asume depth of pile cap	=	1550	mm

OVER HANG= 150 mm

SPACING= 2.01 mm(3*d)

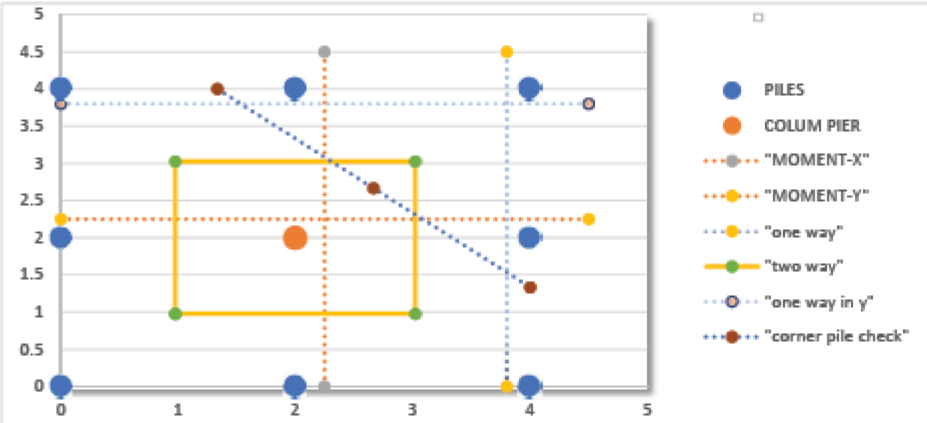
Figure 5. 13 Spread sheet for input for design of pile cap

Arrangement of pile group(ENTER CORDINATES PILE REACTIONS

TAKE 1 ST PILE AS ORIGIN

PILE NUMB	X(M)	Y(M)	VERTICAL(kN)		HORIZONTAL(kN)	
1	0	0	541.6667	OK	0	OK
2	0	2	583.3333	OK	0	OK
3	0	4	625	OK	0	OK
4	2	0	583.3333	OK	0	OK
5	2	4	666.6667	OK	0	OK
6	4	0	625	OK	0	OK
7	4	2	666.6667	OK	0	OK
8	4	4	708.3333	OK	0	OK

Figure 5. 14 Spread sheet for pile reactions



MOMENT about y

P	FORCE(KN)	DIST	MOMENT
6	625	2	1250
7	666.666667	2	1333.3333
8	708.333333	2	1416.6667
		TOTAL	4000

MOMENT about x

P	FORCE(KN)	DIST	MOMENT
3	625	2	1250
5	666.6666667	2	1333.333
8	708.3333333	2	1416.667
		TOTAL	4000

MAX MOMENT= 4000 kN m

Figure 5. 15 Spread sheet showing pile arrangement and critical sections

Forces for one way(X)

P	FORCE(KN)
6	625
7	666.6667
8	708.3333
TOTAL	2000 kN

Forces for one way(Y)

P	FORCE(kN)
3	625
5	666.6667
8	708.3333
TOTAL	2000 kN

Forces for two way

P	FORCE(KN)
1	541.6666667
2	583.3333333
3	625
4	583.3333333
5	666.6666667
6	625
7	666.6666667
8	708.3333333
	5000 kN

FOR CORNER PILE CHECK

Forces for one way(X)

Consider piles out side critical section

P	FORCE(KN)
5	666.666667
7	666.666667
8	708.333333
TOTAL	2041.66667 kN

Figure 5. 16 Spread sheet showing pile reactions considered in different checks

IS 456-design

DIMENSIONS OF PILE CAP		
LENGTH(m)	=	4.97
BREDTH(m)	=	4.97
REVISE		
LENGTH(m)	=	5
BREDTH(m)	=	5
DEPTH(mm)	=	1700
d CALCULATION		
d from moment	=	439.5869823
d from devlopement length	=	725
	Assumed depth is ok	
DIA OF BARS IN PILE CAP(mm)		20

Steel provided

long direction

Steel required (mm^2)	Area of one bar (n	(Number of bars)$_{req}$	Spacing (mm)
6013.24542	314.1592654	30	170

Steel provided (mm^2)	MIN steel	
9424.777961	9300	MINIMUM STEEL is provided

short direction

Steel required (mm^2)	Area of one bar (n	(Number of bars)$_{req}$	Spacing (mm)
6013.24542	314.1592654	30	170

Steel provided (mm^2)	MIN steel	
9424.777961	9300	MINIMUM STEEL is provided

One way shear

In X direction

Steel percent (pt%)	0.121610038	
τ_c (N/mm^2)	0.28	
Nominal shear stress τ_V (N/mm	0.258064516	OK

Figure 5. 17 Spread sheet for design of pile cap (IS 456-2000)

In Y direction

Steel percent (pt%)	0.121610038	
τ_c (N/mm^2)	0.28	
Nominal shear stress τ_V (N/mm	0.258064516	OK

Corner pile

Steel percent (pt%)	0.12	
τc (N/mm2)	0.28	
B(WIDTH FOR CRITICAL SE	5.15571309	
Nominal shear stress τV (N/mm	0.255484407	OK

Two way shear

COLUMN:

Perimeter of critical section (mı	8200	
Area of critical section (mm^2)	12710000	
k_s	1	
τ_c	1.369306394	
Permissible shear (N)	17403884.26	
Shear (N)	5000000	OK

PILE:

Perimeter of critical section (mı	4.5373	
Area of critical section (mm^2)	7032815	
k_s	1	
τ_c	1.369306394	
Permissible shear (N)	9630078.546	
Shear (N)	708333.3333	OK

Figure 5. 18 Spread sheet for design of pile cap (IS 456-2000)

IRC 112-Design

DIMENSIONS OF PILE CAP				
LENGTH(L)	=	4.97		
BREDTH(B)	=	4.97		
REVISE				
LENGTH(L)	=	5		
BREDTH(B)	=	5		
DEPTH CALCULATION				
z =	1345.27			
d from moment	=	452.9108137		
d from devlopement length	=	806		
Assumed depth is ok				
DIA OF BARS IN PILE CAP(mm)		20		

Steel provided

long direction

Steel required (mm^2)		Area of one bar (	(Number of bars)$_{req}$	Spacing (mm)
	6835.36595	314.1592654	38	130
Steel provided (mm^2)		MIN steel		
	11938.05208	11625	MINIMUM STEEL is provided	

short direction

Steel required (mm^2)		Area of one bar (	(Number of bars)$_{req}$	Spacing (mm)
	6835.36595	314.1592654	38	130
Steel provided (mm^2)		MIN steel		
	11938.05208	11625	MINIMUM STEEL is provided	

One way shear

In X direction

k	1.359210604	
Vrd,c	0.271708858	
Steel percent (pt%)	0.154039382	
Nominal shear stress τ_V (N/mm^2)	0.258064516	OK

Figure 5. 19 Spread sheet for design of pile cap (IRC 112-2011)

In Y direction		
k	1.359210604	
Vrd,c	0.271708858	
Steel percent (pt%)	0.154039382	
Nominal shear stress τ_V (N/mm^2)	0.258064516	OK
Two way shear		
Coloumn:		
Perimeter of critical section (mm)	2000	
Area of critical section (mm^2)	3100000	
Vrd max	13451520	N
Shear (N)	5000000	OK
Pile:		
Perimeter of critical section (mm)	4.5373	
Area of critical section (mm^2)	7032815	
Vrd max	30516790.85	N
Shear (N)	708333.3333	OK

Figure 5. 20 Spread sheet for design of pile cap (IRC 112-2011)

Design moment with sap model:

The pile cap is modelled in the SAP2000 with the dimensions and loading as per the example taken

The mesh size = 0.5*0.5 m

The mesh size is selected nearer to the column size

In the location of pile, a spring with vertical stiffness of $\frac{AE}{L}$ value is provided

The load is applied with eccentricities obtained in respective directions with applied moments.

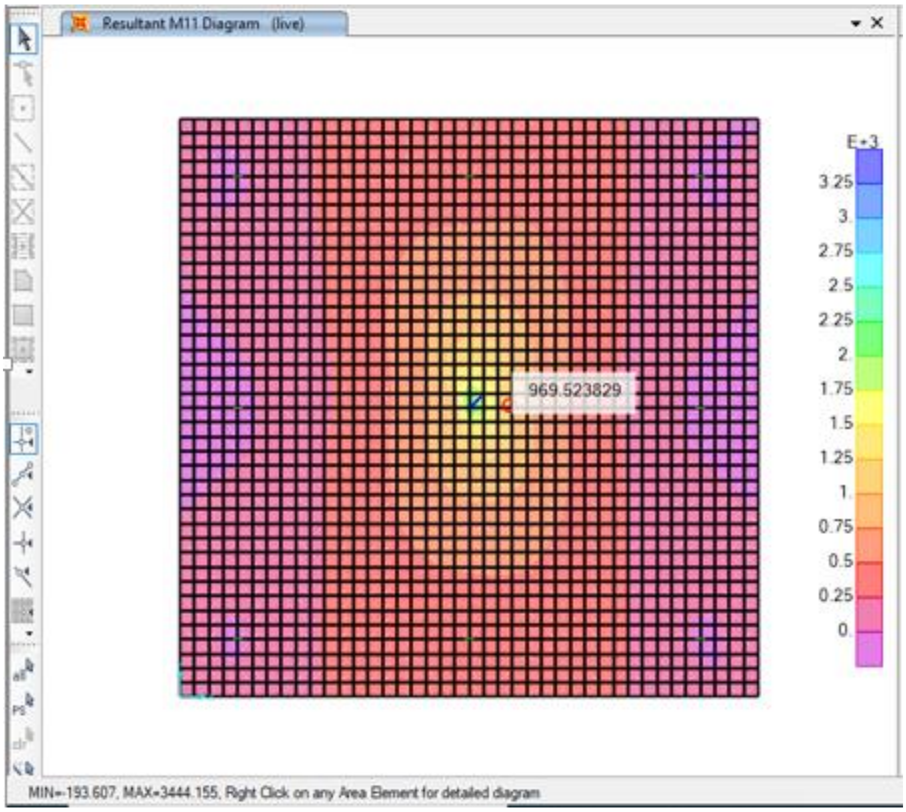


Figure 5. 21 Moment about Y axis from SAP2000

Table 5. 6 Comparison of results

Sr .no	**Description**	**Manual calculation value**	**Sap result**	**unit**
1	Moment per unit width about y @ face of column	4000/5 = 800	970	kN m

It can be seen from Table 5.6 the variation of 20% of moment between the manual calculation result and SAP2000 result.

CHAPTER 6

6 Spread foundation or open foundation

In this chapter the discussion on the design of spread foundations is covered. In case of uniaxial and biaxial bending foundations procedure for finding maximum and minimum pressures is covered. Spread developed is shown for the design of foundations with no eccentricity, uniaxial and biaxial bending is shown and discussed in this chapter.

6.1 General

Spread foundation or an open foundation is shallow type of foundation which is used for buildings and small bridges under the piers. It is called open because the earth is excavated till the bottom of the foundation and then construction will be started, and the early stage of work can be seen with eye. There are different type of open foundations like square, rectangle, trapezoidal etc. It can be individual, strip or raft. In this study the individual isolated footing is consider.

Usually, the foundation is designed such that the pressure under the footing will be not greater than the safe bearing capacity of the soil which is considered and we assume the distribution of pressure is linear under the footing. When the pressure under the footing is in tension then it implies that there is no contact of the soil with foundation in such cases care must be taken in while design of footing by considering the area of the footing which doesn't includes the lift off area due to tension.

6.2 Design of foundation

The basic steps in design of footings are shown in following steps:

- Determine design loads for the size of foundation and limit state design of foundation.
- Determine plan area using allowable bearing capacity of soil and service loads from column including 10% of self-weight.

- Determine the ultimate soil reaction using factored dead load and live load.
- Determine depth from bending considerations.
- Calculate the area of steel should be provided in both directions.
- Check the depth for one way and two-way shear
- Check the depth for development length of reinforcement in columns
- Check the length and breadths of foundation for the development length of reinforcement in the foundation
- Provide the necessary cover to reinforcement and find the total depth of footing required.

The loadings considerations make foundation to divide in to 3 categories they are:

1. No eccentricity
2. Uniaxial
3. Biaxial

The pressure distribution in the no eccentricity case will be constant throughout the foundation, in uniaxial case the pressure linearly varies either in length or breadth directions, in bi axial case the pressure at four corners will be different. In Last two cases the pressure may be even negative that is tension (lift off).

The contact pressure at any point (x, y) in rectangular footings is as Eq. (63)

$$q = \frac{P}{A} \pm \frac{M_X}{I_X} y \pm \frac{M_y}{I_Y} x \tag{63}$$

The contact pressures at all 4 corners in rectangular footings can be simply written as Eq. (64).

$$q = \frac{P}{A}\left(1 \pm \frac{6e_l}{L} \pm \frac{6e_b}{B}\right) \tag{64}$$

Where,

q= contact pressure $\left(\frac{KN}{m^2}\right)$

P= vertical load (KN),

x & y= coordinates of points,

M_x & M_y = moments in x and y direction (KN-M)

e_l & e_b = eccentricities in x and y direction, L & B = length and breadth of foundation.

The above equations are only when used when there is no lift off area that is footing is contact with soil.

The footing will have lift area only when the eccentricity is outside the kern

When the eccentricity is outside the kern area in uniaxial case the pressure as per Eq. (65):

$e_x or e_b > \frac{L}{6}\ or\ \frac{B}{6}$

$$q_{max} = \frac{P}{A}\left(\frac{4B}{3B-6e}\right), q_{min} = 0 \qquad (65)$$

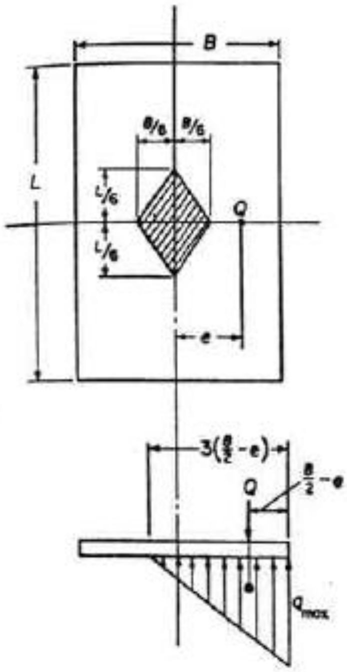


Figure 6. 1 Stress distribution when load is on one of the central lines with eccentricity >$\frac{L}{6}$ or $\frac{B}{6}$(Wayne 1992)

When eccentricity is outside the kern area in biaxial case:

Case 1: $e_l < \frac{L}{6}\ \ and\ \ e_b < \frac{B}{6}$ (66)

Case 2: $\frac{L}{4} > e_l > \frac{L}{6}\ \ and\ \frac{B}{4} > \ e_b > \frac{B}{6}$ (67)

Case 3: $\frac{L}{4} > e_l > \frac{L}{6}\ \ and\ \frac{B}{4} < \ e_b$ (68)

Case 4: $e_l > \frac{L}{4}\ \ and\ \ e_b > \frac{B}{4}$ (69)

For the all cases to find the maximum pressures with consider the lift off can be calculated using the following graph shown in Figure 6.2.

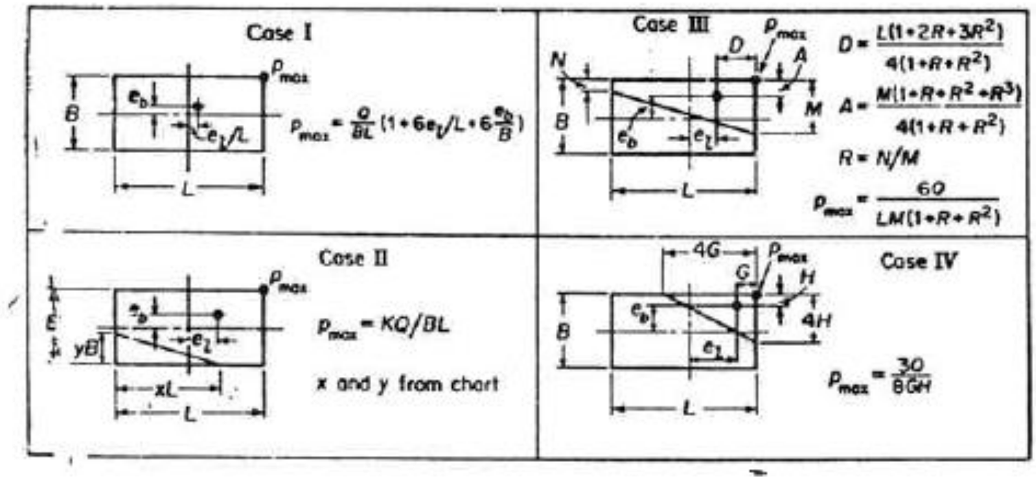


Figure 6. 2 Maximum pressure for different cases in biaxial loading (Wayne 1992)

For the particular case 2 to find the maximum pressure there is a need of other constant this can be calculated from the Figure 6.3.

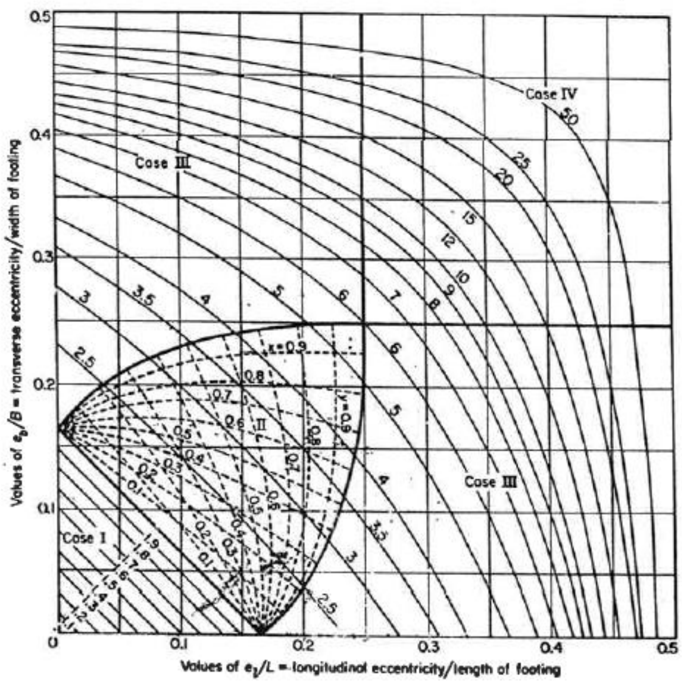


Figure 6. 3 Graph showing the different cases and values of coeffiecnt k(Wayne 1992)

Spread sheet:

For all the cases No eccentricity, uniaxial and Biaxial loading the excel spread sheets developed for the finding maximum pressures and design with lift off for respective case. In this sheets yellow colour cells are input and blue cells are outputs. Excel sheets displays the number of bars to be provided in each direction with spacing for all cases with respective to given input. If the maximum pressure exceed the bearing capacity of soil it displays the message showing to revise the dimensions of footing. If the footing doesn't satisfies the check for shear and development length it displays the message to revise the depth provided for footing.

No eccentricity spread foundation

$(Pz)_{working}$ (kN)	900
SBC (kN/m^2)	120
Size of column	
x direction (m)	0.35
y direction (m)	0.35
Diameter of column main reinforcement φ_1 (mm)	20
Grade of concrete (fck) (N/mm^2)	25
Grade of steel (fy) (N/mm^2)	415
Clear cover for footing (mm)	50
Load factor	1.5
Dimension of footing	
(Area of foorting)$_{required}$ (m^2)	8.25
Length of footing (L) (m)	3
Projection from face of column (L') (m)	1.325
Width of footing (B) (m)	3
Upward pressure (q_u) (kN/m^2)	100
If q_u>SBC	No
Dimensions are ok	
Depth of footing	
Moment about x axis (kNm)	263.34375
Moment about y axis (kNm)	395.015625
Mx per unit length(kN-m/m)	87.78125
My per unit length(kN-m/m)	131.671875
Moment used for design	My
Effective depth (d)$_{required}$ (mm)from moment	195.3605919
Development length (L_d)	645
Diameter of footing reinforcement φ_2 (mm)	16
Total depth (D)$_{provided}$ (mm)	800
Effective depth (d)$_{obtained}$	742, OK

Figure 6. 4 Spread sheet for design of spread footing- no eccentricity

Steel provided

Short direction

Steel required (mm^2)	Area of one bar (mm^2)	$(\text{Number of bars})_{req}$	Spacing (mm)
2880	201.0619298	15	210

Steel provided (mm^2)	Minimum Steel required (mm2)
3015.928947	2880

long direction

Steel required (mm^2)	Area of one bar (mm^2)	$(\text{Number of bars})_{req}$	Spacing (mm)
2880	201.0619298	15	210

Steel provided (mm^2)	Minimum Steel required (mm2)
3015.928947	2880

One way shear

In X direction

Steel percent (pt%)	0.135486476
τ_c (N/mm^2)	0.29
Nominal shear stress τ_V (N/mm^2)	0.0785714285714286, OK

In Y direction

Steel percent (pt%)	0.135486476
τ_c (N/mm^2)	0.29
Nominal shear stress τ_V (N/mm^2)	0.0785714285714286, OK

Two way shear

Perimeter of critical section (mm)	4368
Area of critical section (mm^2)	3241056
k_s	1
τ_c	1.25
Permissible shear (N)	4051320
Shear (kN)	463.2, OK

Figure 6.4a spread sheet for design of spread footing- no eccentricity

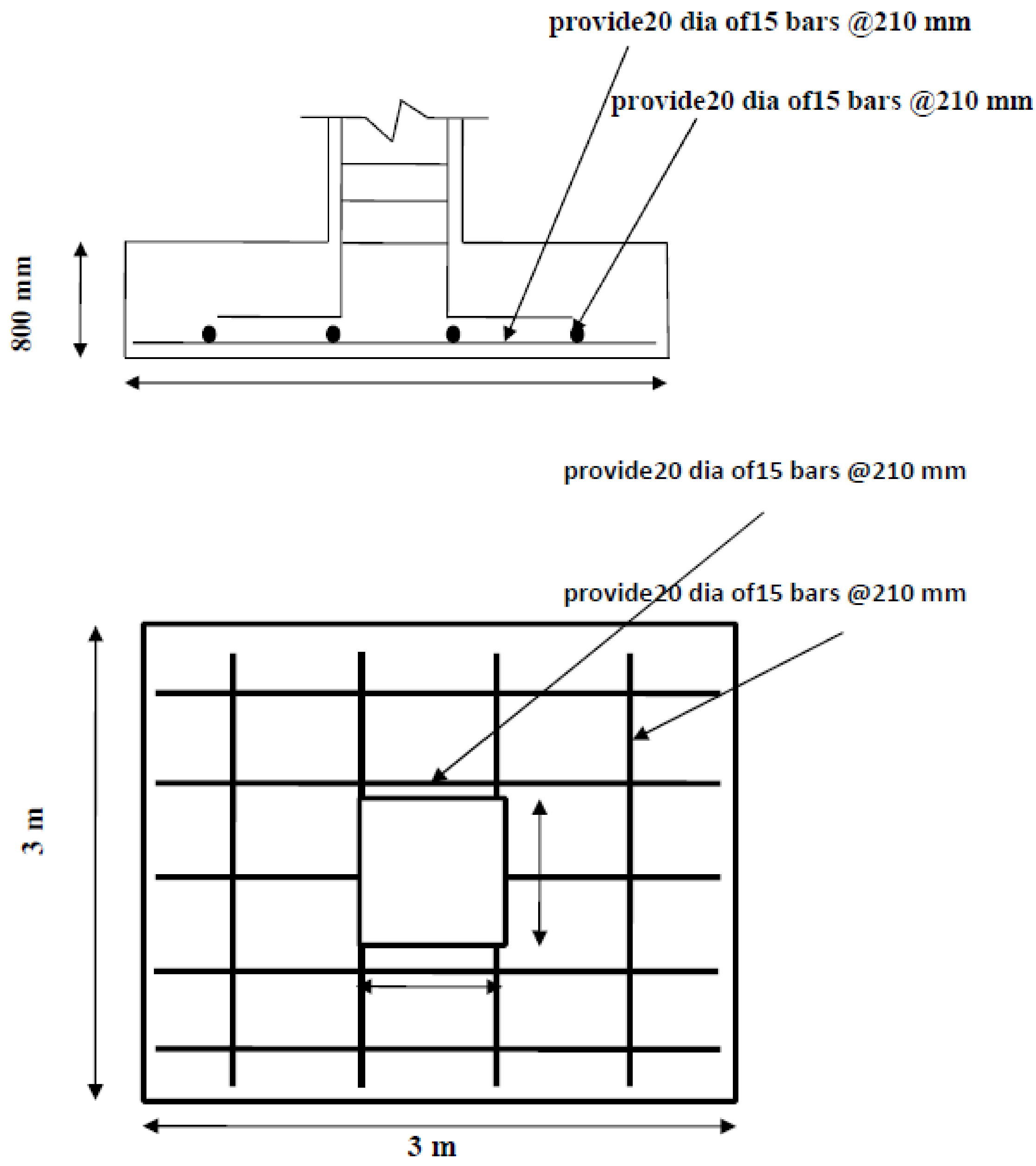


Figure 6. 5 Spread sheet showing dimensions of spread footing- no eccentricity

UNIAXIAL SPREAD FOUNDATION

$(Pz)_{working}$ (kN)	3000
Moment about X axis (Mx') (kNm)	0
Moment about Y axis (My') (kNm)	1500
Eccentricity in X direction e_L (m)	0.5
Eccentricity in Y direction e_B (m)	0
SBC (kN/m^2)	180
Size of column	
X direction (m)	0.35
Y direction (m)	0.35
Diameter of column main reinforcement φ_1 (mm)	20
Grade of concrete (fck) (N/mm^2)	30
Grade of steel (fy) (N/mm^2)	415
Clear cover for footing (mm)	50
Load factor	1.5
Area of footing required (mm^2)	25

Figure 6. 6 Spread sheet for input for design of uniaxial spread footing

Dimension of footing	
Value of projection (L') (m)	2.325
Value of length (L) (m)	5
width for mx(m)	5
width for my(m)	5
Value of projection (B') (m)	2.325
Value of width (B) (m)	5
Bending type	UNIAXIAL
Provided area(m2)	25
lift off area(m^2)	0
maximum pressure(kN/m^2)	192
revision of dimensions required	
Depth of footing	
Moment about x axis (Mx) (kNm)	2432.53125
Moment about y axis (My) (kNm)	3213.373781
Moment used for design	My
Moment in Mx per unit length (kNm)	486.50625
Moment in My per unit length(kNm)	642.6747563
Effective depth $(d)_{required}$ (mm)	393.9993
Development length (L_d)	602
Diameter of footing reinforcement φ_2 (mm)	25
Total depth $(D)_{provided}$ (mm)	700
Effective depth $(d)_{obtained}$	637.5, OK

Figure 6. 7 Spread sheet for design of uniaxial spread footing

Steel provided

longitudinal

Steel required (mm^2)		Area of one bar (mm^2)	Number of bars	Spacing (mm)
	11109.33983	490.8738521	23	210
Steel provided (mm^2)		Mnimum area of steel		
	11290.0986	4200		

Transverse

Steel required (mm^2)		Area of one bar (mm^2)	Number of bars	Spacing (mm)
	14936.05242	490.8738521	31	160
Steel provided (mm^2)		Mnimum area of steel		
	15217.08942	4200		

One way shear

In X direction

Steel percent (pt%)	0.477398884
τ_c (N/mm^2)	0.488247419
Nominal shear stress τ_V (N/mm^2)	0.476470588235294, OK

In Y direction

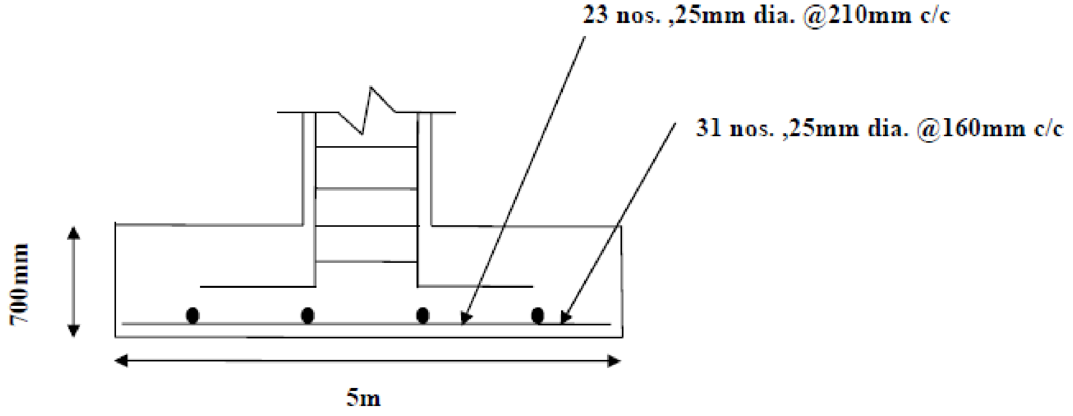
Steel percent (pt%)	0.354199172
τ_c (N/mm^2)	0.424183569
Nominal shear stress τ_V (N/mm^2)	0.569382352941176, REVISION REQUIRED

Two way shear

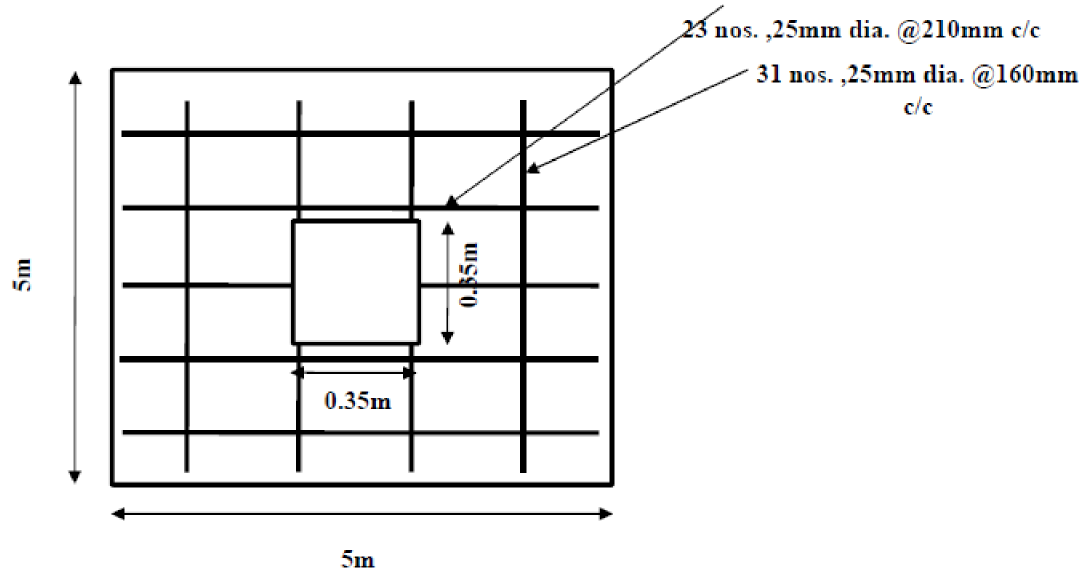
Perimeter of critical section (mm)	3950
Area of critical section (mm^2)	2518125
k_s	1
τ_c	1.369306394
Permissible shear (N)	3448084.663
Shear (KN)	4324.471875, REVISION REQUIRED

Figure 6.7b Spread sheet for design of spread footing uniaxial

Reinforcement Detailing



SECTION ELEVATION



PLAN

Figure 6. 8 Spread sheet sowing dimensions of spread footing

Biaxial Spread foundations	
$(Pz)_{working}$ (KN)	1500
Moment about X axis (Mx') (KNm)	1000
Moment about Y axis (My') (KNm)	1000
Eccentricity in X direction e_L (m)	0.666666667
Eccentricity in Y direction e_B (m)	0.666666667
Eccentricity type	Case 1
SBC (KN/m^2)	150
Size of column	
X direction (m)	0.45
Y direction (m)	0.45
Diameter of column main reinforcement φ_1 (mm)	16
Grade of concrete (fck) (N/mm^2)	30
Grade of steel (fy) (N/mm^2)	415
Clear cover for footing (mm)	50
Load factor	1.5
Area of footing required(m^2)	23.33333333
Dimension of footing	
Value of projection (L') (m)	2.775
Value of length (L) (m)	6
Value of projection (B') (m)	2.775
Value of width (B) (m)	6
Max. Pressure at the base (kN/m^2)	97.33333333
Lift off area (m^2)	not calculated
Lift off (%)	not calculated
Design Provided	Yes

Depth of footing	
Moment about x axis (Mx) (kNm)	1962
Moment about y axis (My) (kNm)	2017
Moment used for design	My
Moment about x axis(kNm)	327
Moment about y axis (kNm)	336.1666667
Effective depth $(d)_{required}$ (mm)	284.9555719
Development length (L_d)	481
Diameter of footing reinforcement φ_2 (mm)	28
Total depth $(D)_{provided}$ (mm)	600
Effective depth $(d)_{obtained}$	536, OK

Figure 6. 9 Spread sheet for design of spread footing biaxial

Steel provided

Longitudinal

Steel required (mm^2)		Area of one bar (mm^2)	Number of bars	Spacing (mm)
	10942.83041	615.7521601	18	350
Steel provided (mm^2)		Minimum area of steel		
	11083.53888	3859.2		

Transverse

Steel required (mm^2)		Area of one bar (mm^2)	Number of bars	Spacing (mm)
	10629.4006	615.7521601	18	350
Steel provided (mm^2)		Minimum area of steel		
	11083.53888	3859.2		

One way shear

In X direction

Steel percent (pt%)	0.344637403
τ_c (N/mm^2)	0.41921145
Nominal shear stress τ_{VX} (N/mm^2)	0.3053, OK

In Y direction

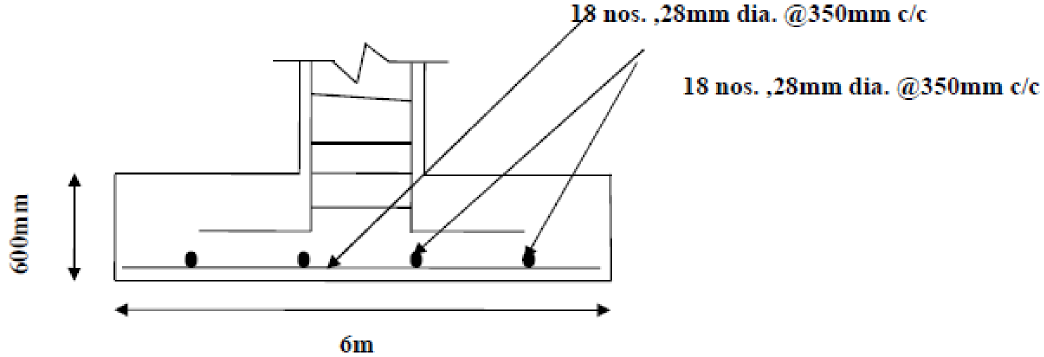
Steel percent (pt%)	1.033912209
τ_c (N/mm^2)	0.777634349
Nominal shear stress τ_{VY} (N/mm^2)	0.3053, OK

Two way shear

Perimeter of critical section (mm)	3944
Area of critical section (mm^2)	2113984
k_s	1
τ_c	1.369306394
Permissible shear (N)	2894691.808
Shear (kN)	2189.2378, OK

Figure 6.9b Spread sheet for design of spread footing Biaxial

Reinforcement Detailing



SECTION ELEVATION

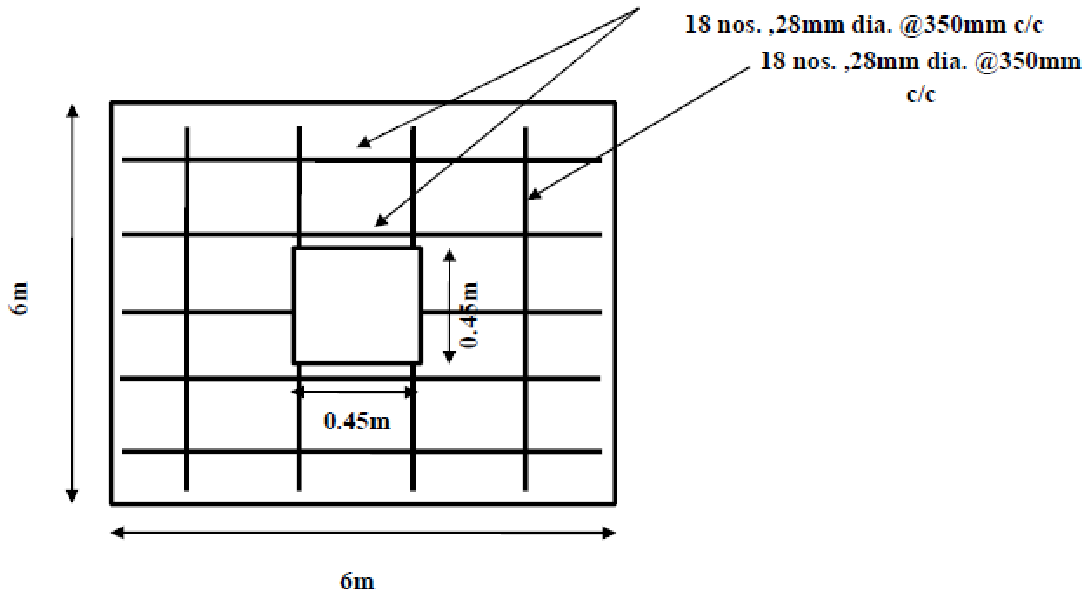


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Figure 6. 10 Spread sheet showing dimensions of spread footing Biaxial

6.3 Study of effect of mesh size

An example in each case provided in the spread sheets were taken and modelled in the SAP2000 by taking the different mesh sizes for comparing the moments that obtained by software with the spread sheets design moments.

6.3.1 No eccentricity

For this case 3 X 3 size footing and 0.35 X 0.35 m column is considered with load acting at centre @ 1350 kN and SBC of soil is 120 $\frac{kN}{m^2}$ with total depth of 800 mm

Spring stiffness provided as shown below:

- Internal Springs = $40 * SBC * Mesh\ Size$
- Edge Springs = $0.5 * 40 * SBC * Mesh\ Size$
- Corner Springs = $0.25 * 40 * SBC * Mesh\ Size$

The models of foundations with different mesh sizes showing the moment about Y axis @ face of the column are shown in following figures.

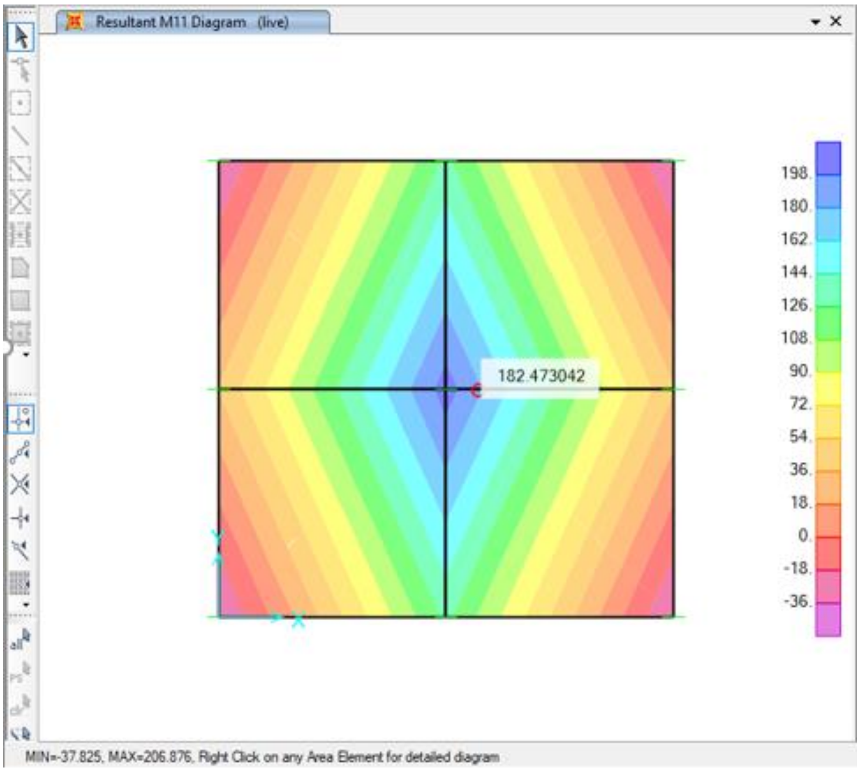


Figure 6. 11 Moment (kN-m/m) in foundation with $\frac{L}{2}\ X\ \frac{B}{2}$ Mesh size

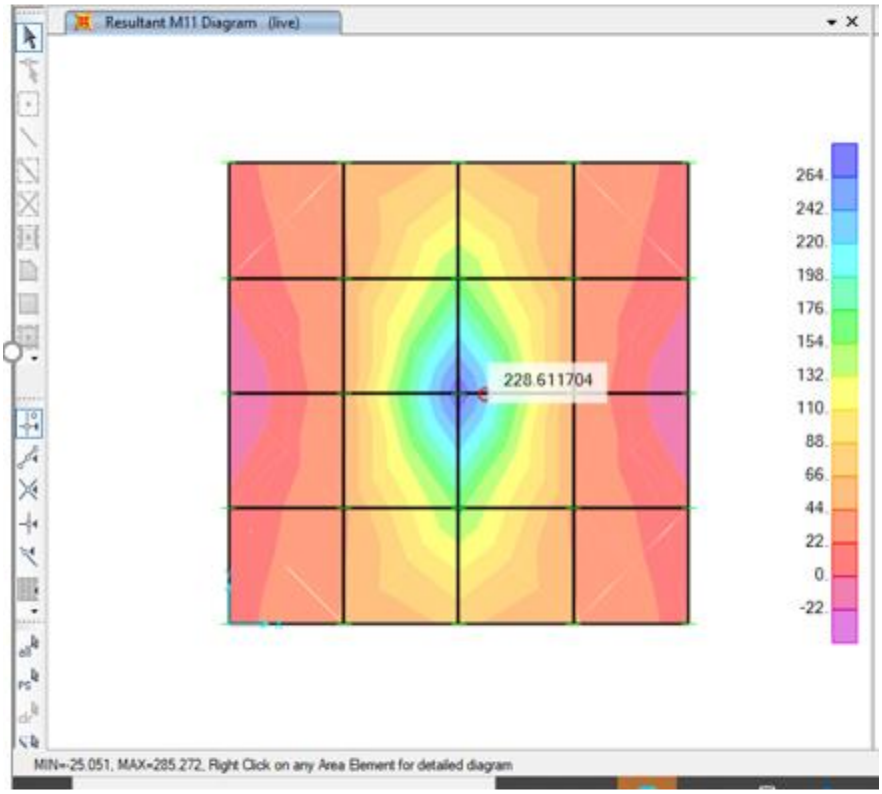


Figure 6. 12 Moment (kN-m/m) in foundation with $\frac{L}{4}\ X\ \frac{B}{4}$ Mesh size

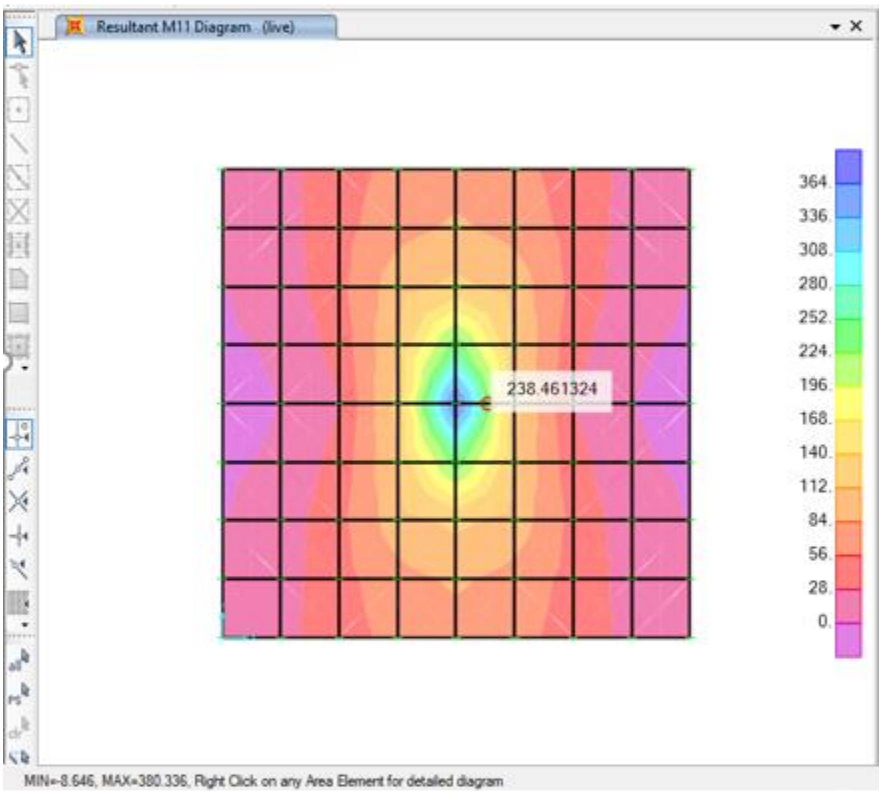


Figure 6. 13 Moment (kN-m/m) in foundation with $\frac{L}{8}\ X\ \frac{B}{8}$ Mesh size

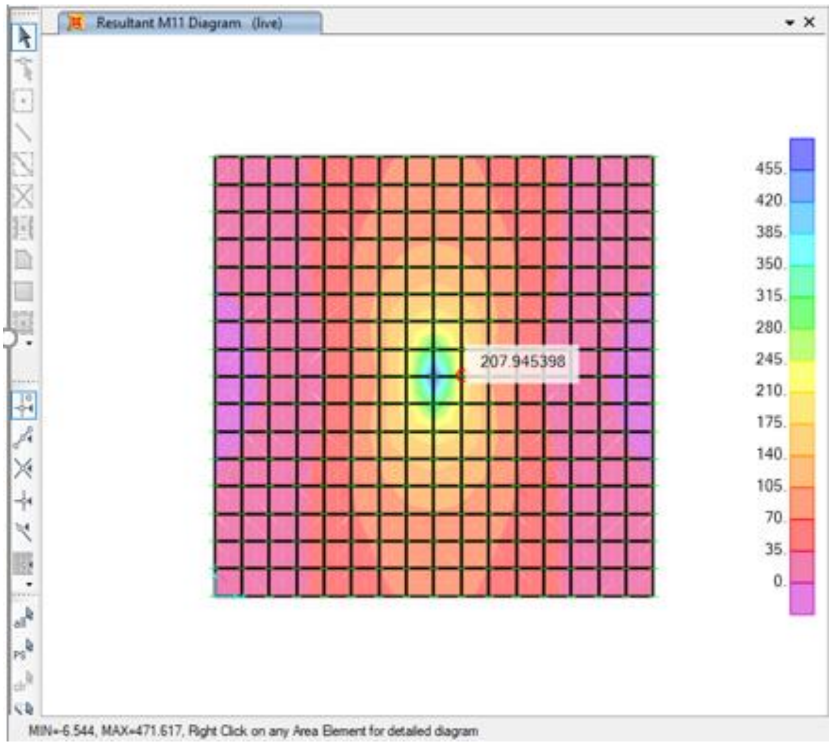


Figure 6. 14 Moment (kN-m/m) in foundation with $\frac{L}{16}\ X\ \frac{B}{16}$ Mesh size

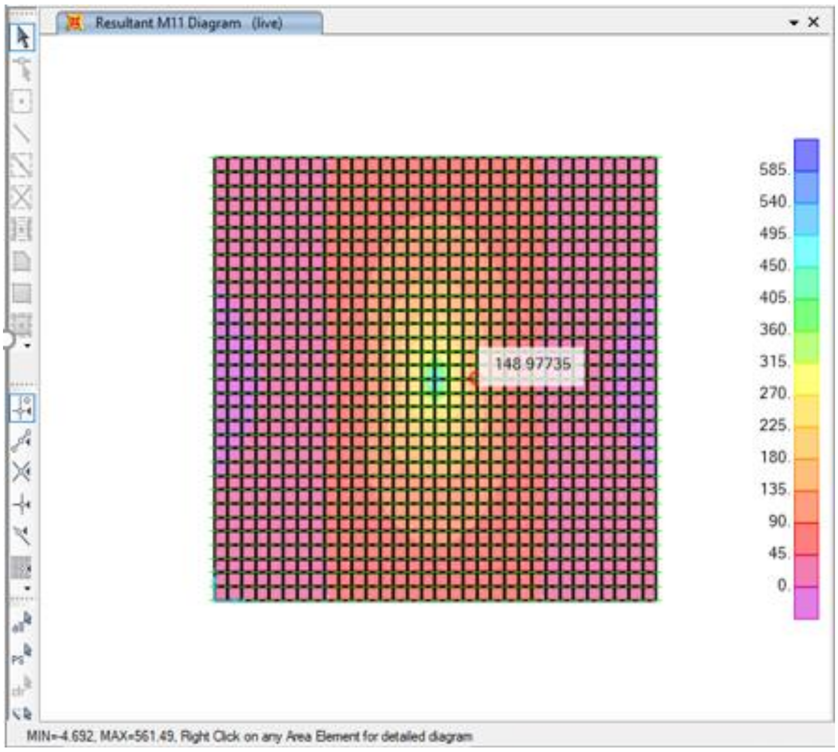


Figure 6. 15 Moment (kN-m/m) in foundation with $\frac{L}{32}$ X $\frac{B}{32}$ Mesh size

6.3.2 UNI-AXIAL:

For this case 5 X 5 size footing and 0.35 X 0.35 m column is considered with load acting at centre @ 4500 kN, Moment about Y axis 2250 kN m and SBC of soil is 180 $\frac{kN}{m^2}$ with total depth of 700 mm.

Spring stiffness provided as shown below:

- Internal Springs = $40 * SBC * Mesh\ Size$
- Edge Springs = $0.5 * 40 * SBC * Mesh\ Size$
- Corner Springs = $0.25 * 40 * SBC * Mesh\ Size$

The models of foundations with different mesh sizes showing the moment about Y axis @ face of the column are shown in following Figures.

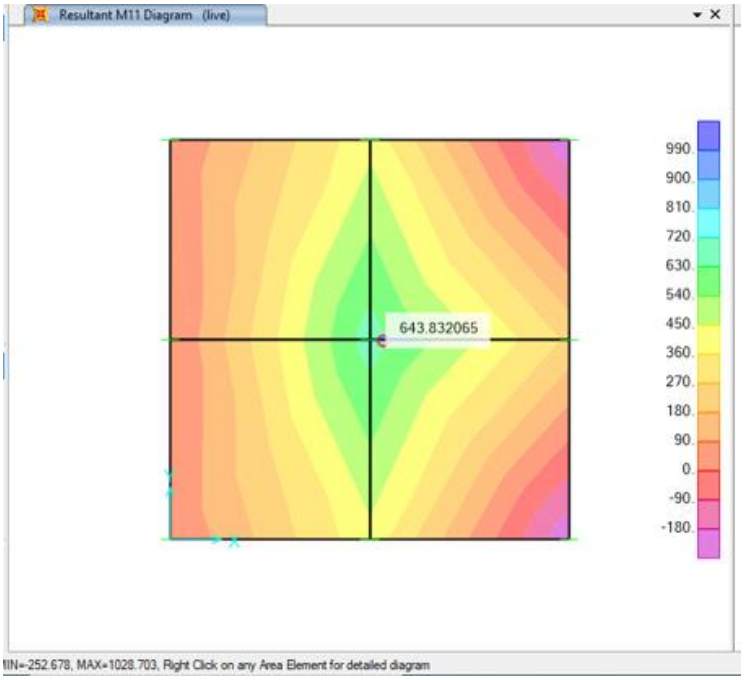


Figure 6. 16 Moment (kN-m/m) in foundation with $\frac{L}{2}$ X $\frac{B}{2}$ Mesh size

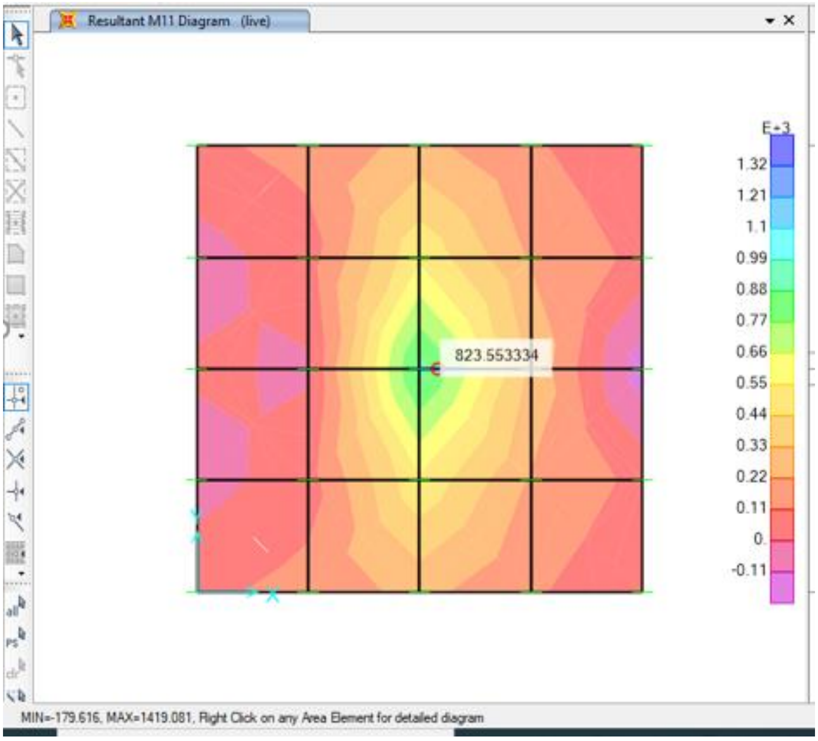


Figure 6. 17 Moment (kN-m/m) in foundation with $\frac{L}{4}$ X $\frac{B}{4}$ Mesh size

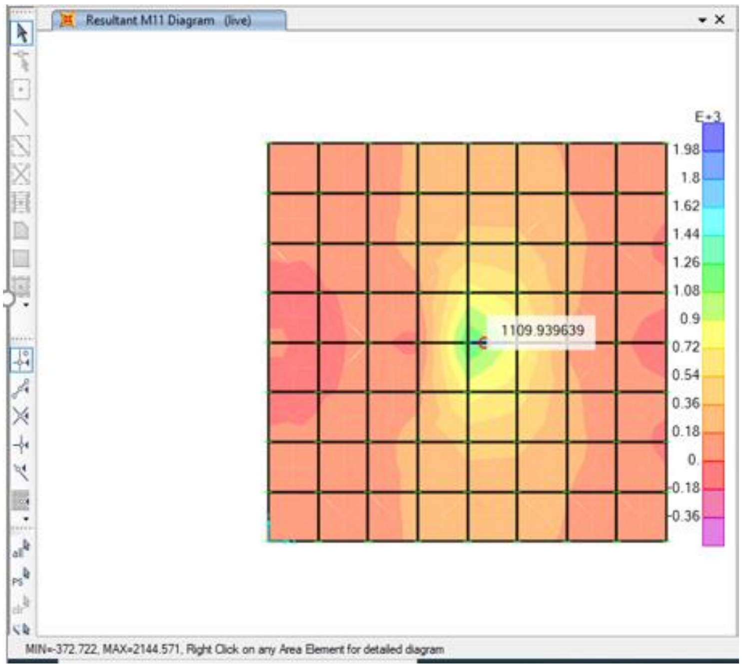


Figure 6. 18 Moment (kN-m/m) in foundation with $\frac{L}{8}$ X $\frac{B}{8}$ Mesh size

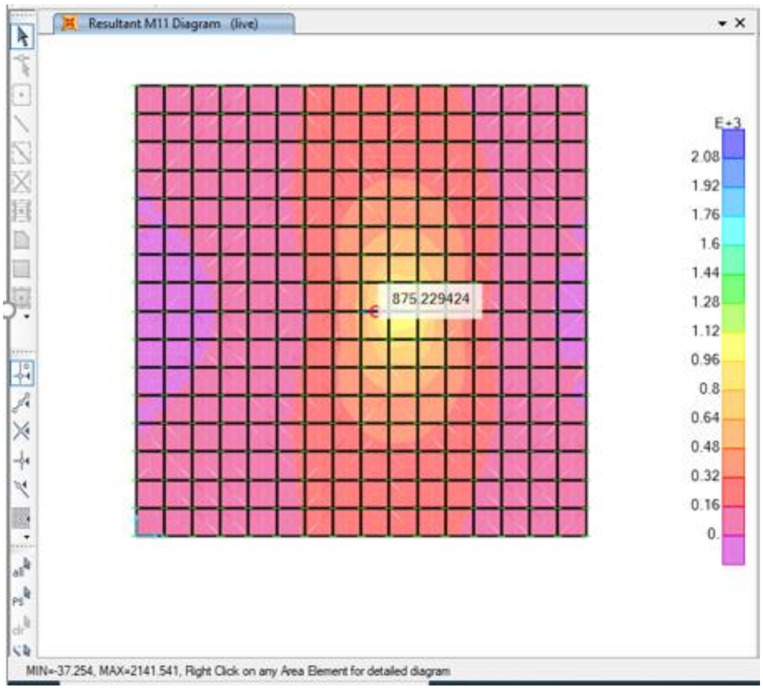


Figure 6. 19 Moment (kN-m/m) in foundation with $\frac{L}{16}$ X $\frac{B}{16}$ Mesh size

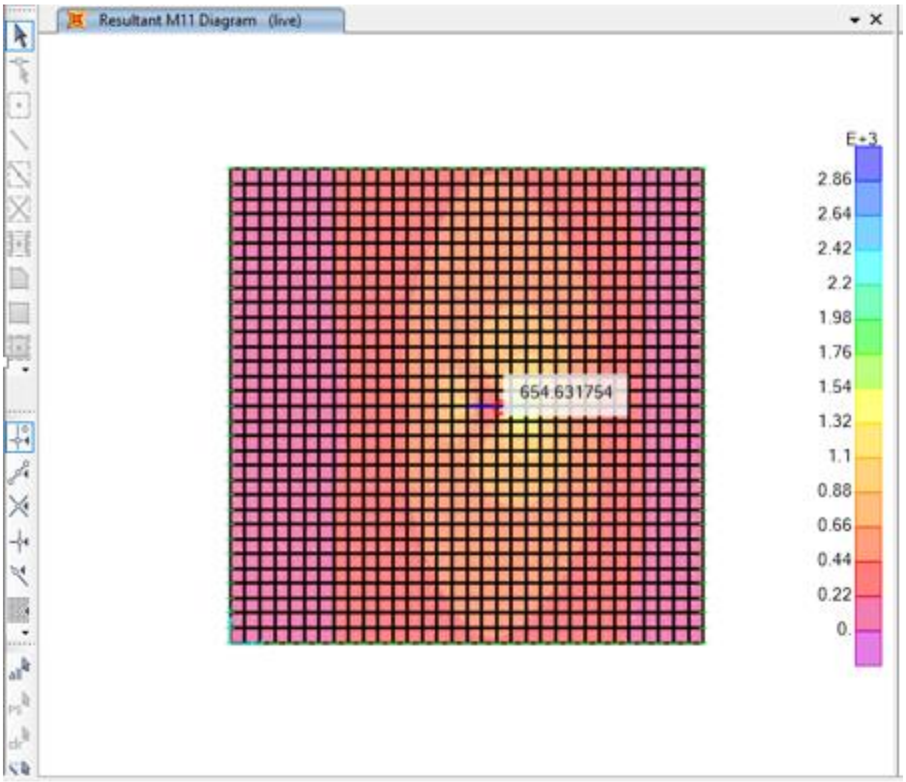


Figure 6. 20 Moment (kN-m/m) in foundation with $\frac{L}{32}$ X $\frac{B}{32}$ Mesh size

6.3.3 BI-AXIAL:

For this case 6 X 6 size footing and 0.45 X 0.45 m column is considered with load acting at centre @ 2250 kN, Moment about Y axis 1500 kN m, Moment about X axis 1500 kN m and SBC of soil is 150 $\frac{kN}{m^2}$ with total depth of 600 mm.

Spring stiffness provided as shown below:

- Internal Springs = $40 * SBC * Mesh\ Size$
- Edge Springs = $0.5 * 40 * SBC * Mesh\ Size$
- Corner Springs = $0.25 * 40 * SBC * Mesh\ Size$

The models of foundations with different mesh sizes showing the moment about Y axis @ face of the column are shown in following Figures.

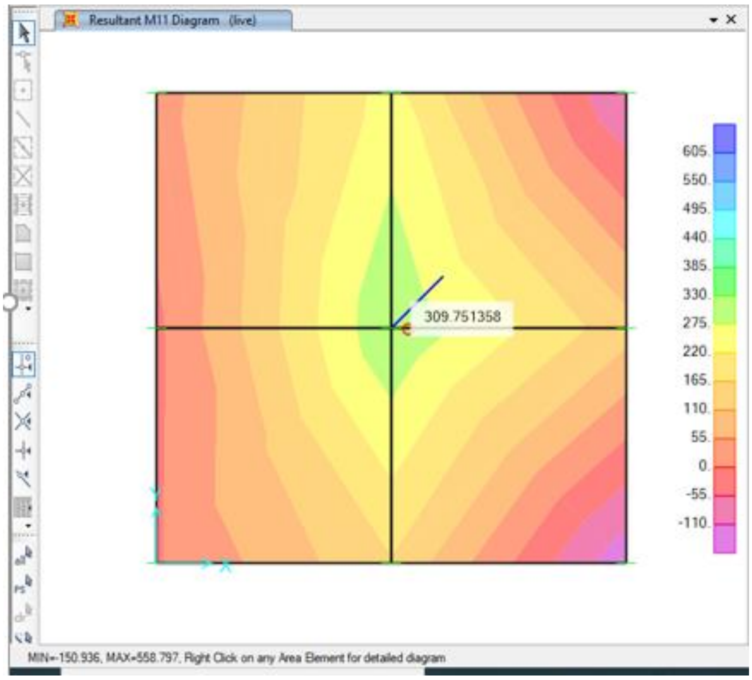


Figure 6. 21 Moment (kN-m/m) in foundation with $\frac{L}{2}$ X $\frac{B}{2}$ Mesh size

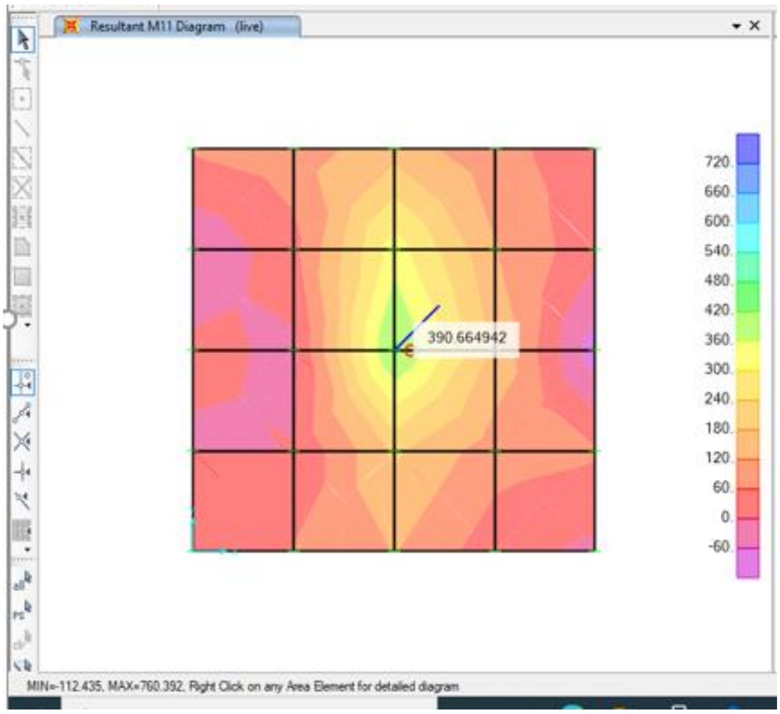


Figure 6. 22 Moment (kN-m/m) in foundation with $\frac{L}{4}$ X $\frac{B}{4}$ Mesh size

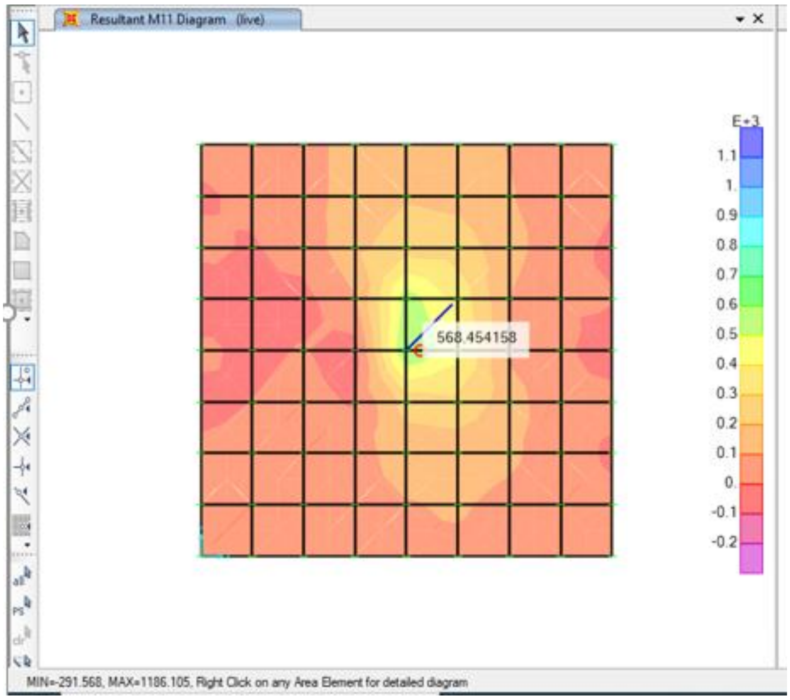


Figure 6. 23 Moment (kN-m/m) in foundation with $\frac{L}{8}$ X $\frac{B}{8}$ Mesh size

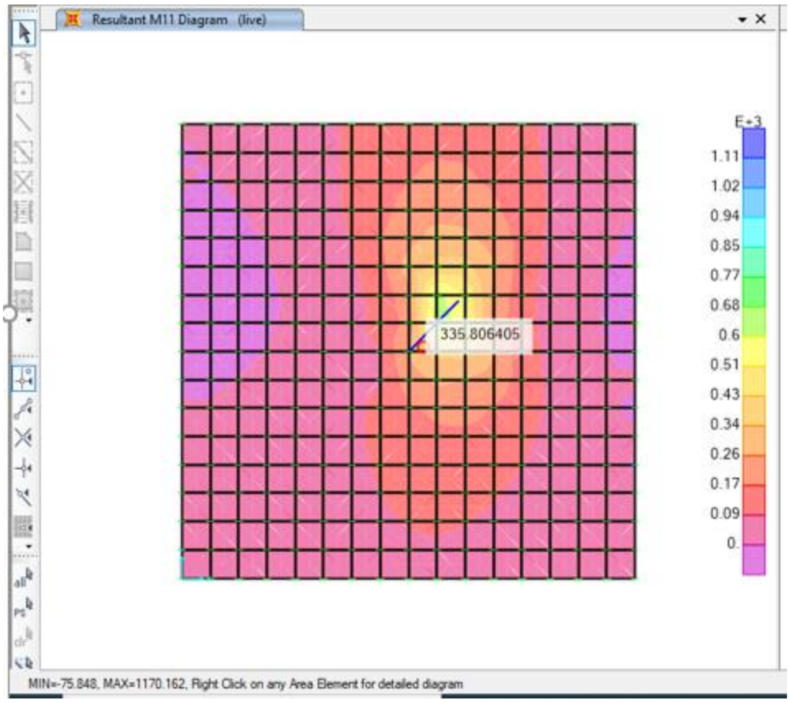


Figure 6. 24 Moment (kN-m/m) in foundation with $\frac{L}{16}$ X $\frac{B}{16}$ Mesh size

6.4 Study of effect of stiffness

The stiffness of the spring depends upon the maximum allowable displacement which relates with the subgrade reaction of soil and also depends upon types of soil and SBC. The subgrade reaction is considered referring the Bowles (1967), in this the author suggested and formulated the subgrade reaction (K_S) with SBC (q_a) of soil for different settlements.

He suggested following equations for respective settlements

$K_S = 40 * SBC$ For 25.4 mm settlement

$K_S = 160 * SBC$ For 6 mm settlement

$K_S = 83 * SBC$ For 12 mm settlement

$K_S = 50 * SBC$ For 20 mm settlement

Spring stiffness = $K_S * mesh\ size$

Using the above equation a footing is modelled in SAP2000 with load of 1350kN at the centre and footing size 3 X 3 m and column size 0.35 X 0.35 m with depth of 800 mm, the following figures shows the moment about Y axis at the face of column with spring stiffness of different settlements provided by the Bowles (1967). The inner springs are provided with this stiffness and edges are decreased by 50 % and corners by 75% of stiffness.

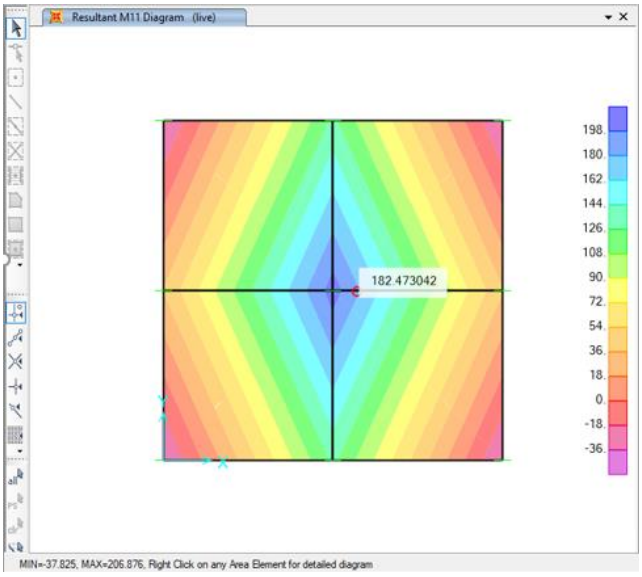


Figure 6. 25 Moment (kN-m/m) in foundation with $K_S = 40 * SBC$ **for** $\frac{L}{2}\ X\ \frac{B}{2}$ **Mesh size**

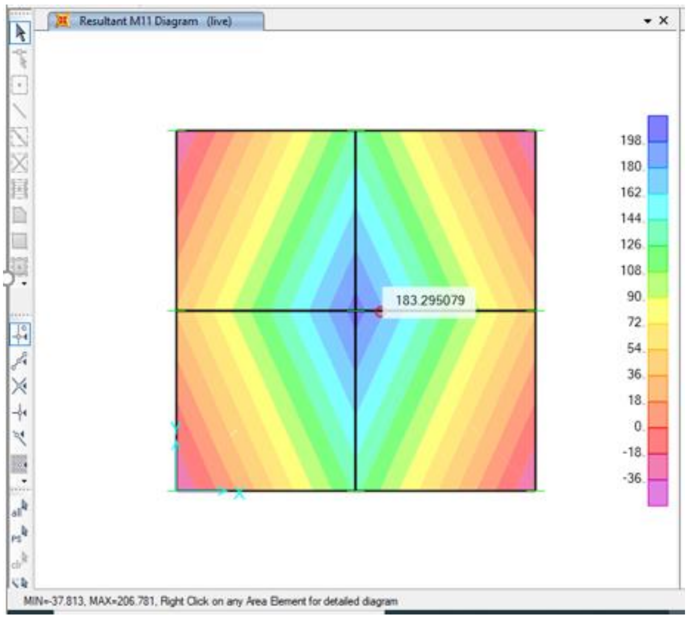


Figure 6. 26 Moment (kN-m/m) in foundation with $K_S = 50 * SBC$ **for** $\frac{L}{2}\ X\ \frac{B}{2}$ **Mesh size**

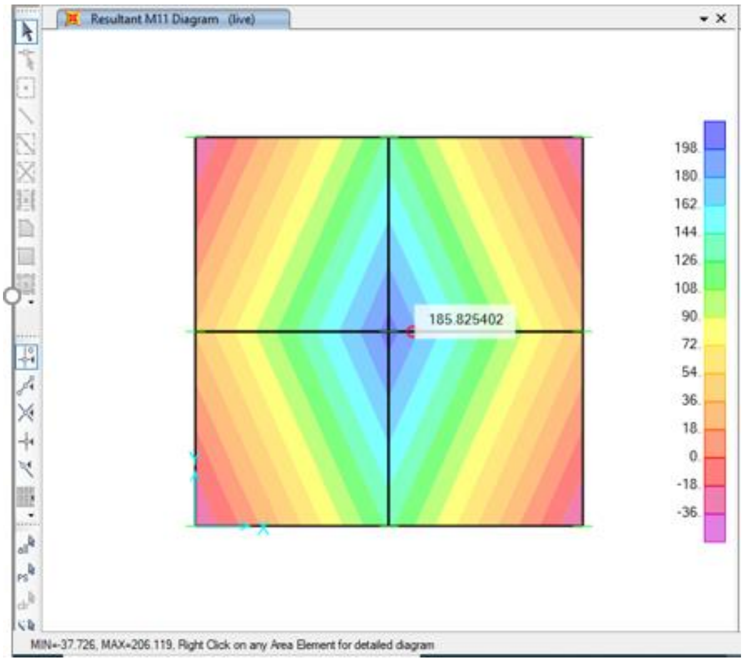
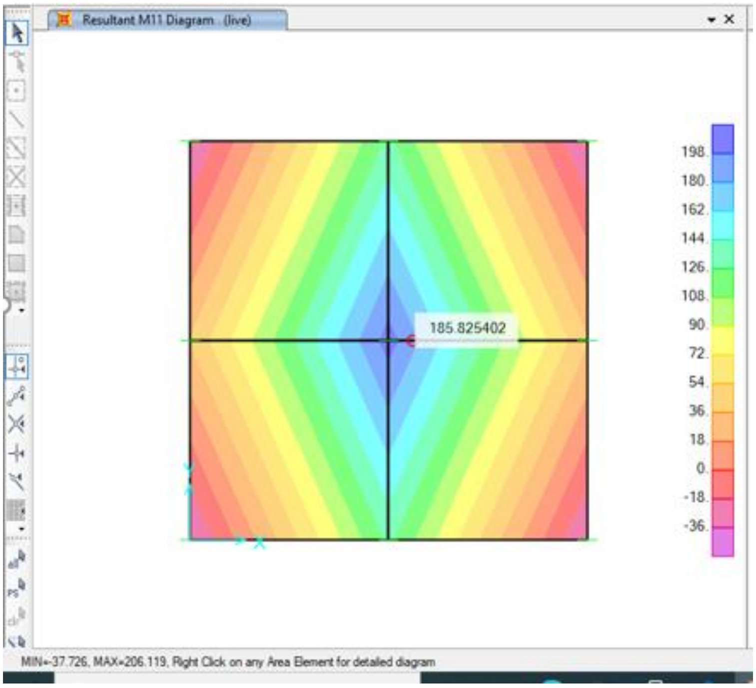


Figure 6. 27 Moment (kN-m/m) in foundation with $K_S = 83 * SBC$ for $\frac{L}{2}$ X $\frac{B}{2}$ Mesh size



Mesh Figure 6. 28 Moment (kN-m/m) in foundation with $K_S = 120 * SBC$ for $\frac{L}{2}$ X $\frac{B}{2}$ size

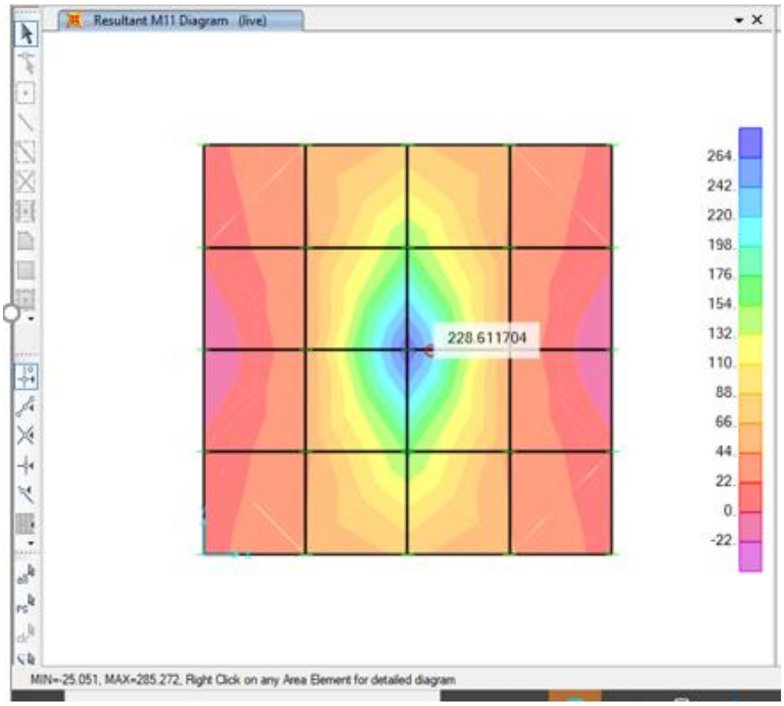


Figure 6. 29 Moment (kN-m/m) in foundation with $K_S = 40 * SBC$ for $\frac{L}{4}$ X $\frac{B}{4}$ Mesh size

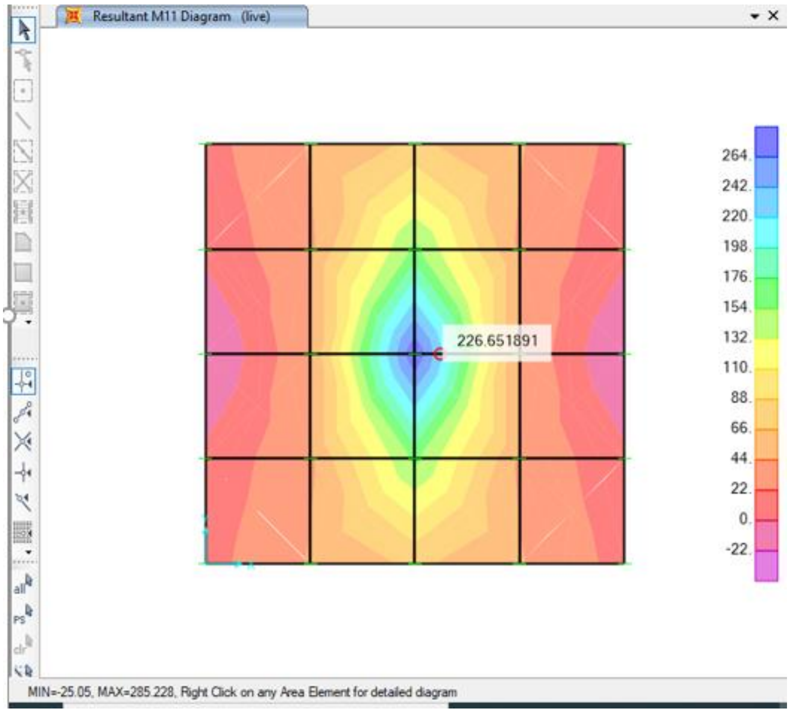


Figure 6. 30 Moment (kN-m/m) in foundation with $K_S = 50 * SBC$ for $\frac{L}{4}$ X $\frac{B}{4}$ Mesh size

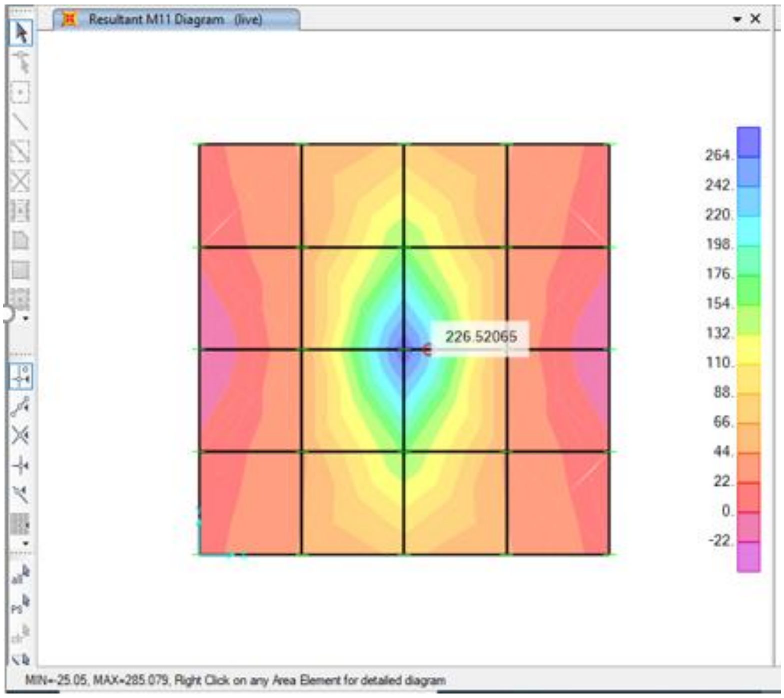


Figure 6. 31 Moment (kN-m/m) in foundation with $K_S = 83 * SBC$ for $\frac{L}{4}$ X $\frac{B}{4}$ Mesh size

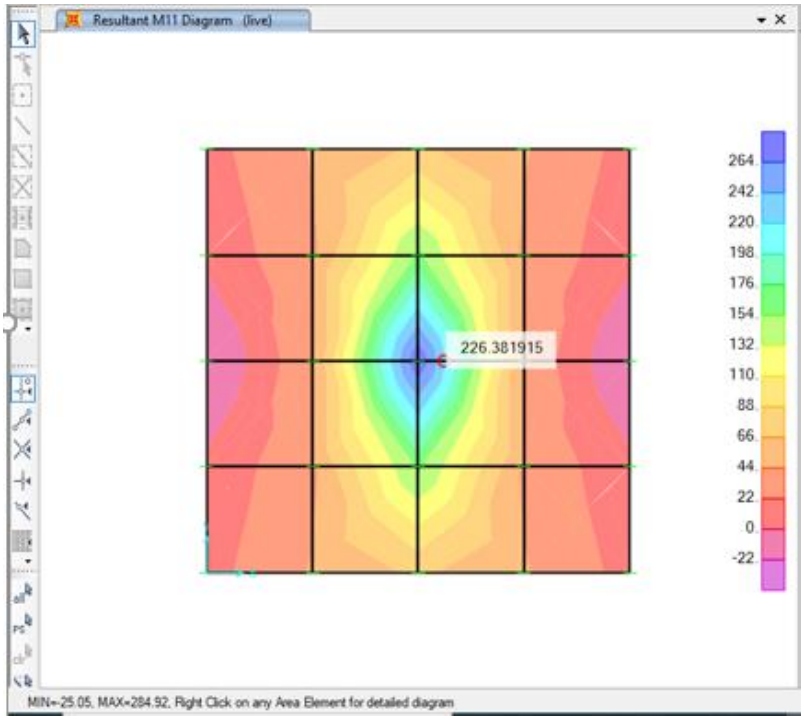


Figure 6. 32 Moment (kN-m/m) in foundation with $K_S = 120 * SBC$ for $\frac{L}{4}$ X $\frac{B}{4}$ Mesh size

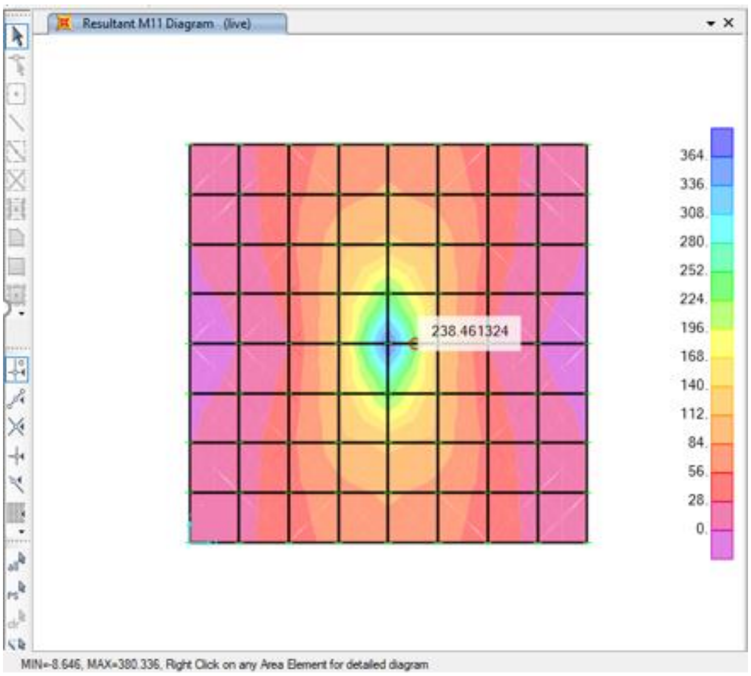


Figure 6. 33 Moment (kN-m/m) in foundation with $K_S = 40 * SBC$ for $\frac{L}{8}$ X $\frac{B}{8}$ Mesh size

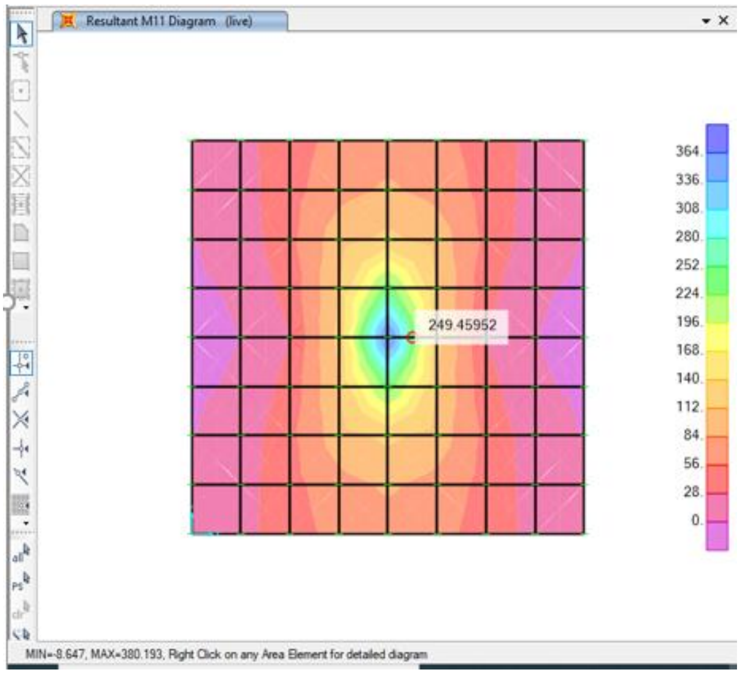


Figure 6. 34 Moment (kN-m/m) in foundation with $K_S = 50 * SBC$ for $\frac{L}{8}$ X $\frac{B}{8}$ Mesh size

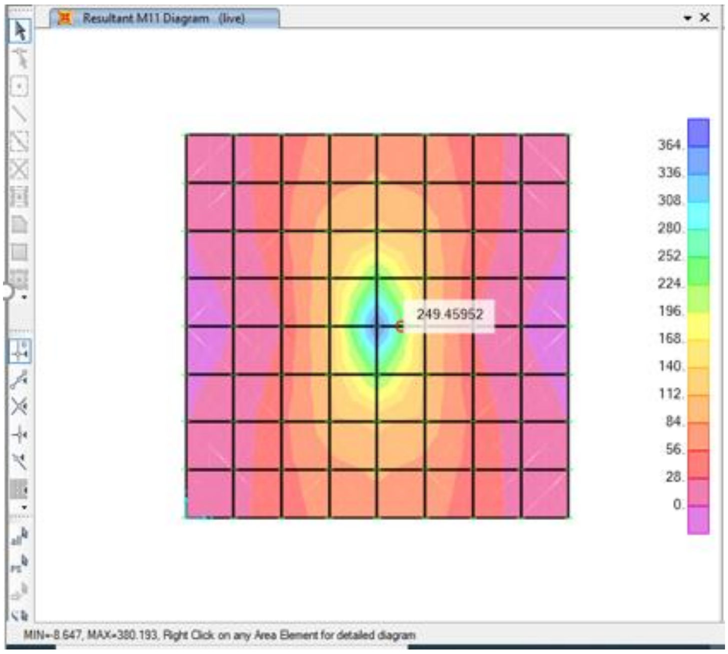


Figure 6. 35 Moment (kN-m/m) in foundation with $K_S = 83 * SBC$ for $\frac{L}{8}$ X $\frac{B}{8}$ Mesh size

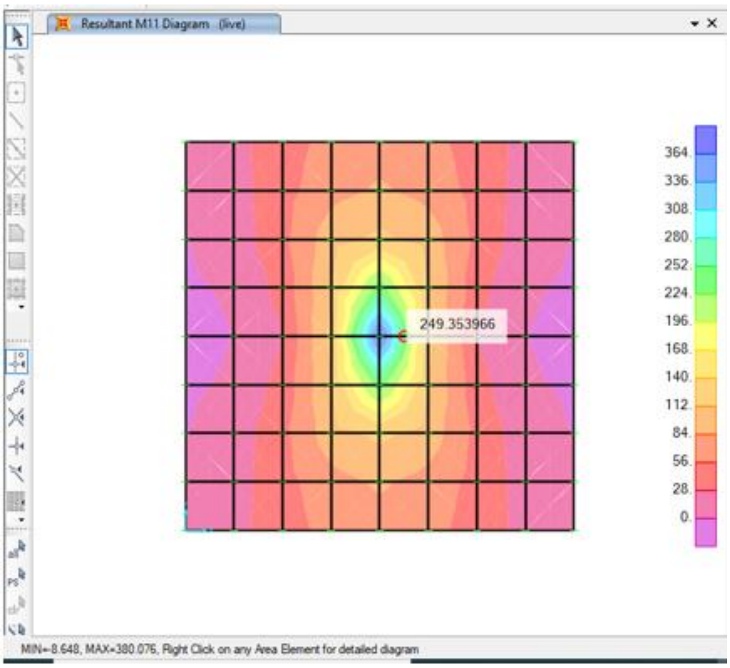


Figure 6. 36 Moment (kN-m/m) in foundation with $K_S = 120 * SBC$ for $\frac{L}{8}$ X $\frac{B}{8}$ Mesh size

6.5 Results

The following tables shows the value of moments about Y axis at the face of column or pier for the foundation with no eccentricity, uniaxial and bi axial cases respectively with the different mesh sizes.

Table 6. 1 Moments about Y (kN-m/m) @ face for different Mesh Size

S.NO	CASE	MESH SIZES					Manual
		$\frac{L}{2}X\frac{B}{2}$	$\frac{L}{4}X\frac{B}{4}$	$\frac{L}{8}X\frac{B}{8}$	$\frac{L}{16}X\frac{B}{16}$	$\frac{L}{32}X\frac{B}{32}$	
1	NO ECCENTRICITY	182	222	256	208	149	132
2	UNI AXIAL	643	823	1109	875	654	643
3	BI AXIAL	310	391	569	335	290	336

Table 6. 2 Moments about Y (kN-m/m) @ face for different K_s values

S.NO	CASE	MESH SIZE	SUB GRADE REACTION COEFFICIENT			
			40	50	83	120
1	No Eccentricity	$\frac{L}{2}X\frac{B}{2}$	182	184	185	185
2		$\frac{L}{4}X\frac{B}{4}$	222	226	226	227
3		$\frac{L}{8}X\frac{B}{8}$	256	252	250	250

It is observed that the model containing the mesh size much lesser than the column size in isolated footings gives the accurate results. It is also observed that there is no significant change in moment values when the foundation modelled with different spring constants stated by Bowles (1967).

CHAPTER 7

7 Conclusions

- In this study the excel spreadsheets for design of substructure components were prepared. In the pier chapter the spread sheet for calculation of forces on circular or rectangular pier is prepared. For design using VBA code P-M interaction curves for the respective input were prepared which helps in design in uniaxial and biaxial bending and respective checks also done using spread sheets. A graph is plotted for comparison of IS 456 and IRC 112 P-M interaction curve of a particular example.
- In the well foundation chapter excel spread sheets were prepared for design of components of well like well steining, well cap, top plug etc and for stability analysis of well foundation separate spread sheet prepared.
- In the pile foundation Excel chapter spread sheets were prepared for the calculation of pile capacities and pile cap design, in this spread sheet it calculate the forces in piles and do respective checks.
- In the spread foundation chapter excel spread sheets were prepared for design of different cases of spread foundation like no eccentricity, uniaxial and biaxial bending, in each spread sheet the maximum and minimum forces are calculated and moment is calculated respectively.
- In the pier chapter from the comparison curve Figure 3.8 of IS 456-2000 and IRC 112-2011 it can be seen that there is slight difference in the curves in maximum portion and IS-456-2000 curve is inwards of IRC 112-2011 curve.
- In the well foundation chapter from the Figure 4.3 & Figure 4.4 it can be concluded that the stress and maximum moment in the well steining is 10% - 20% more than that calculated manually.
- In the pile foundation chapter from the Figure 5.9 it can be concluded that the Moment about Y axis @ face of the column is 10-20% more than that calculated manually.
- In the spread foundation chapter from Table 6.2 it can be seen that the moments for particular type model doesn't varies much as the spring stiffness (which depends on subgrade reaction) varies, so one can consider the spring stiffness with any of the

particular settlement proposed by Bowles (1995) for getting the accurate moments from the SAP2000 modelling.

- It can be seen that the value of the moment at the face of the column for three cases no eccentricity, uniaxial and bi axial with mesh size much lesser than column size is nearly equal to moments obtained from manual calculations.

7.1 Future scope

1. P-M interaction curves for different type of cross sections of piers like circular can be developed.
2. Different points on the P-M curve can be plotted such that it can show amount and type eccentricity occurred, like e_{min} ,e_o, e_{bal}.
3. The input of different load details in piers from superstructure analysis can be linked to developed spread sheet, such that the design of pier will be handy and accurate.
4. In foundation design spread sheets the load details from the analysis of pier can be linked and design can be made easier.
5. Inputs in spread sheet that are given manually from IS codes can be made automated such that the design will be easier.

ANNEXURE A: VBA CODE

The following code is used for calculation of axial and moment values for given cross section at each step value of $\frac{x}{h}$. This values are used for developing the P-M curves in both bending directions. The calculated axial and moment values are for the steel layers in the cross section.

CODE:

```
Sub graph()
GCON = Cells(3, 2).Value                                'grade of concrete
GSTEEL = Cells(2, 4).Value                              'grade of steel
NB = Cells(21, 4).Value                                 'no of bars parallel to width
ND = Cells(22, 4).Value                                 'no of bars parallel to depth
COV = Cells(11, 3).Value                                ' cover at top
SP = Cells(20, 2).Value                                 'spacing provided
AREA = 3.14 * 0.25 * Cells(10, 3).Value * Cells(10, 3).Value     'area of one bar
B = Cells(8, 3).Value                                   ' width
D = Cells(9, 3).Value                                   ' depth
For x = 1 To 15                                         'x/h values count
  FORCE = 0
  MOMENT = 0
  xh = x * 0.2                                          'step value for x/h
  For i = 1 To ND                                       'representing layer number
    If i = 1 Then                                       'top layer
      ddash = COV
      STRAIN = 0.0035 * (1 - (ddash / D) / xh)
      STRESS = STRAIN * 2 * 10 ^ 5
      If STRESS > GSTEEL / 1.15 Then
       STRESS = GSTEEL / 1.15
```

```
  End If
 FORCE = FORCE + STRESS * (NB + 2) * AREA
 MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash)
End If
If i = ND Then                                          'representing bottom layer
 ddash = COV + (ND - 1) * SP
 If xh > ddash / D Then
  STRAIN = 0.0035 * (1 - (ddash / D) / xh)
  STRESS = STRAIN * 2 * 10 ^ 5
  If STRESS > GSTEEL / 1.15 Then
   STRESS = GSTEEL / 1.15
   End If
   FORCE = FORCE + STRESS * (NB + 2) * AREA
   If xh < ddash / D And ddash / D < 0.5 Then
     MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
   End If
   If xh > ddash / D And ddash / D > 0.5 Then
     MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
   Else
     MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
   End If
 Else                                              'representing other layers of steel
  STRAIN = 0.0035 * (1 - (ddash / D) / xh)
  STRESS = -STRAIN * 2 * 10 ^ 5
   If STRESS > GSTEEL / 1.15 Then
   STRESS = GSTEEL / 1.15
   End If
   FORCE = FORCE - STRESS * (NB + 2) * AREA
```

```
     If xh < 0.5 And xh < ddash / D And ddash / D < 0.5 Then
      MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    End If
    If xh > 0.5 And xh > ddash / D And ddash / D > 0.5 Then
      MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    Else
      MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    End If
   End If
 End If
If i > 1 And i < ND Then
  ddash = COV + (i - 1) * SP
  If xh > ddash / D Then
   STRAIN = 0.0035 * (1 - (ddash / D) / xh)
   STRESS = STRAIN * 2 * 10 ^ 5
   If STRESS > GSTEEL / 1.15 Then
    STRESS = GSTEEL / 1.15
   End If
    FORCE = FORCE + STRESS * (NB + 2) * AREA
    If xh < 0.5 And xh < ddash / D And ddash / D < 0.5 Then
      MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    End If
    If xh > 0.5 And xh > ddash / D And ddash / D > 0.5 Then
      MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    Else
      MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    End If
  Else
```

```
STRAIN = 0.0035 * (1 - (ddash / D) / xh)
STRESS = -STRAIN * 2 * 10 ^ 5
 If STRESS > GSTEEL / 1.15 Then
  STRESS = GSTEEL / 1.15
 End If
 FORCE = FORCE - STRESS * (2) * AREA
If xh < 0.5 And xh < ddash / D And ddash / D < 0.5 Then
   MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
 End If
 If xh > 0.5 And xh > ddash / D And ddash / D > 0.5 Then
   MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
 Else
   MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
 End If
End If
End If


Next i
Cells(26 + x, 4).Value = FORCE
Cells(26 + x, 5).Value = MOMENT
Next x
End Sub
```

The following code is similar to the above one but this code is used for calculation of the axial and moment values for balanced section.

CODE:

```
Sub graph2()
GCON = Cells(3, 2).Value                              'grade of concrete
GSTEEL = Cells(2, 4).Value                            'grade of steel
NB = Cells(21, 4).Value                               'no of bars parallel to width
ND = Cells(22, 4).Value                               'no of bars parallel to depth
COV = Cells(11, 3).Value                              ' cover at top
SP = Cells(20, 2).Value                               'spacing provided
AREA = 3.14 * 0.25 * Cells(10, 3).Value * Cells(10, 3).Value    'area of one bar
B = Cells(8, 3).Value                                 ' width
D = Cells(9, 3).Value                                 ' depth
 FORCE = 0
 MOMENT = 0

 xh = 0.8 * 0.617                                     'x/h value of balanced section
 For i = 1 To ND                                      'representing layer number
  If i = 1 Then                                       'top layer
   ddash = COV
   STRAIN = 0.0035 * (1 - (ddash / D) / xh)
   STRESS = STRAIN * 2 * 10 ^ 5
   If STRESS > GSTEEL / 1.15 Then
    STRESS = GSTEEL / 1.15
    End If
   FORCE = FORCE + STRESS * (NB + 2) * AREA
   MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash)
  End If
  If i = ND Then                                      'representing bottom layer
```

```
ddash = COV + (ND - 1) * SP
If xh > ddash / D Then
 STRAIN = 0.0035 * (1 - (ddash / D) / xh)
 STRESS = STRAIN * 2 * 10 ^ 5
 If STRESS > GSTEEL / 1.15 Then
  STRESS = GSTEEL / 1.15
  End If
  FORCE = FORCE + STRESS * (NB + 2) * AREA
  If xh < ddash / D And ddash / D < 0.5 Then
    MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
  End If
  If xh > ddash / D And ddash / D > 0.5 Then
    MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
  Else
    MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
  End If
Else                                              'representing other layers of steel
 STRAIN = 0.0035 * (1 - (ddash / D) / xh)
 STRESS = -STRAIN * 2 * 10 ^ 5
  If STRESS > GSTEEL / 1.15 Then
  STRESS = GSTEEL / 1.15
  End If
  FORCE = FORCE - STRESS * (NB + 2) * AREA
   If xh < 0.5 And xh < ddash / D And ddash / D < 0.5 Then
    MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
  End If
  If xh > 0.5 And xh > ddash / D And ddash / D > 0.5 Then
    MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
```

```
    Else
      MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
    End If
   End If
  End If
 If i > 1 And i < ND Then
   ddash = COV + (i - 1) * SP
   If xh > ddash / D Then
    STRAIN = 0.0035 * (1 - (ddash / D) / xh)
    STRESS = STRAIN * 2 * 10 ^ 5
    If STRESS > GSTEEL / 1.15 Then
     STRESS = GSTEEL / 1.15
    End If
     FORCE = FORCE + STRESS * (NB + 2) * AREA
     If xh < 0.5 And xh < ddash / D And ddash / D < 0.5 Then
       MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
     End If
     If xh > 0.5 And xh > ddash / D And ddash / D > 0.5 Then
       MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
     Else
       MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
     End If
   Else
    STRAIN = 0.0035 * (1 - (ddash / D) / xh)
    STRESS = -STRAIN * 2 * 10 ^ 5
     If STRESS > GSTEEL / 1.15 Then
      STRESS = GSTEEL / 1.15
     End If
```

```
        FORCE = FORCE - STRESS * (2) * AREA
       If xh < 0.5 And xh < ddash / D And ddash / D < 0.5 Then
          MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
        End If
        If xh > 0.5 And xh > ddash / D And ddash / D > 0.5 Then
          MOMENT = MOMENT - Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
        Else
          MOMENT = MOMENT + Abs(STRESS * (NB + 2) * AREA * (0.5 * D - ddash))
        End If
       End If
     End If

     Next i
    Cells(26 + x, 4).Value = FORCE
    Cells(26 + x, 5).Value = MOMENT
End Sub
```

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