SEISMIC BEARING CAPACITY OF STRIP FOOTING ON REINFORCED SOIL SUBJECTED TO ECCENTRIC INCLINED LOAD

A DISSERTATION

SUBMITTED IN THE PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF DEGREE

Of

MASTER OF TECHNOLOGY

In

EARTHQUAKE ENGINEERING

(With Specialization in Soil Dynamics)

By

ABHIMANYU YADAV

(14525001)

DEPARTMENT OF EARTHQUAKE ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY, ROORKEE

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CANDIDATE'S DECLARATION

I hereby declare that the work carried out in this dissertation titled **"SEISMIC BEARING CAPACITY OF STRIP FOOTING ON REINFORCED SOIL SUBJECTED TO ECCENTRIC INCLINED LOAD"** is presented on behalf of partial fulfillment of the requirement for the award of the degree of **MASTER OF TECHNOLOGY** with specialization in **SOIL DYNAMICS** submitted to the department of **EARTHQUAKE ENGINEERING, INDIAN INSTITUTE OF TECHNOLOGY, ROORKEE** under the supervision and guidance of **Dr. DAYA SHANKER**, Assistant Professor, Department of Earthquake Engineering, IIT Roorkee, India.

I have not submitted the matter embodied in this report for the award of any other degree or diploma

Date- MAY 15, 2015

Place- Roorkee **ABHIMANYU YADAV**

CERTIFICATION

This is to certify that the above statement made by the candidate is correct to the best of our knowledge and belief.

Dr. DAYA SHANKER
Assistant Professor
Department of Earthquake Engineering,
IIT Roorkee, Roorkee-247667

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Place- Roorkee

ABHIMANYU YADAV
SOIL DYNAMICS
14525001

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ABSTRACT

Due to the increase in population, demand for construction increases and the use of poor soils becomes absolutely necessary. Settlement and soil bearing capacity play an important role in the design of foundation. Seismicity of the site is another vital parameter in design of the foundation for a structure. Therefore, seismic bearing capacity of soil becomes an important component in the design. In poor soils, ground improvement techniques are commonly used to improve the soil bearing capacity. If poor soil is improved by using geo-synthetic, then it becomes feasible to use shallow foundations instead of deep foundations for the same structure, thus effecting economy.

Through this study, we analysis the seismic bearing capacity of strip footing on reinforced soil subjected to eccentric inclined load. This approach is based on the analysis proposed by Binquet and Lee (1975) for a strip footing subjected to static load. Due to eccentric inclined loading, three types of load distribution acting across the strip footing. We found out the normal stress and shear stress at any point below the footing by superimposition of stresses acting on strip footing due to a uniform vertical load, a triangular vertical load and a horizontal seismic load.

Plots of non-dimensional parameters I_Z, J_Z and M_Z prepared for different cases of seismic acceleration, eccentricity and inclination angles. Bearing capacity of strip footing with different layers of reinforcing ties on earth bed is found out.

CHAPTER -1
INTRODUCTION

1.1 OVERVIEW

Seismic bearing capacity is defined as the maximum net intensity of loading that can be imposed on the soil with no possibility of shear failure or possibility of excessive settlement during the earthquake.

A reinforced earth bed is defined as a soil foundation system containing horizontally bedded thin flat metal strips or ties. Reinforcing soil is used improve its bearing capacity is a very old concept. From literature review it was supposed that this concept was as old as about 3000 years. It was used in Ziggurats, Babylonian temples more than 3000 years ago. But in the modern context this concept gained its importance in early 1970's.

Vidal was the first person in modern times to come up with the idea of reinforcing soil. He used this concept to improve the bearing capacity of footings.

The need to reinforce soil come with the problems faced by designers when they have to deal with places with low bearing capacity of soil and also where there is availability of clay deposit in the top layer. Due to these conditions structure has to undergo excessive settlement. This leads to damage in structure, reduction in durability of structure and also the performance level of structure decreases to the larger extent.

The traditional options are available to overcome this problem are--

- Pile foundation through a weak soil deposit.
- Excavation and replacement of soil with suitable soil.
- Pre-consolidation of soil deposit.
- Stabilizing with injected additive.
- Applying some techniques for densification of soil.
- Increasing the dimensions of the footing.

But all these method are expensive either due to process involved in it or due to need of skilled labour. Also the duration of project is increased vastly.

The appropriate and alternative solution to this problem is to reinforce soil with some reinforcing element so that above mentioned problems can be overcome .Reinforcing soil with geo-synthetic is one such alternative. This is a type of ground improvement technique only. It is used to improve the bearing capacity of the soil. This is being placed horizontally.

Geo-synthetic is defined as a planar, polymeric (synthetic or natural) material used in contact with soil/rock and/or any other geotechnical material in civil engineering applications.

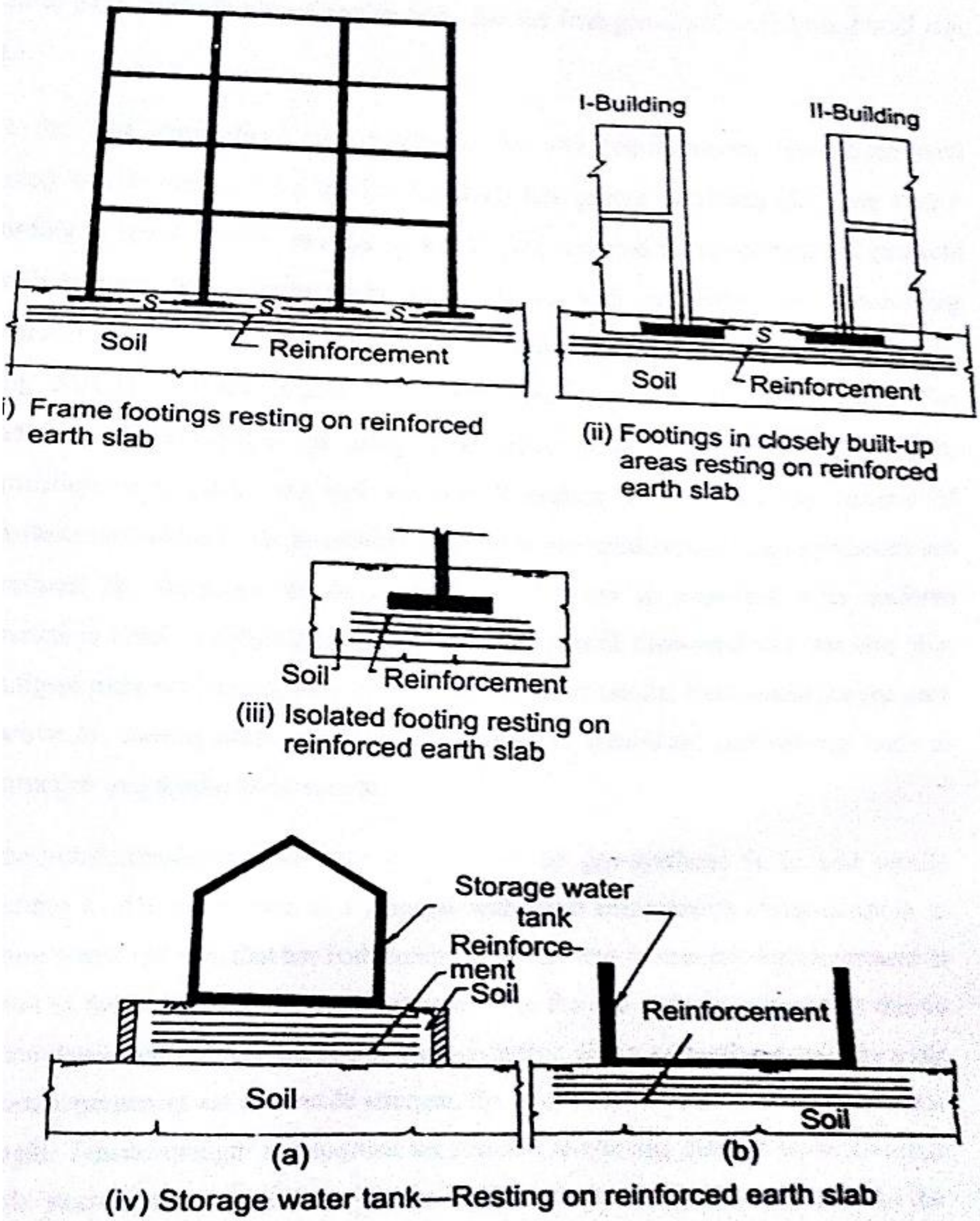


Fig.1.1: Applications of reinforced earth technique to foundation problems

These materials can perform following functions: Separation, filtration, protection, hydraulic barrier, surface erosion control and reinforcement (to resist stresses and contain deformation in geotechnical structures).

A reinforced earth bed is defined as a soil foundation system containing horizontally bedded thin flat metal strips or ties. These are being placed in free drainage granular soils as good bonding is being required between the ties and the soil. This bonding is in terms of frictional force that is being acting in between ties and soil. This placing of strips in soil provides tensile strength to the soil. It is well known fact that soil can withstand compressive forces but it is very poor in taking tensile forces. Thus, geo-synthetic takes care of tensile forces that are coming on soil.

1.2 SCOPE OF WORK

With passage of time many investigators studied the behaviour of footings resting on reinforced soil as well as evaluate seismic bearing capacity of footing. Some of these investigators are: Binquet and Lee (1975),Saran and Talwar(1981),Sreekanieth H.R (1987),Al-Karni et al.(1993), Apoorva A.(2012), Wasim A.(2014).

To compute the seismic bearing capacity of strip footing on reinforced soil bed subjected to eccentric inclined load, first formulation of stress equations in parametric forms has been done, to reduce the calculations. Non dimensional parameters plots have been prepared corresponding to the shear and normal stress.

The seismic bearing capacity of strip footing in eccentric inclined case compared with static as well as dynamics case of static vertical loading. The comparison of bearing capacity has made with respect to number of layers of reinforcement provided.

1.3 ORGANIZATION OF THE THESIS

The report of this dissertation consists of four chapters. This report has been organised as shown below:

1. **CHAPTER 1** deals with introduction, scope and organization of report.
2. **CHAPTER 2** deals with review of literature on bearing capacity of strip footing on reinforced soil bed.
3. **CHAPTER 3** deals with formulation of the stress equations for eccentric inclined load for finding seismic bearing capacity.
4. **CHAPTER 4** deals with conclusion of the thesis.

CHAPTER-2
LITERATURE REVIEW

2.1 General

Since the pre-historic times, the techniques of soil improvement have gained application. In order to enhance the quality of the soil condition, the soil improvement techniques had become extremely necessary. The necessities of improving the soil lead to the increase in the development of the soil improvement techniques. With the advancement of science and technology, extensive research started to be carried out in the field of soil improvement.

There is a vast area in which research has been done in the case of ground improvement techniques all across the world. However, most of the researchers advocate the use of geosynthetics, as the use of geosynthetics is more convenient, speedy and it can readily used in any type of weather conditions. But the most important aspect of using geosynthetics is that they can be used for very poor soils also, which are not improved by any other conventional method. Moreover, the use of geosynthetics in convenient to clients, designers, contractors as well as engineers.

The main findings of these researchers were that by preparing a suitable reinforced earth bed, the ultimate bearing capacity of the footing can be increased by 3 to 4 times and the settlement/tilt can also be brought down to 30% for the same footing resting on unreinforced soil bed. Researchers also found out the use of geo-synthetics, according to their need, finance and utility.

2.2 Binquet and Lee(1975)

Binquet and Lee (1975) gave a theoretical analysis to find out the pressure intensity of isolated strip footing resting on reinforced earth slab for a given settlement. A mechanistic working hypothesis regarding significant movement and load transfer has been assumed which allows for the determination of maximum tie force, T_D. The footing and Zone I (fig.1.2) of soil, bounded laterally by the loci of maximum shear are assumed to move down, forcing the soil in Zone II to move outwards. The reinforcement stresses τ_{XZmax} are assumed to undergo two right angled bends at the failure surface so that the tie force is directed upwards at the failure surface. Equilibrium of an element of soil, encompassing a reinforced strip, in Zone I yield the value of tie force which is compared with the tensile strength and frictional resistance of strip length in Zone II to determine the bearing capacity ratio in the rupture and pull out failures respectively. The exercise is carried out for all the strips and the minimum value of bearing ratio and the corresponding failure mode is obtained. Comparison of analytical values with experimental results (Binquet and Lee, 1975) showed a fairly good agreement for the other two soil particles. The method requires bearing capacity of unreinforced soil in order to compute that of reinforced soil and reliability of the latter is dependent on the accuracy of the former.The failure plane is the locus of the points of maximum shear stress as shown in fig(2.1)

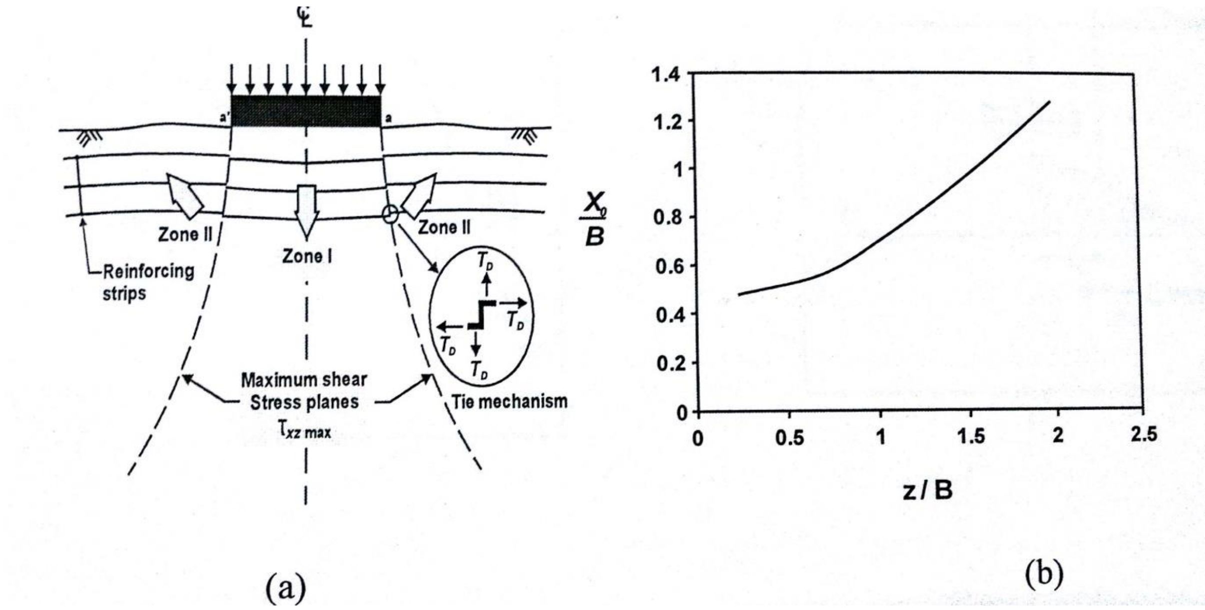


FIG 2.1 Locus of failure plane on either side of footing (Binquet and Lee 1975)

The values of non dimensionless stress parameters for a given depth have been computed and plotted as shown in fig.(2.3).The values of normal and shear stresses at different locations have been calculated using following Boussinesq's equations:

$$\sigma_Z = \frac{q}{\pi}[\alpha + \sin\alpha.\cos(\alpha + 2\delta)] \quad ...(2.1)$$

$$\sigma_X = \frac{q}{\pi}[\alpha - \sin\alpha.\cos(\alpha + 2\delta)] \quad ...(2.2)$$

$$\tau_{XZ} = \frac{q}{\pi}[\sin\alpha.\cos(\alpha + 2\delta)] \quad ...(2.3)$$

where,

q is the pressure intensity acting on a strip footing of width B, α and β angles defined the position of the point where the stresses σ_Z, σ_X and τ_{XZ} are obtained. (figure 2.3).

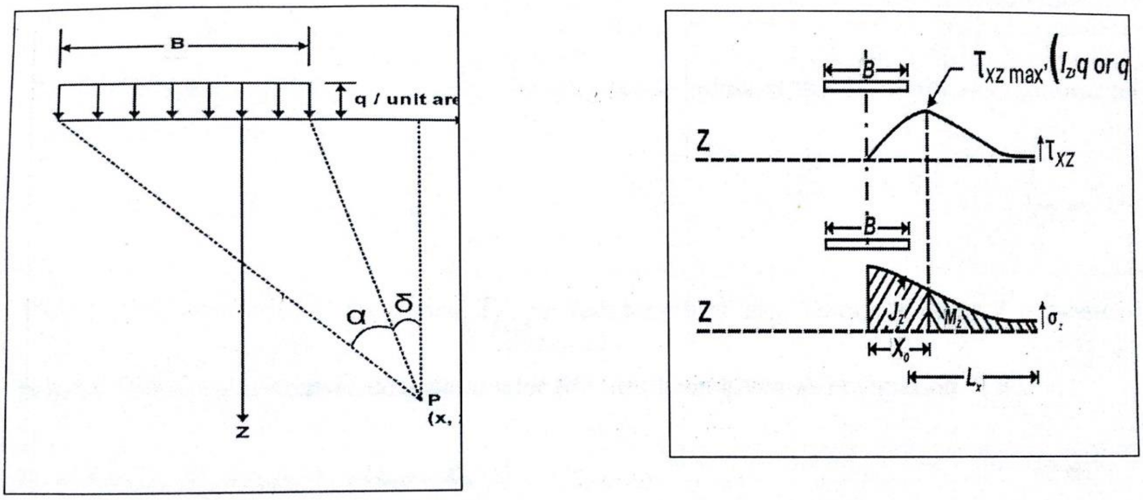


Figure 2.2 Non dimensional stress parameters at certain depth, using Boussinesq's equations.

The values of maximum shear stress, normal stress at the points on plane of failure and pull-out resistance have been expressed in terms of non- dimensional parameters as given in equations (2.4),(2.5),and(2.6) .The values of I_z , J_z and M_z have been computed and plotted as shown in fig.(2.3)

$$I_Z = \tau_{XZ\,\max} \qquad \ldots(2.4)$$

$$J_Z = \frac{\int_0^{Xo} \sigma_Z dx}{qB} \qquad \ldots(2.5)$$

$$M_Z = \frac{\int_{Xo}^{Lo} \sigma_Z dx}{qB} \qquad \ldots(2.6)$$

The tie force developed in the tie expressed in terms of non-dimensional parameters I_z and J_z as shown in equation(2.7)

$$T_D = [(J_Z B - I_Z \Delta H] (q-q_0) \qquad \ldots (2.7)$$

Above equation can be expressed in terms of pressure ratio (p_r) as under

$$T_D = [(J_Z B - I_Z \Delta H] q_0 (p_r -1) \qquad \ldots(2.8)$$

The pull-out frictional resistance of tie, T_f, per unit length of strip footing at depth Z has been expressed in terms of pressure ratio and non-dimensional parameter M_z as in equation..

$$T_f = 2f_e L_{dr} [M_Z B q_0 p_r ++ L_{dr} \Upsilon (L_0 - X_0) Z] \quad \ldots(2.\ 9)$$

And For the footing at depth D_f from the ground surface, is given by:

$$T_f = 2f_e L_{dr} [M_Z B q_0 p_r ++ L_{dr} \Upsilon (L_0 - X_0)(Z + D_f)] \quad \ldots (2.10)$$

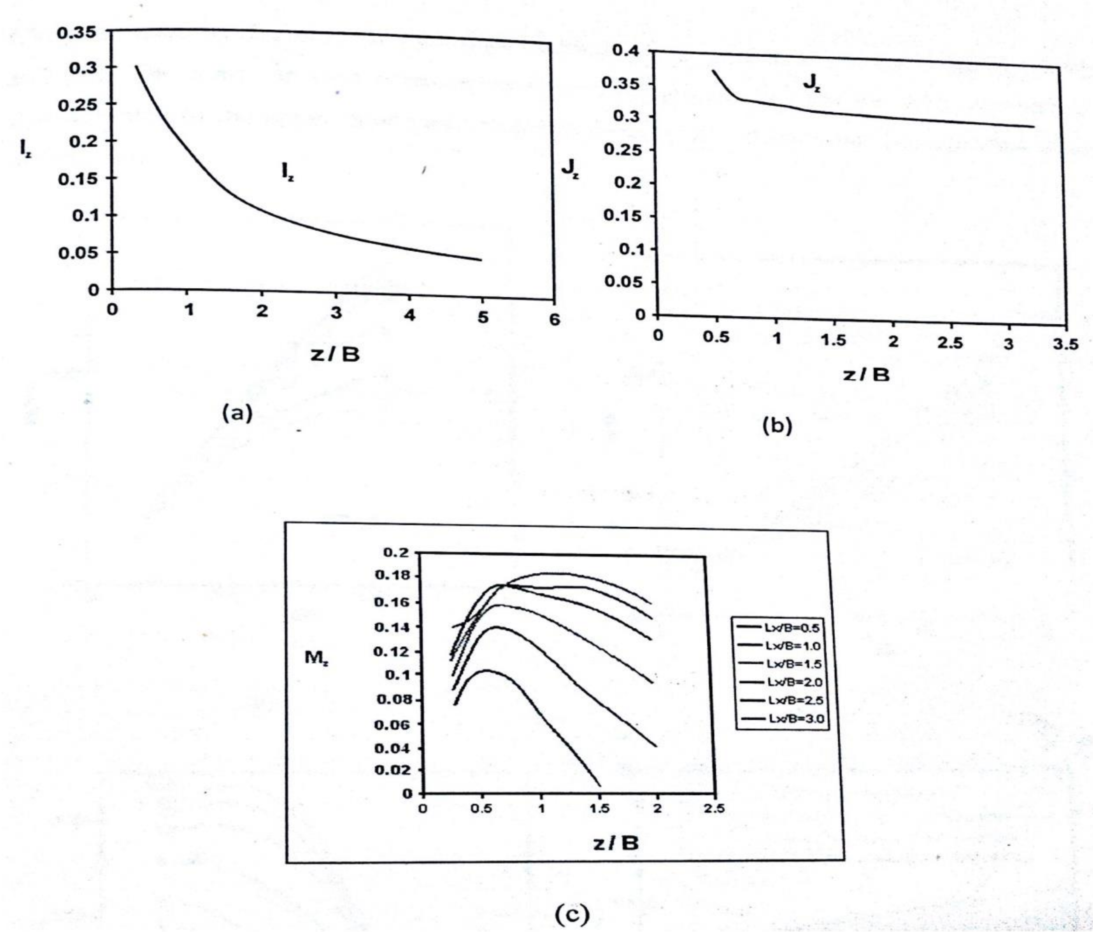
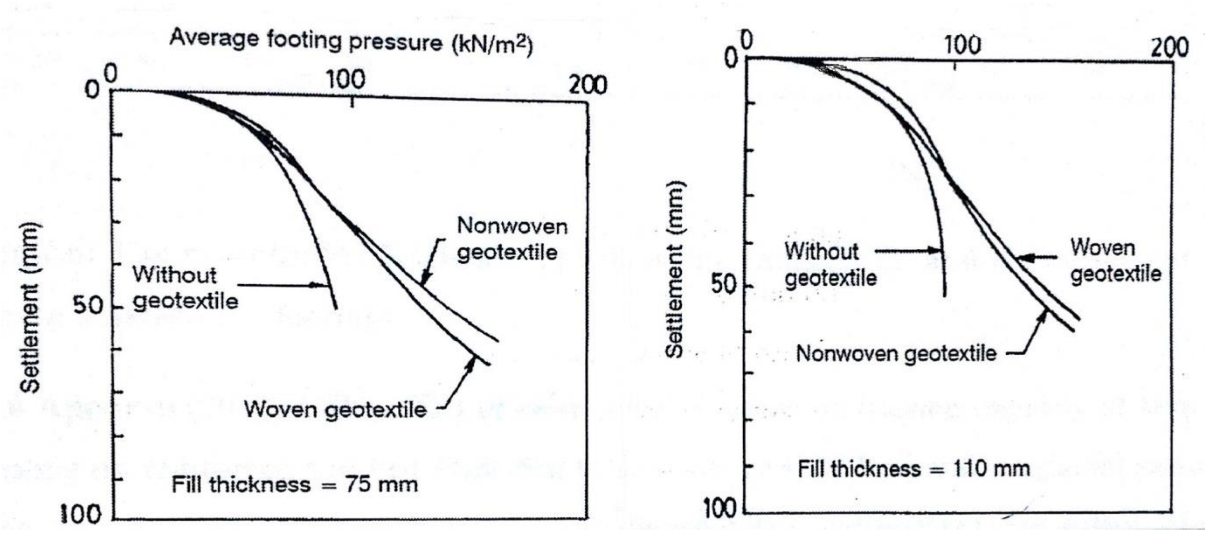


Figure 2.3.Non-dimensional stress parameters I_z,J_z and M_z (Binquet and Lee 1975)

2.3 Sahu et al.(1999)

He has worked on experimental observations for bearing capacity and settlement on pavements reinforced with geotextiles(Woven and Non Woven).he has found that pressure intensity increases in proportion to the settlement of the footing as shown in figure(2.4).Increase in load carrying capacity of the has been expressed as the ratio of Load carrying capacity of reinforced soil to that of unreinforced soil.



(a) **(b)**

Figure2.4:Settlement vs pressure relationships for different furnance ash layer thicknesses: (a) 75mm ; (b) 110mm

2.4 Apoorva (2012)

She established the effect of seismic acceleration on bearing capacity of strip footing resting on reinforced soil subjected to the tangential seismic load and uniform vertical load.she has proposed the plane of failure considering the effect of acceleration and plot the design charts involving non-dimensional parameters (I_z,J_z and M_z).

Then she has computed the pressure ratios for different combinations of reinforcement layers and analysis the bearing capacity of unit width strip footing on reinforced bedsubjected to different values of seismic accelerations.

Camparison of bearing capacity of strip footing subjected to seismic loading with bearing capacity of footing subjected to static loading has been made.It has been established that for a given value of seismic excitation bearing capacity of strip footing increases with increase in reinforcement layers.

CHAPTER -3
ANALYSIS OF STRIP FOOTING ON REINFORCED OF SOIL BED

3.1 Assumptions

The following assumptions are made for analysis:

1. With respect to the outer zones, the central soil zone is moving in vertical down direction. The junction of the downward moving and outward moving soil zones is considered as the plane of failure at which maximum shear stress is acting at every depth Z.
2. At the plane of failure, the ties are considered to undergo bends at two right angled bends such that tensile force T_D is acting vertically.
3. The coefficient of friction between soil and tie is assumed to vary with depth ,according to the following equation:
    a. $f_e = m \times f$
    … (1)
    b. where , m = mobilization factor given by
    c. $m= [(1-\frac{z}{B})0.7+.3]$ for z/B < 1.0 … (2a)
    d. $m= [(2-\frac{z}{B})0.3]$ for z/B > 1.0 … (2b)
4. Let X be the number of layers of reinforcement provided in the foundation soil bed. The tie force developed in the reinforcement is to be in the proportion of m_1: m_2: ..... : m_N such that, $m_1 + m_2 + ... + m_N=1$ and failure is considered for various combinations of tie-pull-out and tie rupture at different levels.
5. The evaluation of the forces in the analysis has been done by considering the same size of footing and same settlement for a footing on reinforced and unreinforced soil bed.
6. In order to calculate the stress distribution in the soil mass, theory of elasticity has been used.

7. For the calculation of forces on the reinforced as well as the unreinforced soil element, the principle of superposition is used .

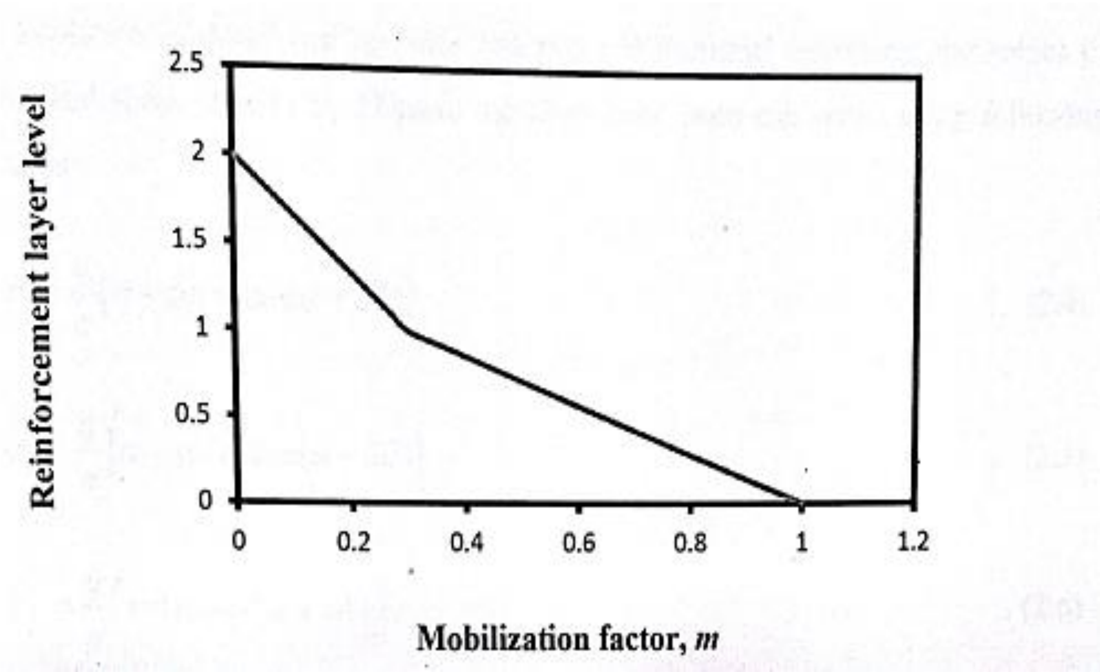


Fig.3.1 Friction mobilization at different reinforcement layer level.

3.2 Pressure ratio (p_r)

The aim of the analysis is to obtain the pressure that can be imposed on the footing resting on reinforced earth slab corresponding to a given settlement of the same footing resting on reinforced soil.

For convenience in expressing and comparing the data a term pressure ratio has been introduced and is defined as

$$P = \frac{q}{q_o}$$

Where

q= average contact pressure of footing on reinforced soil at settlement Δ.

q_0= average contact pressure of footing on unreinforced soil at same settlement Δ.

Therefore, the prerequisite of the analysis is to have the pressure settlement characteristics of the actual footing resting on unreinforced soil bed. This can be obtained by suitably interpreting the plate load test data to standard penetration test data.

3.3 Analysis of strip footing on reinforced soil

3.3.1 Eccentric Inclined Loading

An Eccentric inclined Load acts at an eccentricity of e, inclination of angle B with vertical axis as shown in figure 3.2

In above case, stress below the footing is on account of superposition of stresses due to

1. A uniform vertical pressure=q(1 ± av)(1-6*e/b)*cosβ ...3.1
2. A triangular vertical pressure=q(1 ± av)(12e/b)* cosβ ...3.2
3. Horizontal seismic stress=q*ah + q*sinβ ...3.3

In the seismic eccentric inclined Load, the load intensity right side and left side will be

q(1 ± av)(1+6*e/b)* cosβ and q(1 ± av)(1-6*e/b)* cosβ respectively.

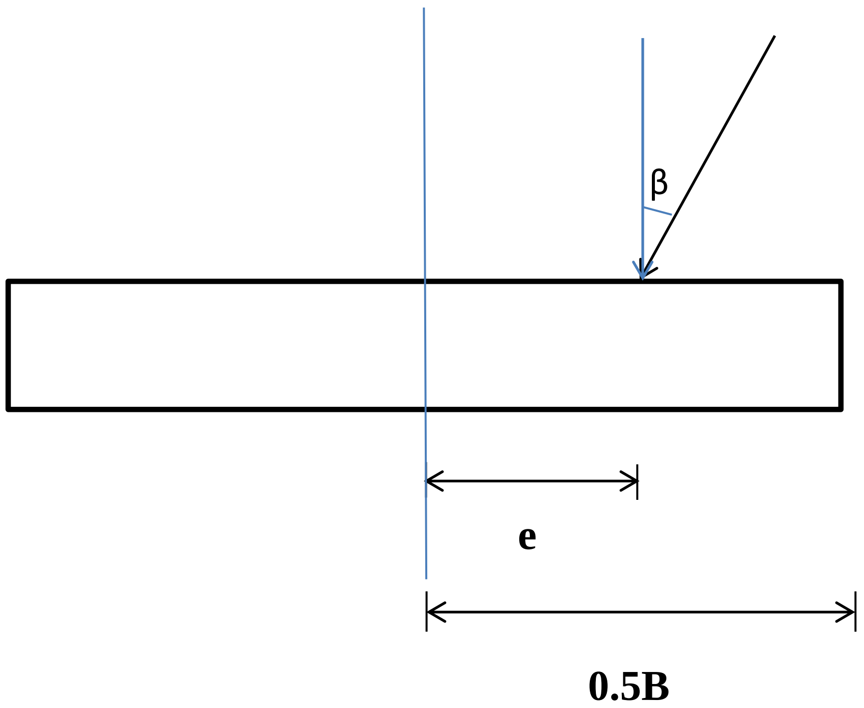


Fig. 3.2 Eccentric inclined load on strip footing

3.3.2 Normal Stress

The normal stress is due to superposition of normal stresses on account of above three loading conditions, and are given as:

$$\sigma_z = \sigma_{z1} + \sigma_{z2} + \sigma_{z3} \qquad \ldots 3.4$$

σ_{z1} is normal stress due to rectangular load

σ_{z2} is normal stress due to triangular load

σ_{z3} is normal stress due to tangential load

these are given as:

$$\sigma_{z1} = \frac{q(1 \pm a_v)(1 - 6 * \frac{e}{B})\cos\beta}{\pi}[\alpha + \sin\alpha.\cos(\alpha + 2\delta)\ ] \qquad \ldots 3.4a$$

$$\sigma_{z2} = \frac{q(1 \pm a_v)(12 * \frac{e}{b})\cos\beta}{2\pi}[\frac{2\alpha(x + 0.5B)}{B} - \sin(2\delta)] \qquad \ldots 3.4b$$

$$\sigma_{z3} = \frac{q(a_h + \sin\beta)}{\pi}[\sin\alpha.\sin(\alpha + 2\delta)] \qquad \ldots 3.4c$$

further α and δ can be associated with x and z as follow:

tanδ =x-0.5B /z

tan(δ+α)= x+0.5B /z

By putting these values in the equations in above three equations, we further reduced the normal stress equations.

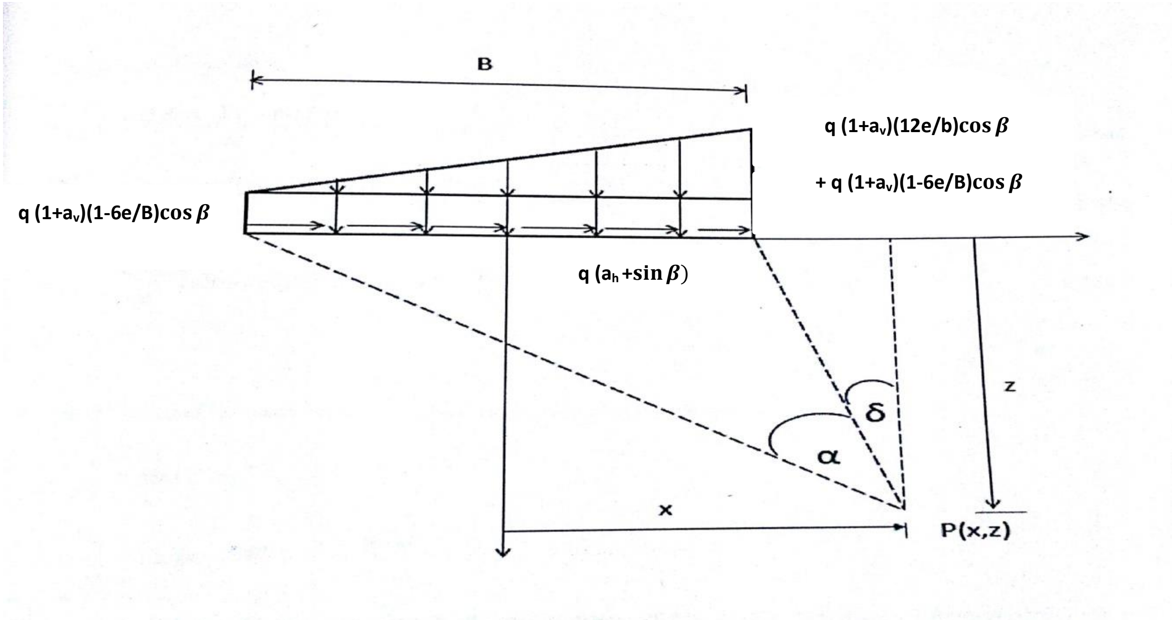


Fig. 3.3 Uniform vertical, uniform horizontal and linearly increasing triangular loads

3.3.3 Shear Stress

The shear stress is due to superposition of shear stresses on account of above three loading conditions, and are given as:

$$\tau_z = \tau_{xz1} + \tau_{xz2} + \tau_{xz3} \qquad \ldots 3.5$$

τ_{xz1} is shear stress due to rectangular load.

τ_{xz1} is shear stress due to triangular load.

$\tau_{xz3}$is shear stress due to tangential load.

these are given as:

$$\tau_{xz1} = \frac{q(1 \pm a_v)(1 - 6 * \frac{e}{b}) \cos\beta}{\pi} [\sin\alpha . \sin(\alpha + 2\delta)] \qquad \ldots 3.5a$$

$$\tau_{xz2} = \frac{q(1 \pm a_v)(12 * \frac{e}{b}) * \cos\beta}{2\pi} [1 + \cos(2\delta) - (\frac{2z\alpha}{B})] \qquad \ldots 3.5b$$

$$\tau_{xz3} = \frac{q}{\pi} (\alpha_h + \sin\beta)[\alpha - \sin\alpha . \cos(\alpha + 2\delta)] \qquad \ldots 3.5c$$

further α and δ can be associated with x and z as follow:

tanδ =x-0.5B /z

tan(δ+α)= x+0.5B /z

by putting these values in the above three equations ,we further reduced the shear stresses equations.

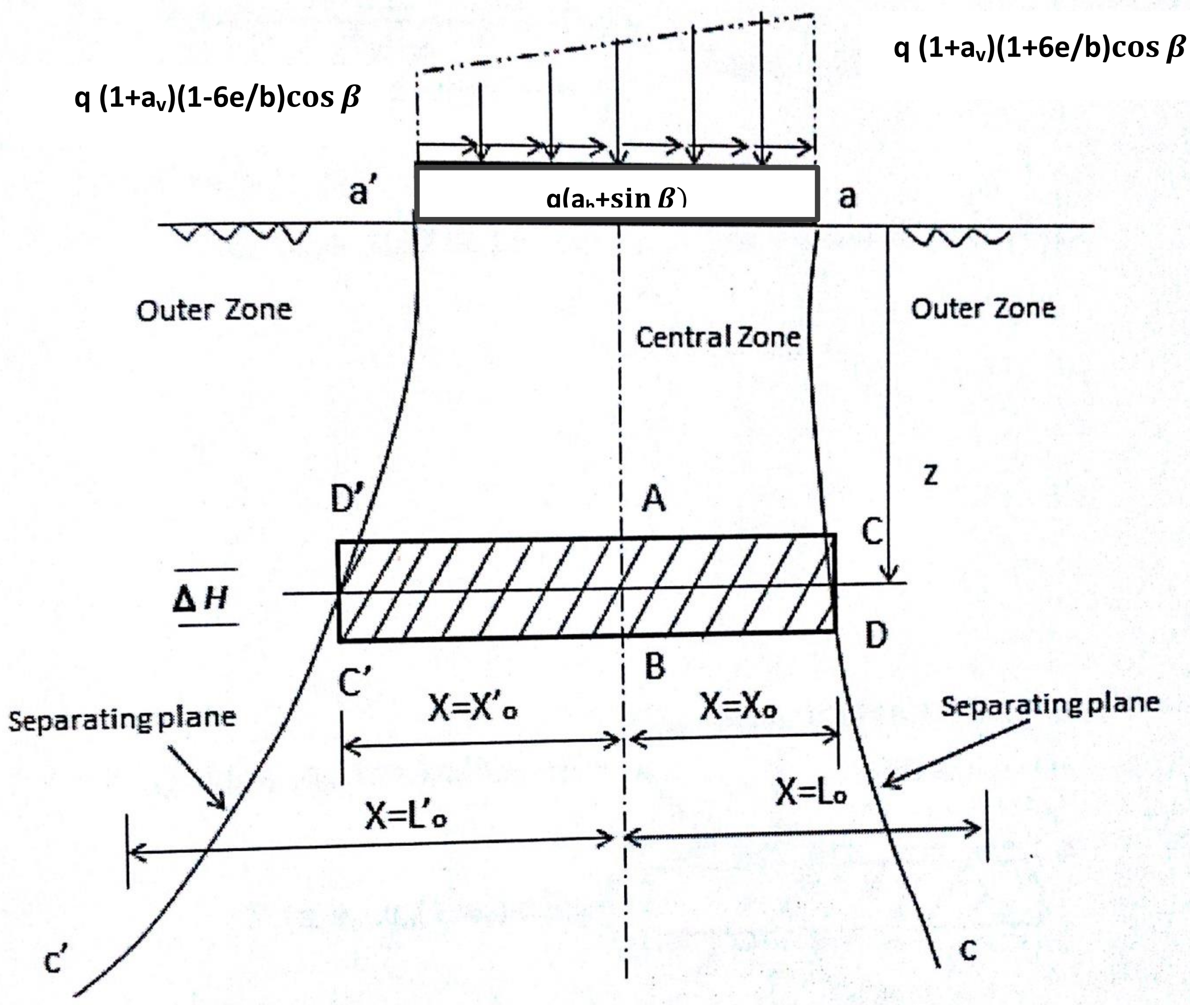


FIG.3.4 Failure plane on both side of strip footing(unsymetrical)

3.4 Computation of developed tie force (T_d)

To evaluate the forces developed in the ties due to applied load on the footing, it was assumed that the plane separating the downward and lateral moving zones is the locus of points of maximum shear stress τ_{XZmax} at every depth Z. In Fig.3.4.,ac and a'c' are assumed separating planes.

Consider an element ABCD and ABC'D, at any depth Z (Fig.3.4) which is the volume of soil lying between two adjacent layers of reinforcement. The forces acting on the element are shown in this figure for unreinforced and reinforced foundation soil. F_{VAD} (q a_h sinβ,q(1+a_v)cosβ,e/B,Z),F_{VBC}(q(a_h+sinβ,q(1+a_v)cosβ,e/B,Z),F'$_{VAD'}$(q(a_h+sinβ),q(1+a_v)cos β,e/B,Z),F'$_{VBC'}$(q(a_h+sinβ),q(1+a_v)cosβ,e/B,Z) are the normal forces and S(q(a_h+ sinβ),q(1+a_v)cosβ,e/B, Z) and S (q (a_h+ sinβ),q(1+a_v)cosβ,e/B, Z) are the vertical shear force acting on the boundaries of the element of unreinforced soil. These forces are due to normal and shear stresses at depth Z, caused by the applied bearing pressure q on the footing. A similar set of forces also exist for the reinforced soil foundation which will be caused by footing bearing pressure q. in addition, there will be force developed in the tie, $T_{D.}$

Equilibrium of the forces in vertical direction in the unreinforced soil (element D'C'CD) may be expressed as-

F_{VAD}(q_0(a_h+sinβ),q_0(1+a_v)cosβ,e/B,Z)-F_{VBC}(q_0(a_h+sinβ),q_0(1+a_v)cosβ,e/B,Z)+ F'$_{VAD'}$(q_0(a_h+sinβ),q_0(1+a_v)cosβ,e/B,Z)-F'$_{VBC'}$(q_0(a_h+sinβ),q_0(1+a_v)cosβ.e/B,Z)-S(q_0(a_h +sinβ),q_0(1+a_v)cosβ,e/B,Z)-S'(q_0(ah+sinβ),q_0(1+a_v)cosβ,e/B,Z)+dW(1+a_v)=0 …3.6

For single layer of reinforced in the foundation soil at depth Z, the equilibrium of element D'C'CD in vertical direction may be expressed as-

F_{VAD} (q(a_h+sinβ),q(1+a_v)cosβ, e/B, Z) - F_{VBC}(q(a_h+ sinβ), q(1+a_v)cosβ, e/B, Z) + F'$_{VAD'}$ (q(a_h+sinβ),q(1+a_v)cosβ,e/B,Z)-F'$_{VBC'}$(q(a_h+sinβ),q(1+a_v)cosβ.e/B,Z-S(q(a_h+sinβ),q(1+a_v)cosβ,e/B,Z)-S'(q(a_h+sinβ),q(1+a_v)cosβ,e/B,Z)+dW(1+a_v) +2T_D =0 …3.7

It has been assumed in the analysis that forces are evaluated for the same size of footing B and the same settlement, for a footing on reinforced and unreinforced soil, so F_{VBC} shall be same for reinforced and unreinforced soil. The additional load (q-q_a) shall be taken by the reinforced above the level CC'.

F_{VBC}(q_0(a_h+sinβ),q_0(1+a_v)cosβ,e/B,Z)-F'$_{VBC'}$(q_0(a_h+sinβ),q_0(1+a_v)cosβ,e/B,Z)= F_{VBC}(q(a_h+sinβ),q(1+a_v)cosβ,e/B,Z)-F'$_{VBC'}$(q(a_h+sinβ),q(1+a_v)cosβ,e/B,Z) …3.8

Combining above equations, we get

$F_{VAD}(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)+F'_{VAD'}(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)-F_{VAD}(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)F'_{VAD'}(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)=S(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)+S'(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)-S(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)-S'(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)+2T_D$ …3.9

For the case of reinforced sand, the normal and shear forces are given as:

$F_{VAD}(q(a_h+\sin\beta),q(1+a_v)\cos\beta, e/B, Z)=\int_0^{Xo} \sigma_z (q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z,x)dx$ …3.10a

$F'_{VAD'}(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B, Z)= \int_0^{Xo} \sigma_z (q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z,x)dx$ …3.10b

$S(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)= \tau_{xz} (q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z,x_0)\Delta H$ …3.10c

$S'(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)=\tau'_{xz} (q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z,x_0)\Delta H$ …3.10d

For the case of unreinforced sand, the normal and shear forces are given as:

$$F_{VAD}(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,\frac{e}{B},Z)=\int_0^{Xo}\sigma_Z(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,\frac{e}{B},Z,x)dx$$ …3.11a

$$F'_{VAD'}(q_o(a_h+\sin\beta),q_o(1+a_v)\cos\beta,\frac{e}{B},Z)=\int_0^{Xo}\sigma_z(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,\frac{e}{B},Z,x)dx$$ …3.11b

$S(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)=\tau_{xz}(q_0(a_h+\sin\beta), q_0(1+a_v)\cos\beta, e/B, Z,x_0)\Delta H$ ...3.11c

$S'(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)=\tau_{xz}'(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B, Z,x_0)\Delta H$ ...3.11d

The equations (3.10a),(3.10b),(3.10c) and(3.10d) for the reinforced soil case can be written in dimensionless forms as under:

$F_{VAD}(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)=J_Z qB$ …3.12a

Where $$J_Z=\frac{\int_0^{Xo}\sigma_z(q(a_h+\sin\beta),q(1+a_v)\cos\beta,\frac{e}{B},Z,x)dx}{qB}$$ …3.12b

$F'_{VAD'}(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)=J_z'qB$ …3.13a

Where $J'_Z = \dfrac{\int_0^{Xo} \sigma'_z (q(a_h + \sin\beta), q(1+a_v)\cos\beta, \frac{e}{B}, Z, x)dx}{qB}$ …3.13b

$S(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)= I_Z q\Delta H$ …3.14a

Where $I_z = \dfrac{\tau_{xz} (q(a_h + \sin\beta), q(1+a_v)\cos\beta, \frac{e}{B}, Z, x_0)dx}{qB}$ …3.14b

$S'(q(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z) = I'_Z q\Delta H$ …3.15a

Where $I'_Z = \dfrac{\tau'_{xz} (q(a_h + \sin\beta), q(1+a_v)\cos\beta, \frac{e}{B}, Z, x_0)dx}{qB}$ …3.15b

Similarly The equations (3.11a),(3.11b),(3.11c) and(3.11d) for the reinforced soil case can be written in dimensionless forms as under:

$F_{VAD}(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B,Z)= J_Z q_0 B$ …3.16a

Where, $J_Z = \dfrac{\int_0^{Xo} \sigma_z (q_0(a_h + \sin\beta), q_0(1+a_v)\cos\beta, \frac{e}{B}, Z, x)dx}{qB}$ …3.16b

$F'_{VAD}(q_0(a_h+\sin\beta),q(1+a_v)\cos\beta,e/B,Z)=J_Z q_0 B$ …3.17a

Where, $J'_Z = \dfrac{\int_0^{Xo} \sigma'_z (q_0(a_h + \sin\beta), q_0(1+a_v)\cos\beta, \frac{e}{B}, Z, x)dx}{qB}$ …3.17b

$S(q_0(a_h+\sin\beta),q_0(1+a_v)\cos\beta,e/B, Z) = I_Z q_0 \Delta H$ …3.18a

Where, $I_Z = \dfrac{\tau_{xz} (q_0(a_h + \sin\beta), q_0(1+a_v)\cos\beta, \frac{e}{B}, Z, x_0)dx}{qB}$

$S' (q_0(a_h +\sin\beta), q_0(1+a_v)\cos\beta ,e/B, Z) = = I'_Z q_0 \Delta H$ …3.19a

Where, $I'_Z = \dfrac{\tau'_{xz} (q_0(a_h + \sin\beta), q_0(1+a_v)\cos\beta, \frac{e}{B}, Z, x_0)dx}{qB}$ …3.19b

By substituting the dimensionless parameters I_Z , J_Z , the equation (3.9) can be written as:

$J_Z qB + J'_Z qB - J_Z q_0 B - J'_Z q_0 B = = I_Z q \Delta H + I'_Z q \Delta H - I_Z q_0 \Delta H - I'_Z q_0 \Delta H + 2T_D$ …3.20

Or

$(J_Z + J'_Z)(q-q_0)B = (I_Z + I'_Z)(q-q_0)\,\Delta H + 2T_D$

Or

$2T_D = (J_Z + J'_Z)(q-q_0)B = (I_Z + I'_Z)(q-q_0)\,\Delta H$

Or

$2T_D=[(J_Z+J'_Z)B-(I_Z + I'_Z)\Delta H]\,(q-q_0)$ …3.21

In terms of pressure ratio $q=q_0p_r$, so above the expression can be expressed as:

$2T_D=[(J_Z+J'_Z)B-(I_Z + I'_Z)\Delta H]q_0(p_r-1)$ …3.22

Or

$2T_D =[J_{Zm}B-I_{Zm}\,\Delta H]q_0(p_r-1)$ …3.23

Where, $J_{zm} = \frac{(J_z + J'_z)}{2}$ and $I_{zm} = \frac{(I_z + I'_z)}{2}$ …(3.24a,b)

3.5 Determination of pull-out resistance

The resistance of the tie against pull out is because of normal force acting on it across its length beyond the failure plane in soil region.
The normal force comprises of two components:

I. Applied bearing pressure
II. Overburden pressure

The forces on account of applied force is given as:

$$F_{V1}(q(a_h + \sin\beta), q(1+a_v), \frac{e}{B}, Z) = L_{dr} \int_{Xo}^{Lo} \sigma_z (q(a_h + \sin\beta), q(1+a_v)\cos\beta, \frac{e}{B}, Z, x)dx$$

…3.24a

$$F'_{V1}(q(a_h + \sin\beta), q(1+a_v), \frac{e}{B}, Z) = L_{dr} \int_{X'o}^{L'o} \sigma_z (q(a_h + \sin\beta), q(1+a_v), \frac{e}{B}, Z, x)dx$$

…3.24b

In which

L_0=0.5B+L_x=L_r/2 …3.25

L_x =stretch length of reinforcement the beyond footing edge.

L_r =length of reinforcement

L_{dr} =linear density of the reinforcement (=1.for geo-grid, mat or sheets)

The equations () and() can be written in the form of the dimensionless parameters as follows:

$$F_{V1}(q(a_h + \sin\beta), q(1+a_v), e/B, Z) = L_{dr} BqM_Z$$ …3.26a

$$F'_{V1}(q(a_h + \sin\beta), q(1+a_v), e/B, Z) = L_{dr} BqM_Z$$ …3.26b

Where M_Z and M'_Z are dimensionless parameters given by

$$M_Z = \frac{\int_{Xo}^{Lo} \sigma_z (q(a_h + \sin\beta), q(1+a_v)\cos\beta, \frac{e}{B}, Z, x)dx}{qB}$$ …3.27a

$$M'_Z = \frac{\int_{X'o}^{L'o} \sigma_z (q(a_h + \sin\beta), q(1+a_v)\cos\beta, \frac{e}{B}, Z, x) dx}{qB} \quad \ldots 3.27b$$

The value of M_z can be evaluated by using above equation for any length of reinforcement provided.

The second component of normal force is given as

$F_{V2}=L_{dr}\Upsilon(L_0-X_0)Z$ …3.28a

$F'_{V2}=L_{dr}\Upsilon(L'_0-X'_0)Z$ …3.28b

In which, ϒ=unit weight of the soil.

Hence, total normal force is given by :

$F_{VT}=F_{V1}+ F'_{V1}+ F_{V2}+ F'_{V2}$ …3.29

Using the above equations, we get

$F_{VT=}L_{dr}BM_Zq+L_{dr}BM'_Zq + L_{dr}\Upsilon(L_0-X_0)Z + L_{dr}\Upsilon(L'_0-X'_0)Z$ …3.30

The tie-pull-out frictional resistance, T_f, per unit length of strip footing at depth Z in terms of pressure ratio may be written by multiplying right part of equation(3.30) by $2f_e$.

Where f_e =soil tie coefficient of friction is given by

$f_e=m*f$ …3.31a

$f=\tan\varphi_f$ …3.31b

φ_f =soil-reinforcement friction angle

where , m = mobilization factor given by

a. $m= [(1-\frac{z}{B})0.7+.3]$ for z/B < 1.0 …3.31c
b. $m= [(2-\frac{z}{B})0.3]$ for z/B > 1.0 …3.31d

$2T_f = 2f_e\, L_{dr}\, [M_ZBq_0p_r + L_{dr}\Upsilon(L_0-X_0)Z\,] +2f_e\, L_{dr}\, [M'_ZBq_0p_r + L_{dr}\Upsilon(L'_0-X'_0)Z\,]$ …3.32

For the footing at depth D_f

$2T_f = 2f_e\, L_{dr}\, [M_ZBq_0p_r + L_{dr}\Upsilon(L_0-X_0)(Z+D_f)\,] +2f_e\, L_{dr}\, [M'_ZBq_0p_r + L_{dr}\Upsilon(L'_0-X'_0)(Z+D_f)\,]$

…3.33

3.6 Determination of Pressure Ratio

a) In no case pull-out force should exceed the resistance offered by reinforcing tie, in terms of equations it is expressed as:

$mT_{Di}<T_{fi}$ …3.34

where i=1,2,3….N

b) In no case the developed tie force should exceed the rupture force

$mT_{Di}<T_{Rf}$ …3.35

and in critical case, we solve the equation below to give pressure ratio as following:

$mT_{Di}=T_{fi}$ and $mT_{Di}=T_{Rf}$ …3.36(a,b)

for a given combination of layers of reinforcements the minimum of the pressure ratio found is taken as critical pressure ratio and this used for calculating bearing capacity of reinforcing soil.

3.7 Determination of ultimate bearing capacity of footing on reinforced soil bed

By using the equation for bearing capacity of reinforcing soil bed, bearing capacity of
the strip footing on reinforcing soil can be found out as shown below:

$q_{ur} = q_r + \Upsilon D_r N_{qr}$ …3.37

where, $q_r = q_u p_{ru}$ …3.38

D_r=depth of reinforcement layer
N_{qr}= bearing capacity factor for soil under consideration
Υ =Unit weight of soil

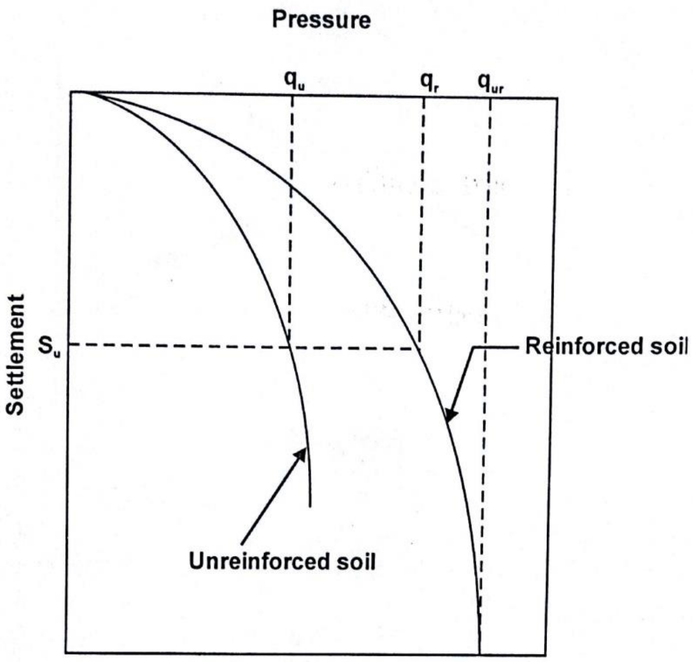


Fig.3.5 Pressure Settlement Curve for different types of soil bed

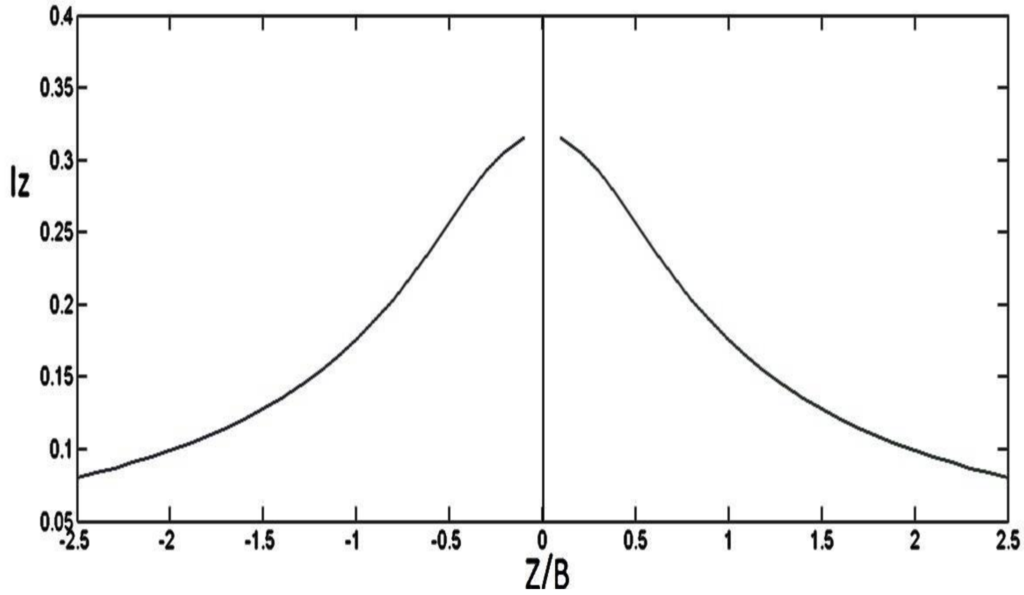
TABLE 3.1: Seismic Reduction factors for N_q and N_Υ (Hasan Rangwala2014)

a_h	SRFγ			SRFq		
	e/B=0.0	e/B=0.1	e/B=0.2	e/B=0.0	e/B=0.1	e/B=0.2
0.000	1.00	0.60	0.33	1.00	0.71	0.54
0.025	0.90	0.48	0.26	0.78	0.66	0.50
0.050	0.79	0.39	0.20	0.74	0.61	0.46
0.075	0.62	0.32	0.16	0.69	0.56	0.42
0.100	0.42	0.26	-	0.65	0.51	0.37

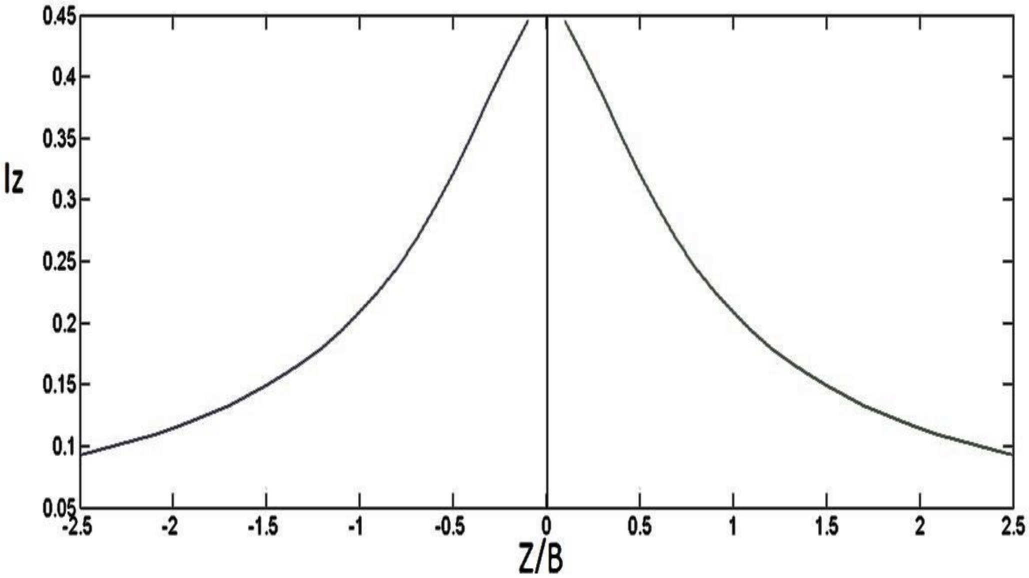
3.8 Dimensionless Parameters (I_z, J_z, M_Z)

The values of the dimensionless parameters Iz, J_z and M_Z have been calculated by using the proper MATLAB CODES, at different cases seismic acceleration and eccentricity. Plots for the same have been given in figures :

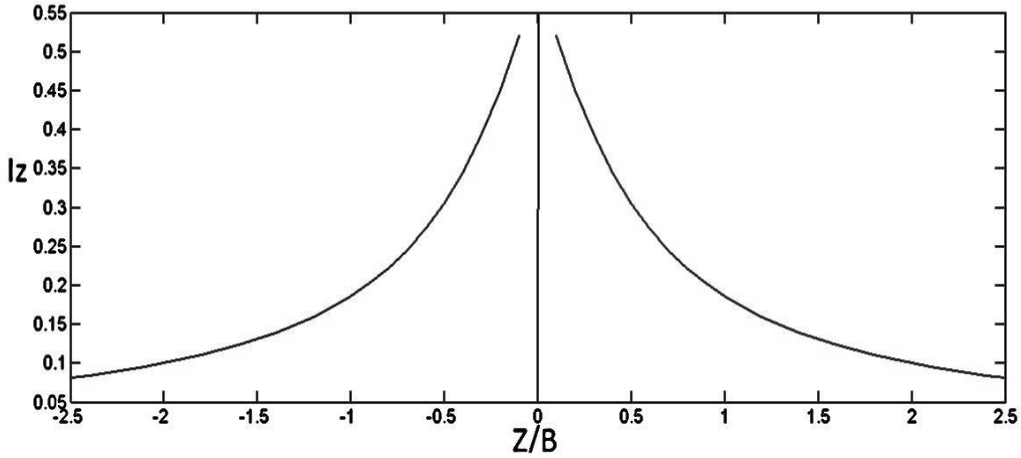
Plots of I_Z for a_h =0.0, different values of eccentricities and inclination angles



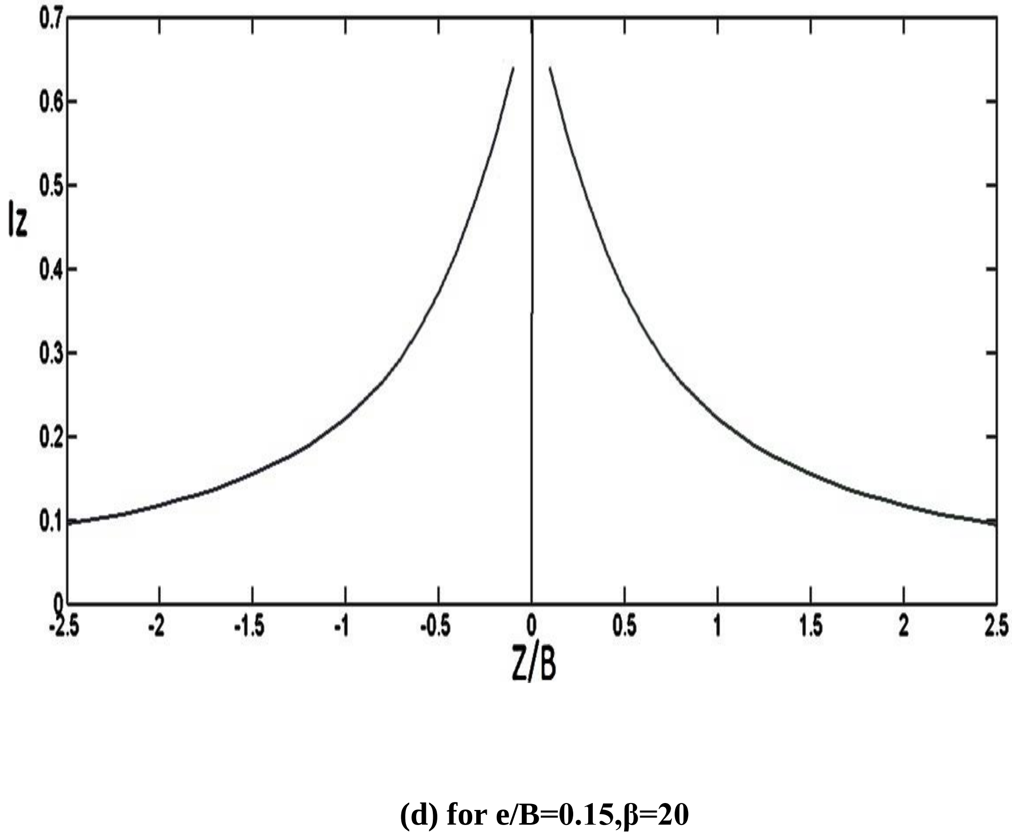
(a) for e/B=0,β=0



(b) for e/B=0,β=20



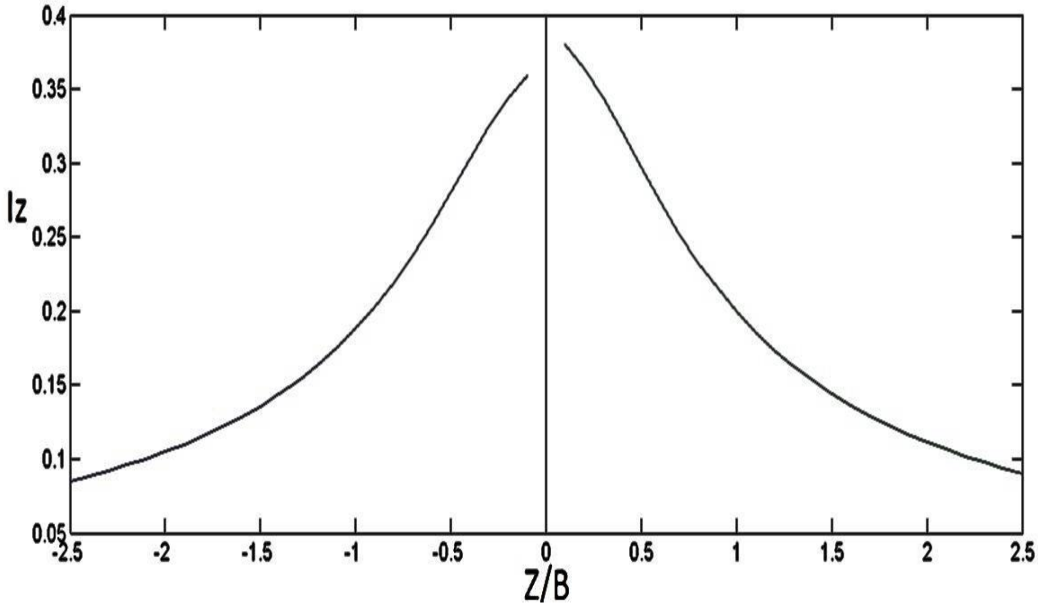
(c) for e/B=0.15,β=0



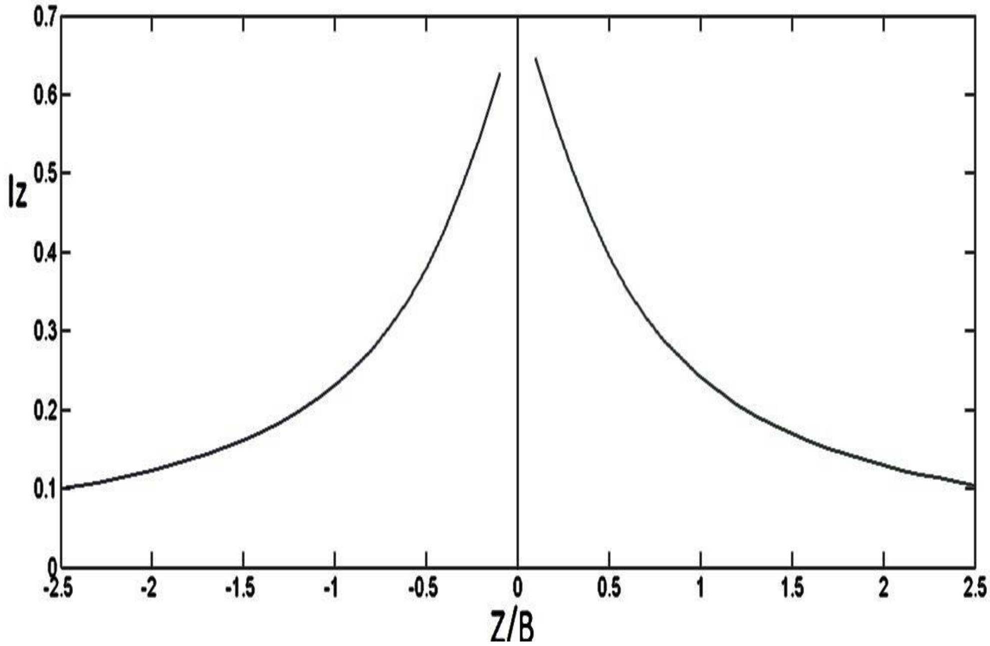
(d) for e/B=0.15,β=20

Fig.3.6: I_Z for a_h =0.0 for different values of eccentricities and inclination angles

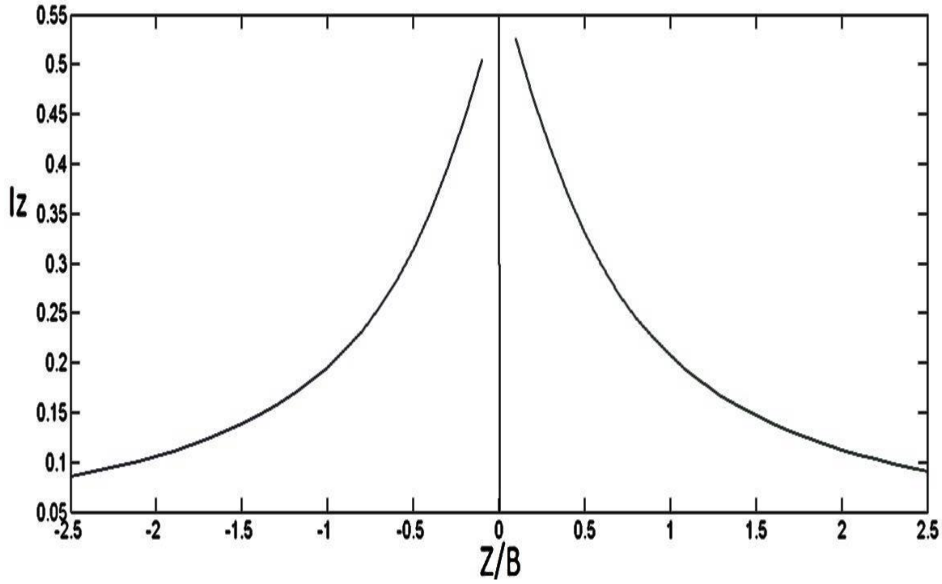
Plots for a_h=0.10



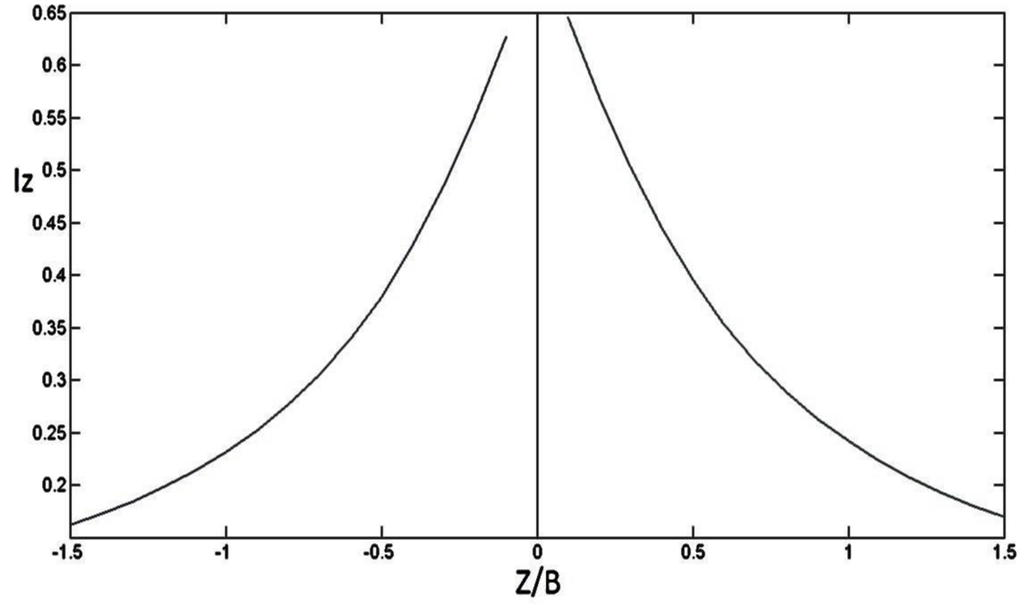
(a) for e/B=0,β=0



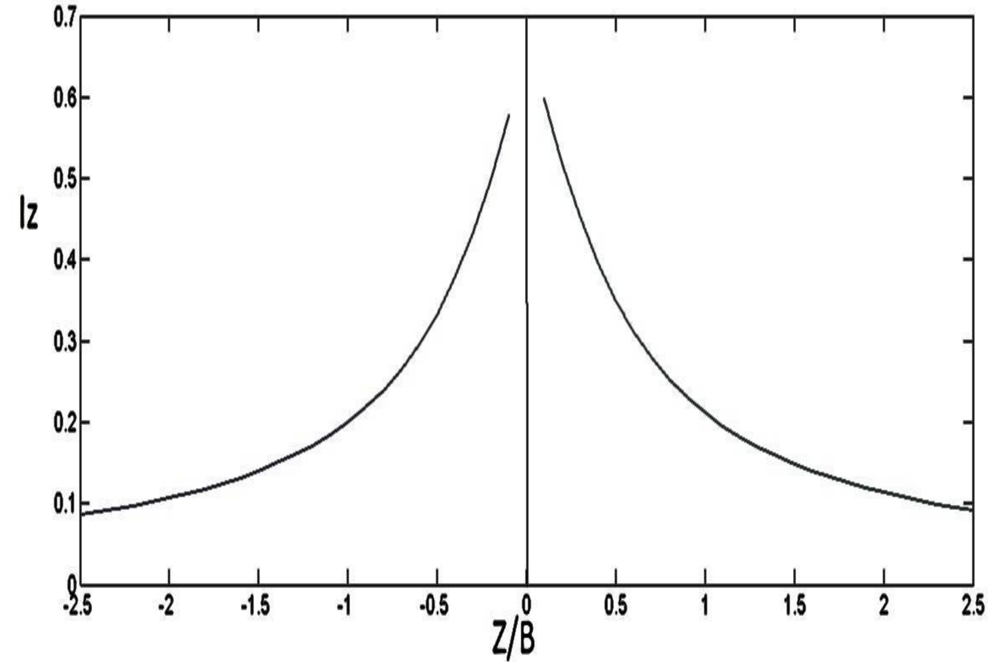
(b) for e/B=0,β=20



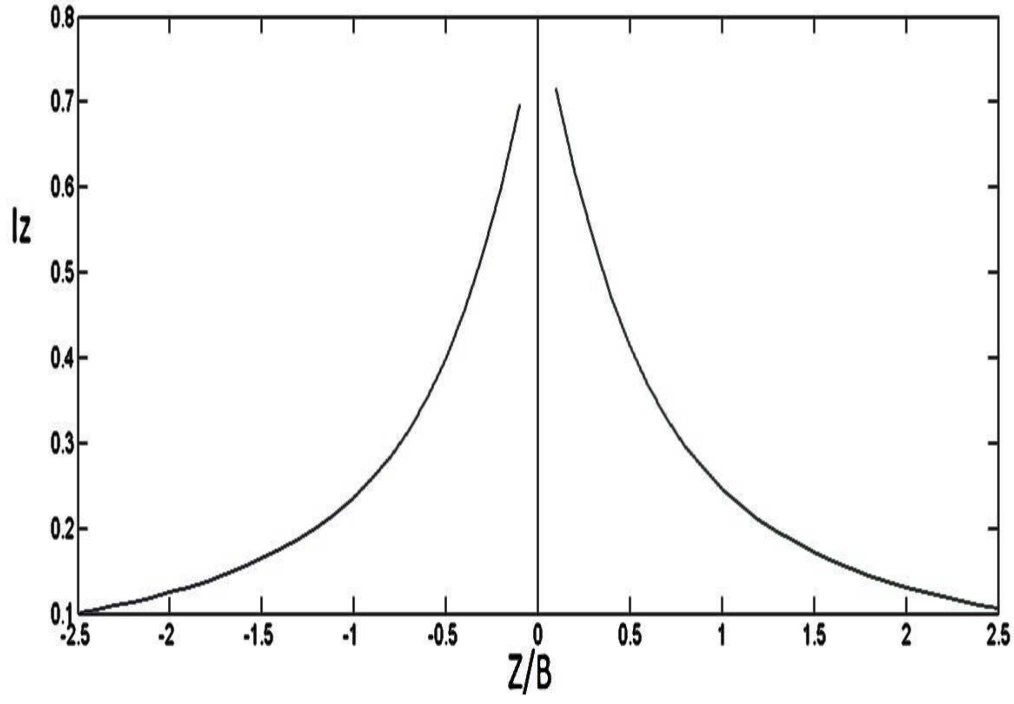
(c) for e/B=0.10,β=0



(d) for e/B=0.10,β=20



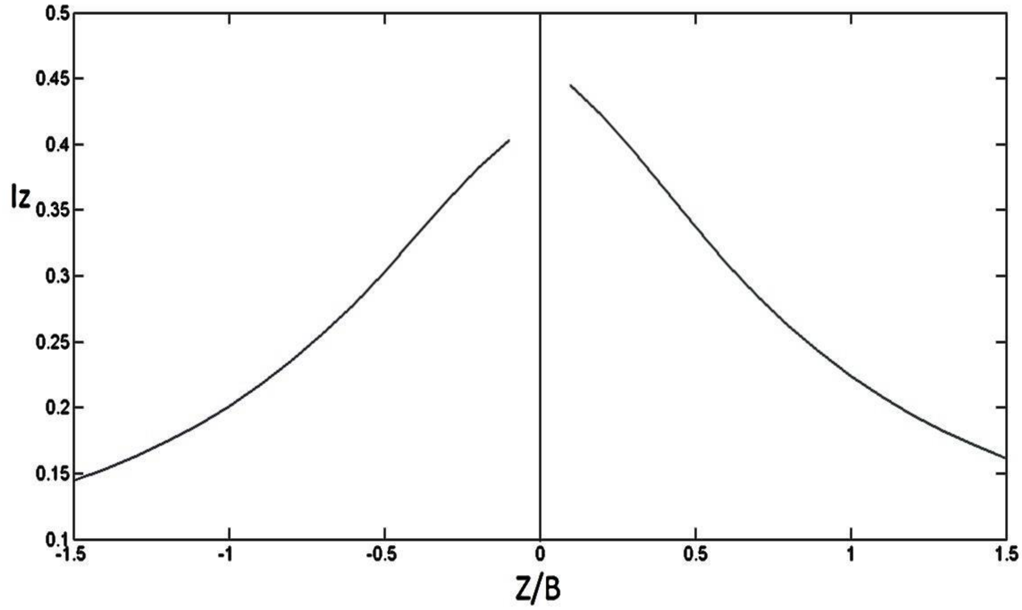
(e) for e/B=0.15,β=0



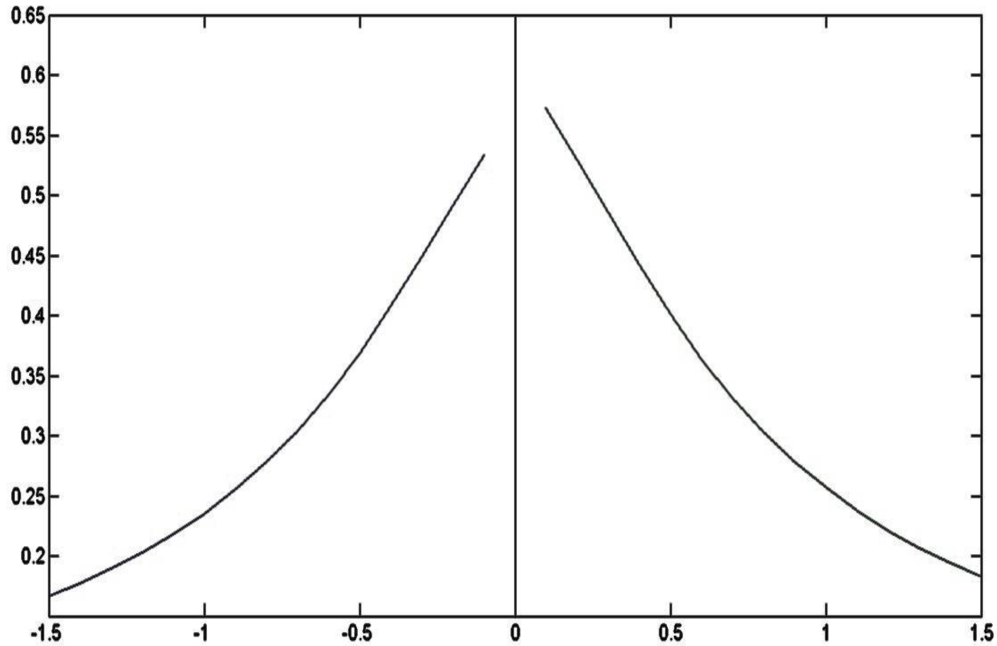
(f) for e/B=0.15,β=20

Fig.3.7: I_Z for a_h =0.10 for different values of eccentricities and inclination angles

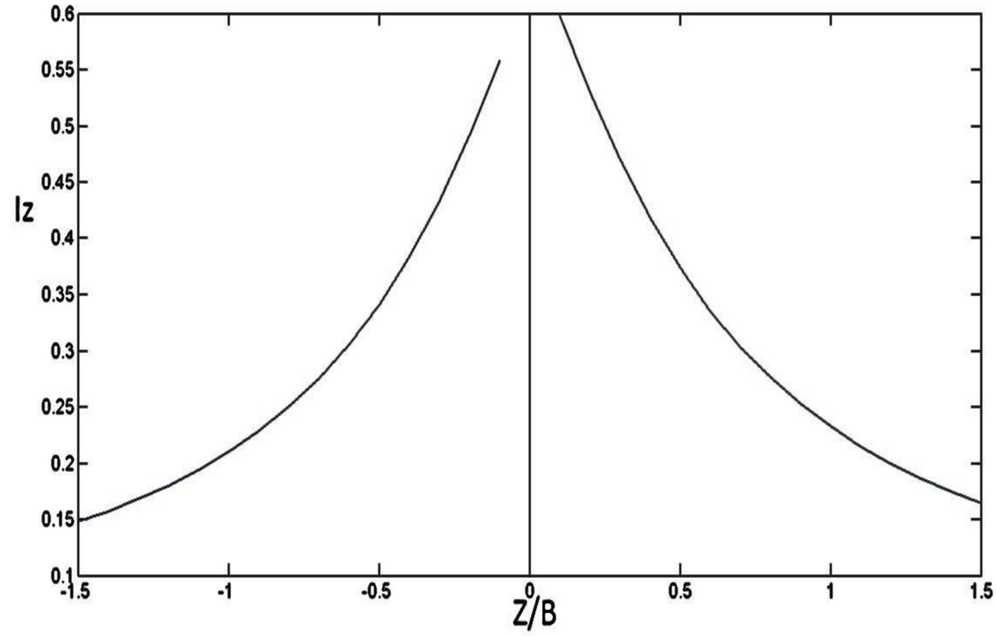
Plots for a_h=0.20



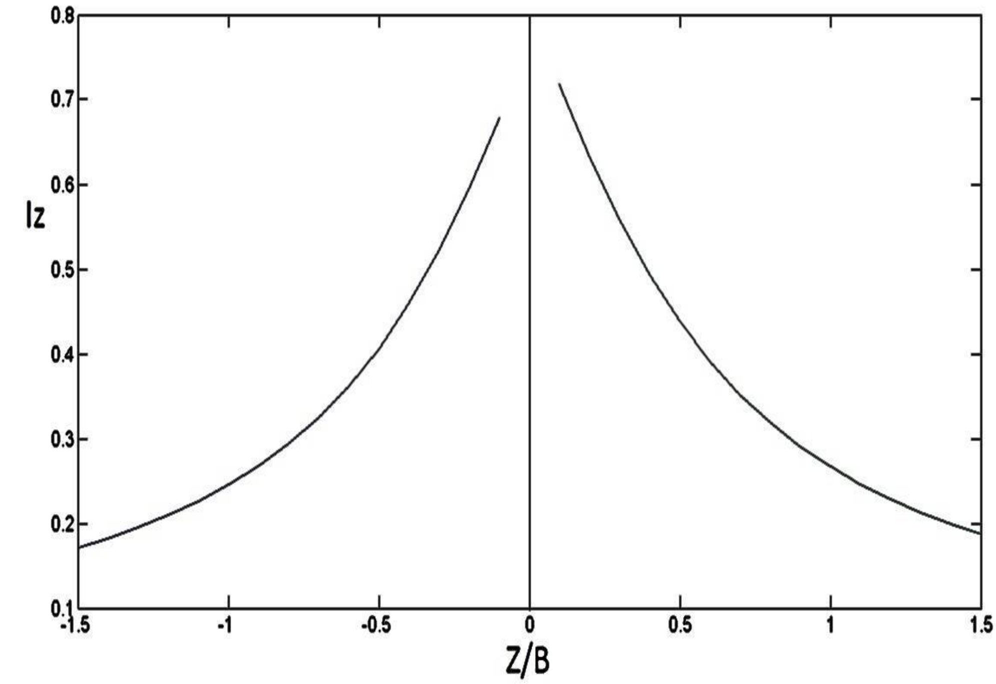
(a) for e/B=0,β=0



(b) for e/B=0,β=20



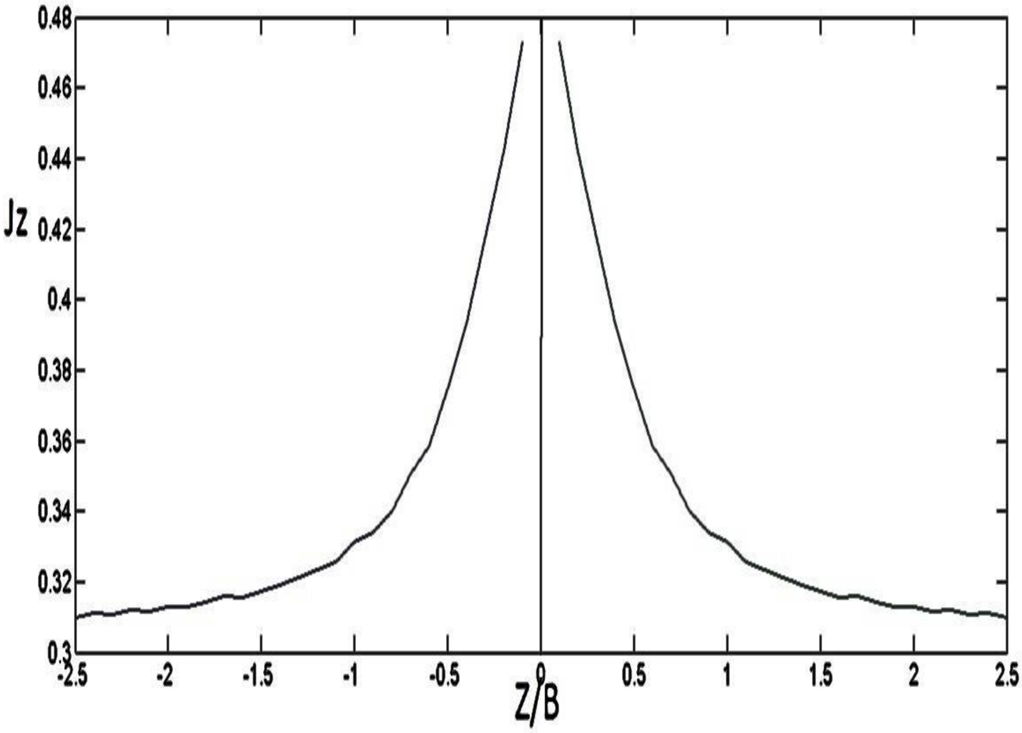
(c) for e/B=0.1,β=0



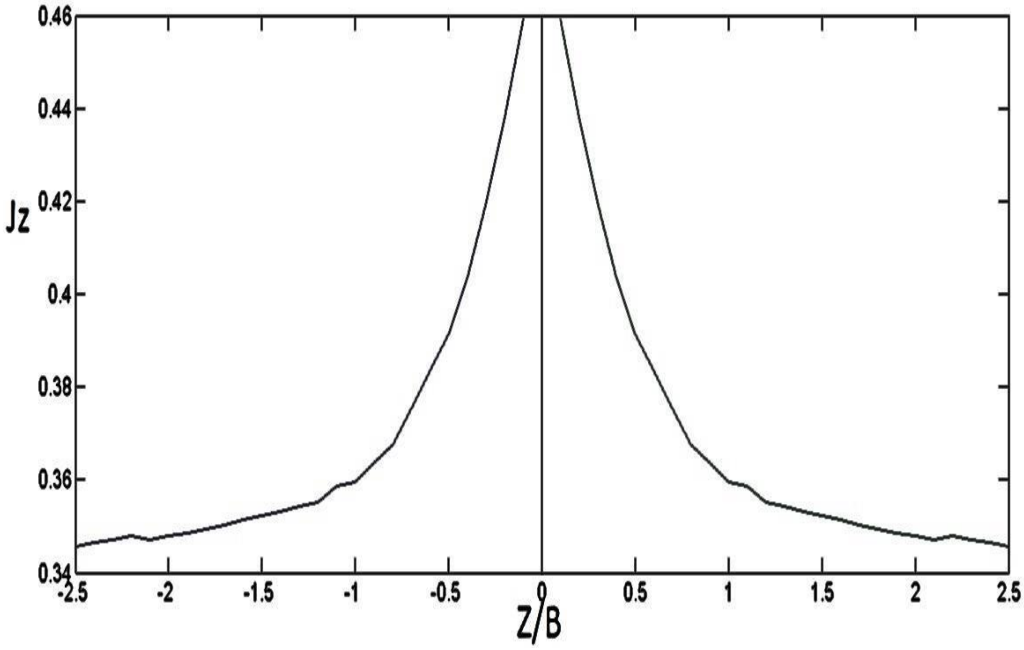
(d) for e/B=0.1,β=20

Fig.3.8: I_Z for a_h =0.20 for different values of eccentricities and inclination angles

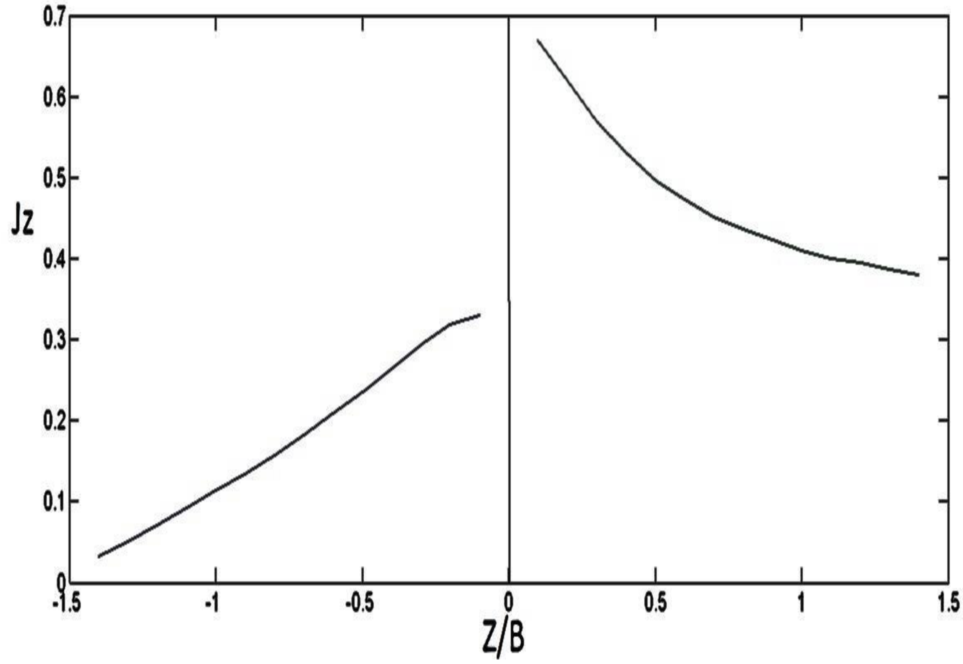
Plots of J_Z for a_h =0.0, different values of eccentricities and inclination angles



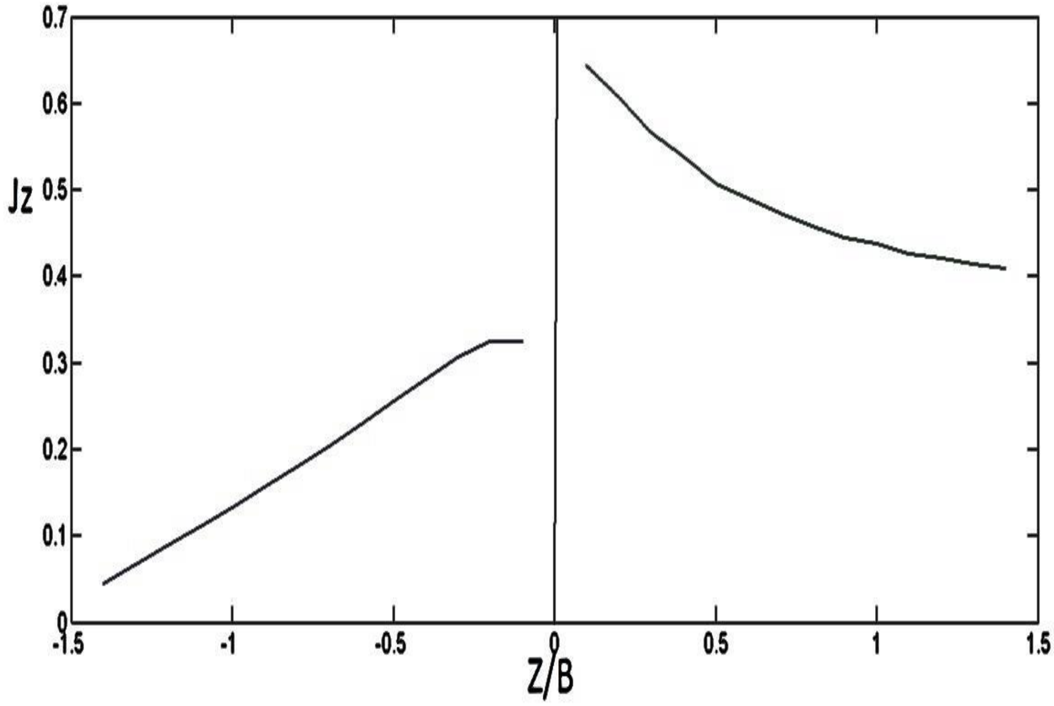
(a) for e/B=0,β=0



(b) for e/B=0,β=20



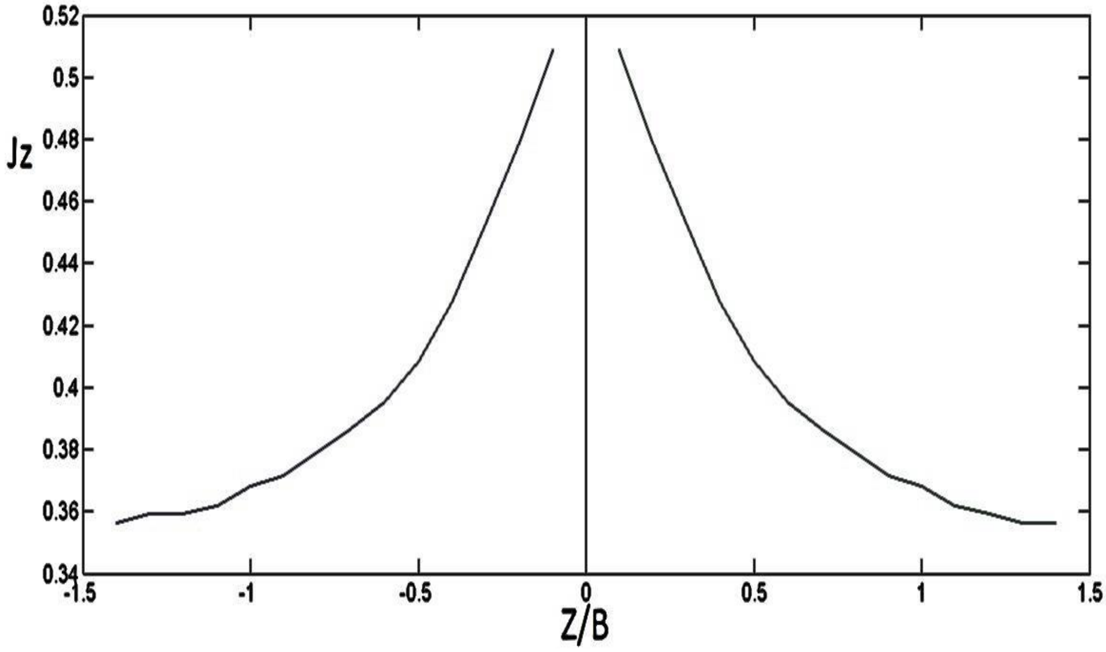
(c) for e/B=0.15,β=0



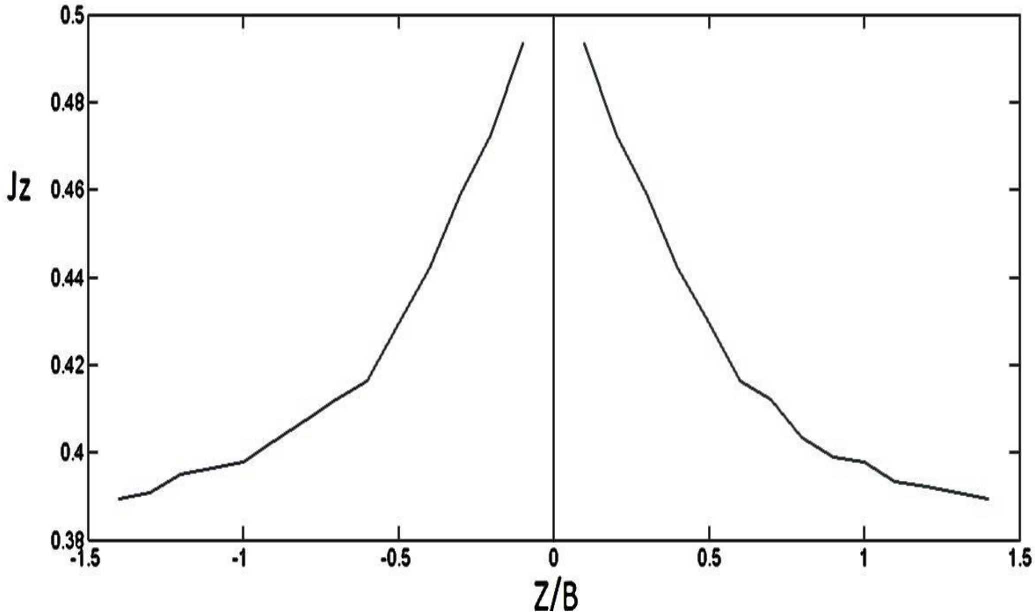
(d)for e/B=0.15,β=20

Fig.3.9: J_Z for a_h =0.0 for different values of eccentricities and inclination angles

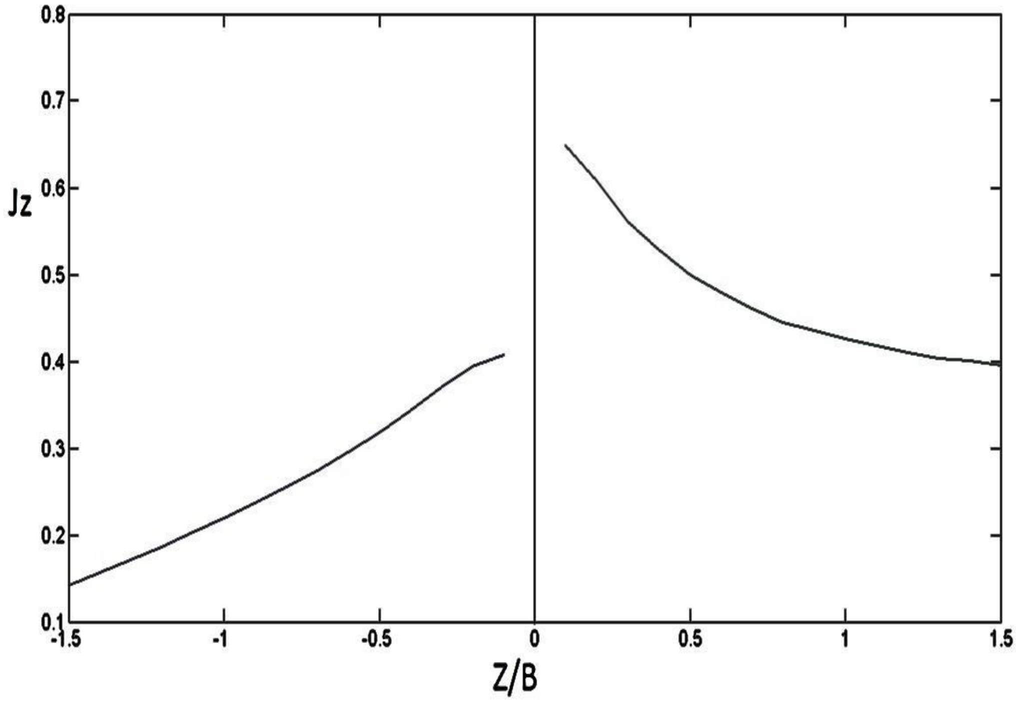
Plots for a_h=0.1



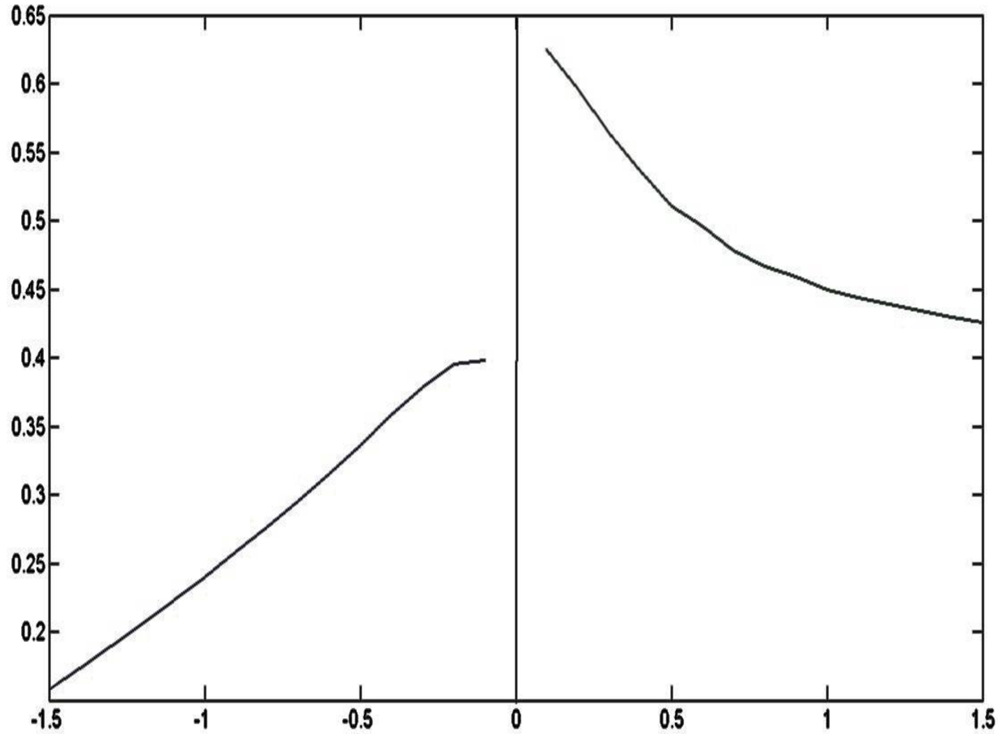
(a) for e/B=0.0,β=0



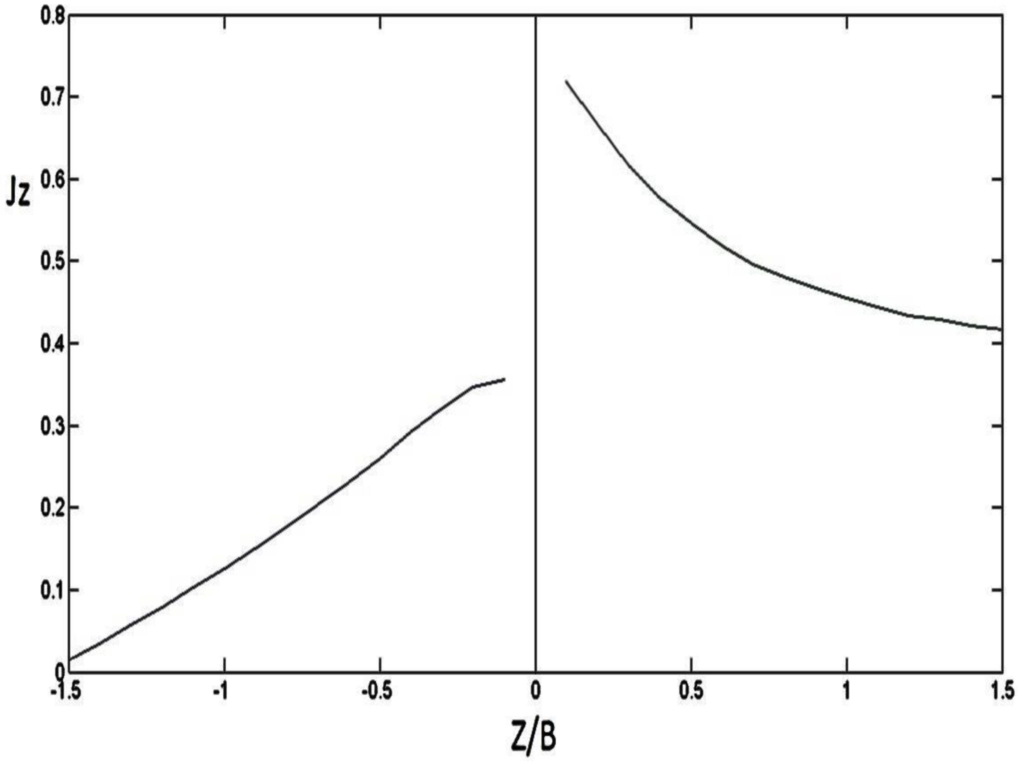
(b) for e/B=0.0,β=20



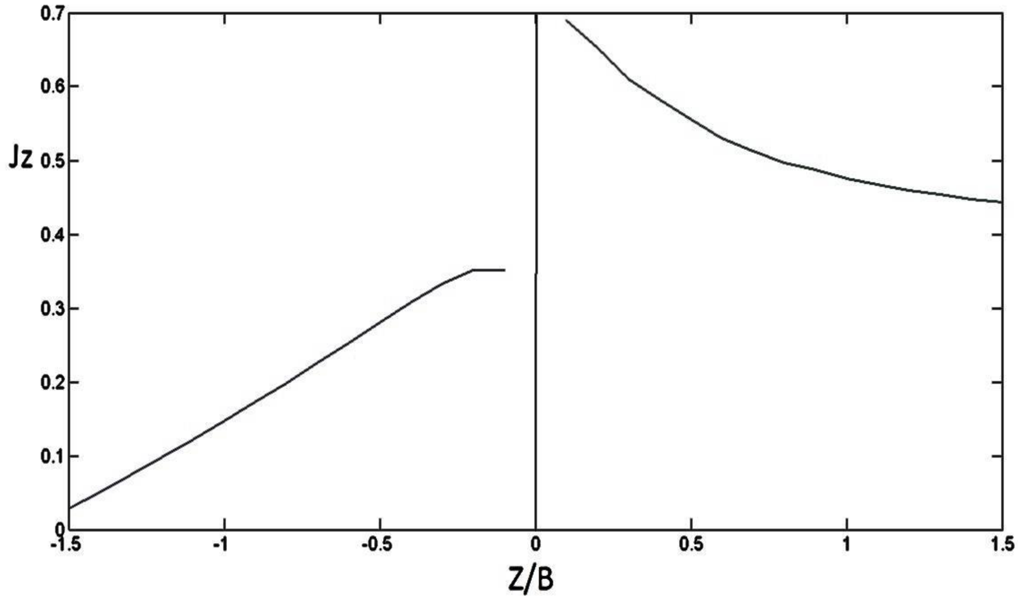
(c) for e/B=0.1,β=0



(d) for e/B=0.1,β=20



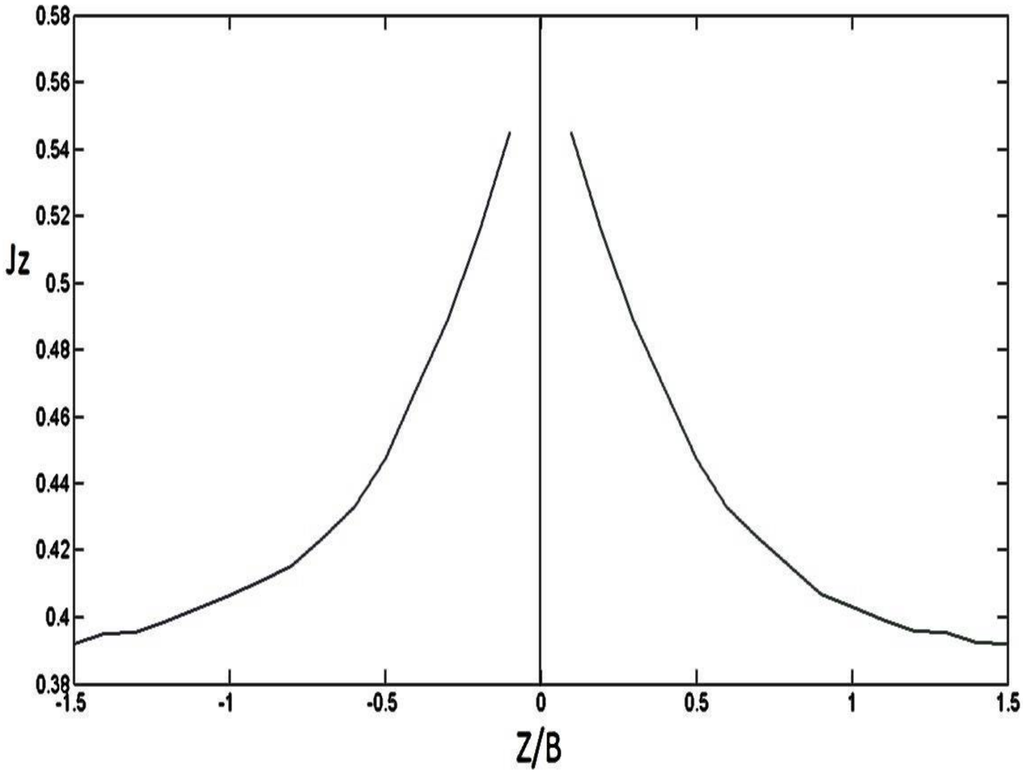
(e) for e/B=0.15,β=0



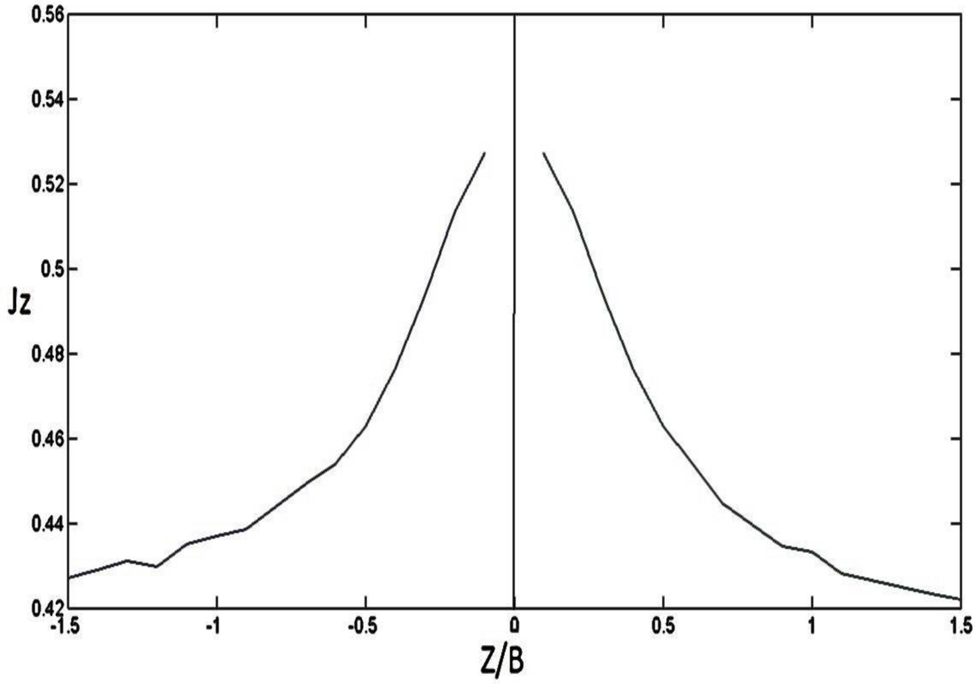
(f) for e/B=0.15,β=20

Fig.3.10: J_Z for a_h =0.10 for different values of eccentricities and inclination angles

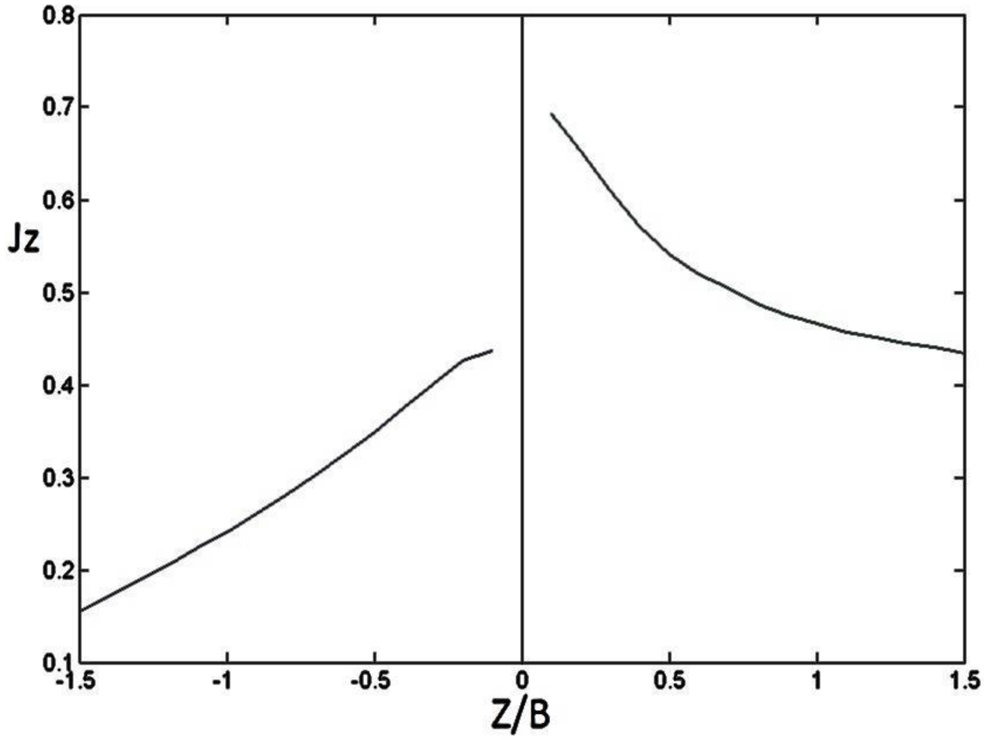
Plots for a_h =0.20



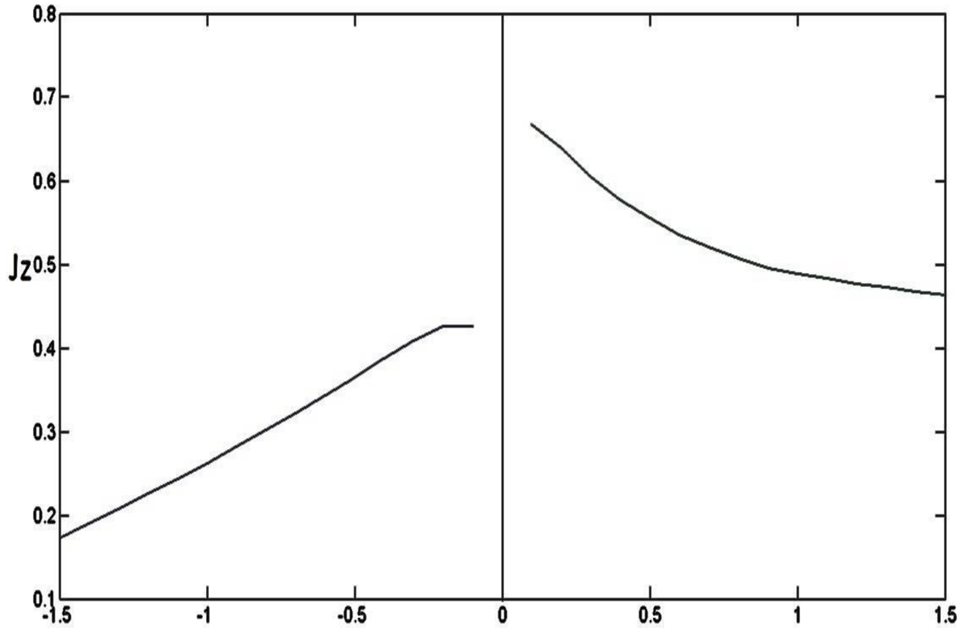
(a) for e/B=0,β=0



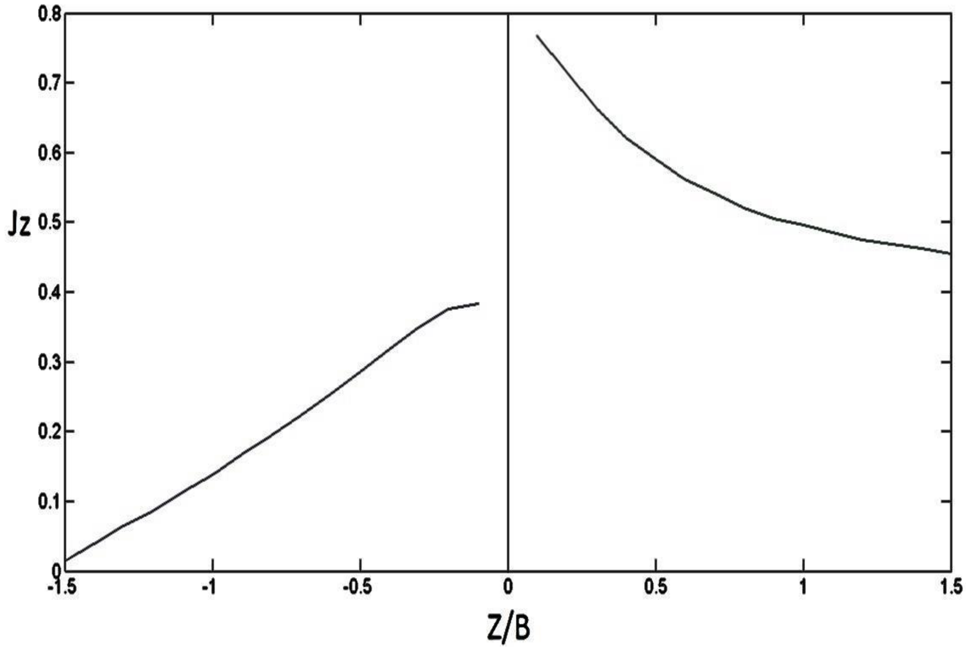
(b) for e/B=0,β=20



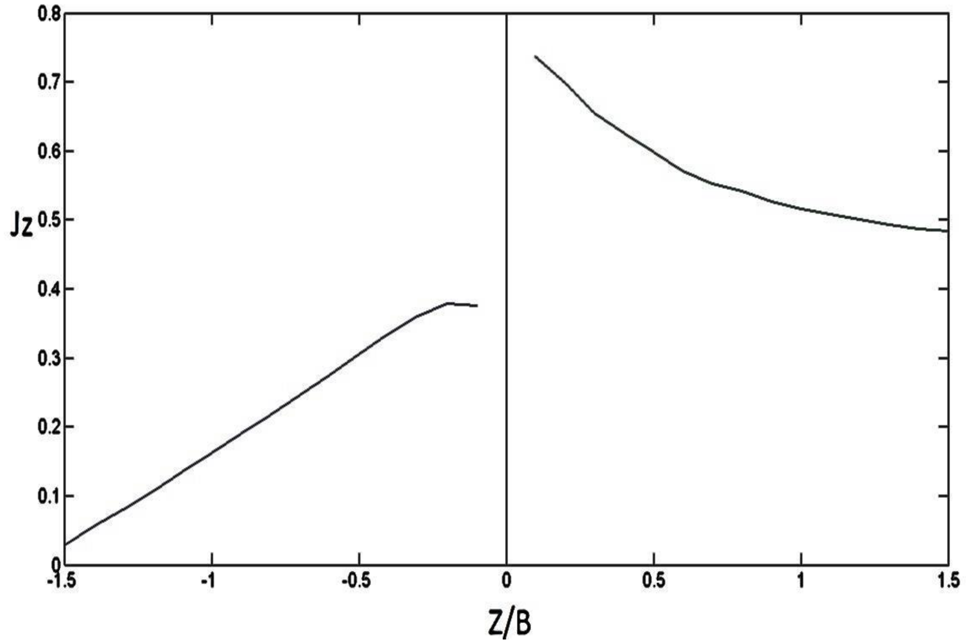
(c) for e/B=0.10,β=0



(d) for e/B=0.10,β=20



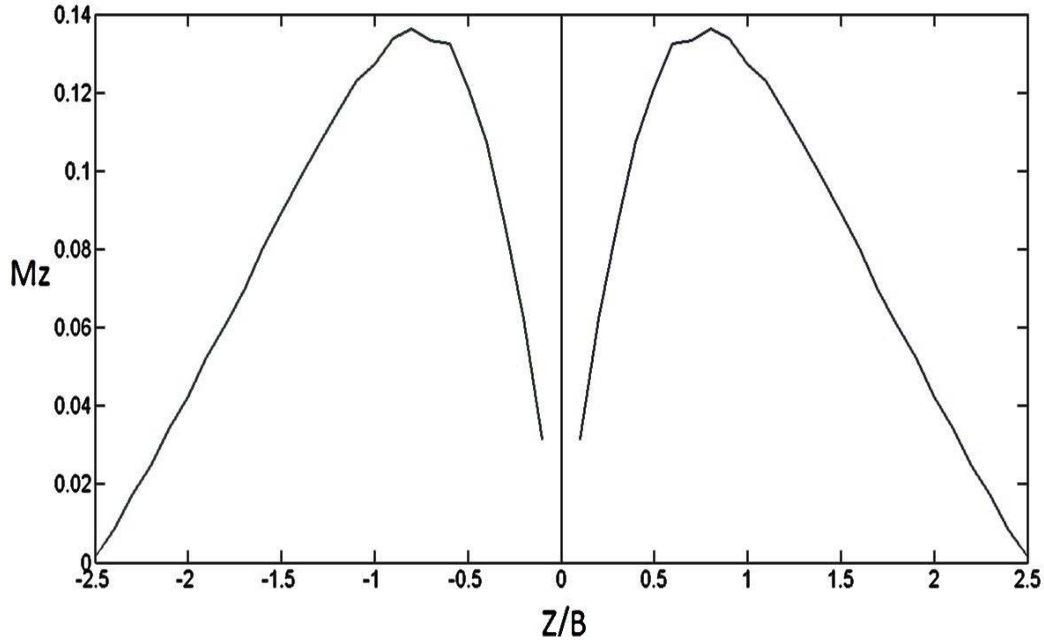
(e) for e/B=0.15,β=0



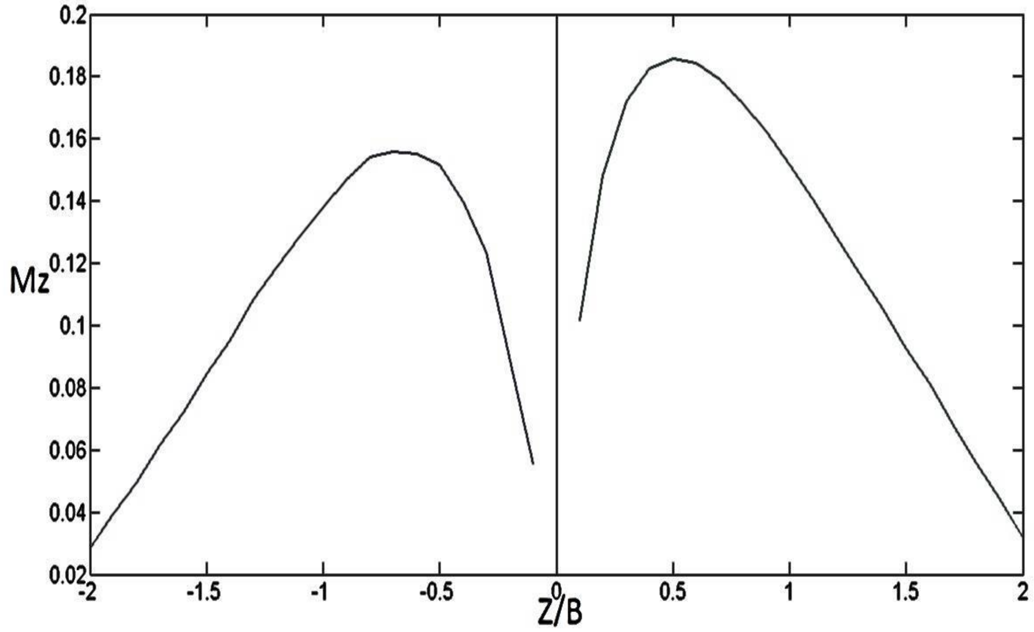
(f) for e/B=0.15,β=20

Fig.3.11: J_Z for a_h =0.10 for different values of eccentricities and inclination angles

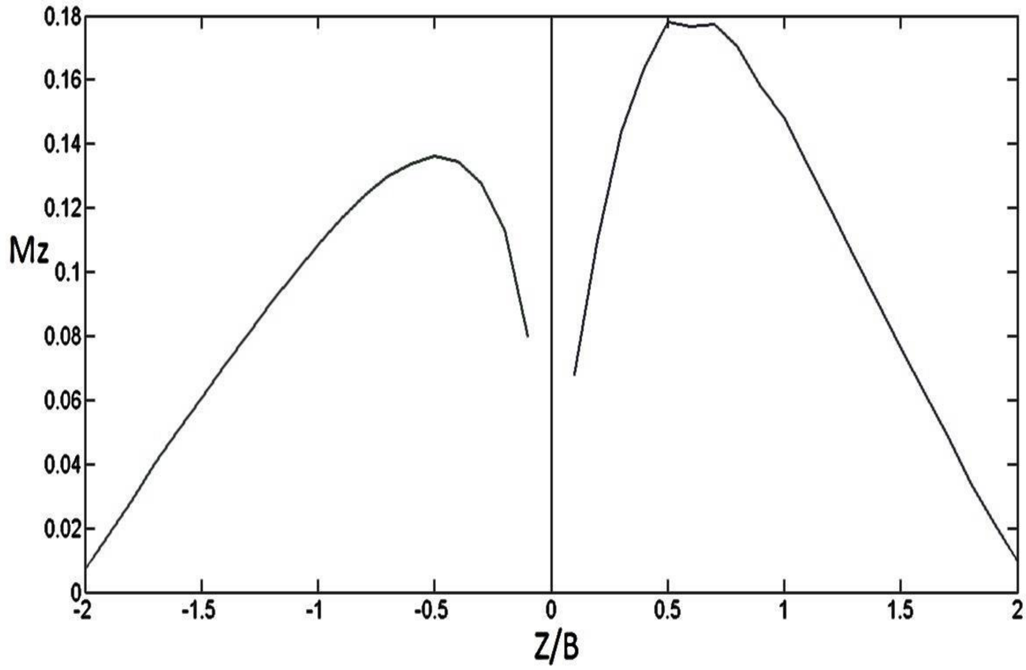
Plots of M_Z for l_0 =1.5B



(a) For a_h=0, e/B=0, β=0



(b) For a_h=0.1, e/B=0.1, β=0



(C)For a_h=0.1, e/B=0.1, β=20

Fig.3.12: M_Z for different values of seismic acceleration, eccentricities and inclination angles

3.8 SAMPLE PROBLEM

A plate load test was conducted on a square plate 600mm x 600mm at a depth of 1.0m below ground surface on a sandy soil ,which extends upto a large depth. From the test data 20mm was taken as the permissible settlement and for this settlement pressure on the actual footing was computed as 72.0KN/m^2.

Design a strip footing of size 1m with its base at a depth of 1.0m after reinforcing the foundation soil with geogrid having yield/rupture strength of 20KN/m^2 and soil reinforcement friction angle from pull out test as 18 degrees.

Take ϒ(soil)=16.3 KN/m^{3}and factor of safety for shear=3.

For non seismic case, take N_{yE}=19.7,N_{qE}=22.5 and φ=30 degrees.

For a_h=0.10 and e=0.10, take N_{yE}=5.122,N_{qE}=11.475 and φ=30 degrees

Take three cases of reinforcement in each condition:

a) Single layer of reinforcement provided at a depth of 0.40m below base of the footing.
b) Two layers of reinforcements are provided, first at a depth of 0.40m and second at a depth of 0.50m.
c) Three layers of reinforcements are provided, first at a depth of 0.40m , second at a depth of 0.50m and third at a depth of 0.60m.

Solution:

1) For reinforced sand
We take outer edge extension of 1.0B i.e.1.00m

Soil tie coefficient of friction $f_e = m \times f$ and $f = \tan\varphi_f$

where, m = mobilization factor given by

$m = [(1-\frac{z}{B})0.7+.3]$ for z/B < 1.0

$m = [(2-\frac{z}{B})0.3]$ for z/B > 1.0

$T_D = [(J_Z B - I_Z \Delta H] q_0 (p_r -1)$

$T_f = 2f_e L_{dr} [M_Z B q_0 p_r ++ L_{dr} \Upsilon (L_0 - X_0)(Z+D_f)]$

Computation of bearing capacity at a_h=0.0

TABLE 3.2 :Calculation of forces for pressure ratio determination at different depths for a_h=0.0

z/B	X_0/B	J_Z	I_Z	M_Z	f_e	ΔH	T_D	T_f
0.40	0.52	0.394	0.275	0.107	0.234	0.40m	$0.284q_0(p_r-1)$	$0.0501q_0p_r+10.466$
0.50	0.54	0.375	0.256	0.121	0.211	0.10m	$0.349q_0(p_r-1)$	$0.0562q_0p_r+9.919$
0.60	0.56	0.358	0.358	0.132	0.189	0.10m	$0.334q_0(p_r-1)$	$0.0498q_0p_r+9.242$

For 20mm settlement and ah=0 ,q_0=72KN/m^2(allowable soil pressure)

We know that, $mT_D=T_f$,where m is mobilization factor. We have three equations from table as under:

$m_1 \times 0.284q_0(p_r-1) =0.0501q_0p_r+10.466$

$m_2 \times 0.349q_0(p_r-1)= 0.0562q_0p_r+9.919$

$m_3 \times 0.334q_0(p_r-1) =0.0498q_0p_r+9.242$

and $m_1+m_2 + m_3 =1$

Case-1: Single layer of reinforcement: m_1 =1, m_2=0

a) For failure in rupture
   $m_1 \times 0.284q_0(p_r-1) =10.466$

   p_r=1.512

b) For failure in pull-out
   $m_2 \times 0.349q_0(p_r-1)= 0.0562q_0p_r+9.919$
   p_r=

   critical p_r=1.512

Case-2: Two layers of reinforcement : $m_1+m_2=1$

a. For failure in rupture:

$m_1 \times 0.284q_0(p_r-1) = 10.466$

$m_2 \times 0.349q_0(p_r-1) = 9.919$

$p_r=1.907$

b. For failure in pull-out:

$m_1 \times 0.284q_0(p_r-1) = 0.0501q_0p_r+10.466$

$m_2 \times 0.349q_0(p_r-1) = 0.0562q_0p_r+9.919$

$m_1+m_2=1$

$p_r = 2.452$

critical $p_r = 1.907$

Case-3: Three layers of reinforcement : $m_1+m_2+m_3=1$

a) For failure in rupture:

$m_1 \times 0.284q_0(p_r-1) = 10.466$

$m_2 \times 0.349q_0(p_r-1) = 9.919$

$m_3 \times 0.334q_0(p_r-1) = 9.242$

$p_r=2.29$

b) For failure in pull-out:

$m_1 \times 0.284q_0(p_r-1) = 0.0501q_0p_r+10.466$

$m_2 \times 0.349q_0(p_r-1) = 0.0562q_0p_r+9.919$

$m_3 \times 0.334q_0(p_r-1) = 0.0498q_0p_r+9.242$

$p_r = 2.452$

critical $p_r = 2.29$

Hence ,pressure intensity corresponding to a settlement of 20mm,for different layered of reinforcement soil ($q=q_0p_r$)

a) Single layer of reinforcement: 72x1.512=104.184KN/m^2.
b) Two layers of reinforcement: 72x1.907=134.064KN/m^2.
c) Three layers of reinforcement: 72x2.29=164.884KN/m^2

From the above example ,it is clear that for a particular given settlement, pressure taken by the strip footing on reinforced soil is higher than the corresponding pressure on the footing resting on unreinforced soil.
Amount of the pressure taken by the footing increases significantly with the increase in the number of reinforcement layers.

2) Ultimate bearing capacity of reinforced sand

$q_{ur} = q_u p_{ru} + \Upsilon D_r N_{qE}$
where,p_{ru} =pressure ratio corresponding to the ultimate bearing pressure ,
q_u is ultimate bearing capacity of strip footing on unreinforced sand at a depth 0f 1.0m.
$q_u = \Upsilon D_r N_{qE} + 0.5*\Upsilon*BN_{\Upsilon E}S_{\Upsilon}$
where S_{Υ} =shape factor
=1(for strip footing having L/B≥ 8)
q_u = 16.3 x 1.0 x22.5+0.5 x 16.3x 19.7
= 527.305KN/m^2
Safe ultimate bearing capacity, q_s =527.305/3
q_s =176.768 KN/m^2

Hence ,the limiting cases are given below:
$m_1 \times 0.284q_u(p_{ru}-1) = 0.0501q_u p_{ru} + 10.466$

$m_2 \times 0.349q_u(p_{ru}-1) = 0.0562q_u p_{ru} + 9.919$

$m_3 \times 0.334q_u(p_{ru}-1) = 0.0498q_u p_{ru} + 9.242$

<u>CASE -1</u> Single layer of reinforcement : m_1 =1, m_2=0

a. <u>For failure in rupture:</u>

$m_1 \times 0.284q_u(p_{ru}-1) = 10.466$
p_{ru} =1.069

b. <u>For failure in pull-out:</u>
$m_1 \times 0.284q_u(p_{ru}-1) = 0.0501q_u p_{ru} + 10.466$
p_{ru} =1.658

critical p_r=1.069

CASE-2: Two layers of reinforcement: $m_1+m_2=1$

a. For failure in rupture:

$m_1 \times 0.284q_u(p_{ru}-1) =10.466$
$m_2 \times 0.349q_u(p_{ru}-1)= 9.919$

p_{ru} =1.123

b. For failure in pull-out:

$m_1 \times 0.284q_u(p_{ru}-1) =0.0501q_up_{ru}+10.466$

$m_2 \times 0.349q_u(p_{ru}-1)= 0.0562q_up_{ru}+9.919$

p_{ru} =1.658

critical p_{ru}=1.123

CASE-3: Three layers of reinforcement : $m_1+m_2+m_3=1$

a. For failure in rupture:

$m_1 \times 0.284q_u(p_{ru}-1) =10.466$

$m_2 \times 0.349q_u(p_{ru}-1)= 9.919$

$m_3 \times 0.334q_u(p_{ru}-1) =9.242$

p_{ru}=1.175

b. For failure in pull out:

$m_1 \times 0.284q_u(p_{ru}-1) =0.0501q_up_{ru}+10.466$
$m_2 \times 0.349q_u(p_{ru}-1)= 0.0562q_up_{ru}+9.919$
$m_3 \times 0.334q_u(p_{ru}-1) =0.0498q_up_{ru}+9.242$

p_{ru}=2.225

critical p_{ru}=1.175

Ultimate bearing capacity of reinforced soil : $q_{ur} = q_u p_{ru} + \Upsilon D_r N_{qE}$

a. Single layer reinforced soil:
   Ultimate bearing capacity
   q_{ur1} =527.308×1.069+16.3×(1+0.4)×22.5=1077.142KN/m^2
   Safe bearing capacity: q_{sr1} =359.047KN/m^2

b. Double layered reinforced soil:
   Ultimate bearing capacity
   q_{ur2} =527.308×1.123+16.3×(1+0.5)×22.5=1142.283KN/m^2
   Safe bearing capacity: q_{sr2} =380.761KN/m^2

c. Three layered reinforced soil:
   Ultimate bearing capacity
   q_{ur3} =527.308×1.175+16.3×(1+0.6)×22.5=1206.387KN/m^2
   Safe bearing capacity: q_{sr3} =402.128KN/m^2

Safe bearing capacity of soil bed increases significantly on reinforcing. Safe bearing capacity of unreinforced soil (i.e.175.769KN/m^2) is increased to 359.044KN/m^2 for Single layer of reinforcing tie,380.761KN/m^2 for two layers of reinforcing tie, and 402.126 KN/m^2 for three layers of reinforcing tie.

Computation of bearing capacity at a_h=0.10 and e=0.10

TABLE 3.3: Calculation of forces for pressure ratio determination at different depths for a_h=0.10 and e=0.10.

z/B	X_0/B	X_0/B	J_{Zm}	I_{Zm}	M_{Zm}	f_e	ΔH	T_D	T_f
0.40	0.56	0.55	0.437	0.359	0.161	0.234	0.40m	$0.358q_0(p_r-1)$	$0.0753q_0p_r+10.03$
0.50	0.59	0.58	0.414	0.323	0.169	0.211	0.10m	$0.382q_0(p_r-1)$	$0.0714q_0p_r+9.196$
0.60	0.63	0.62	0.388	0.290	0.171	0.189	0.10m	$0.294q_0(p_r-1)$	$0.0645q_0p_r+8.554$

For 20mm settlement and ah=0.10 , q_0=67.585KN/m^2 (allowable soil pressure)

We know that, $mT_D=T_f$,where m is mobilization factor .We have three equations from table as under:

$m_1 \times 0.358q_0(p_r-1) =0.0753q_0p_r+10.039$

$m_2 \times 0.382q_0(p_r-1)= 0.0714q_0p_r+9.196$

$m_3 \times 0.294q_0(p_r-1) =0.0645q_0p_r+8.554$

And $m_1+m_2 + m_3 =1$

<u>CASE-1:</u> Single layer of reinforcement : m_1 =1, m_2=0

a. <u>For failure in rupture:</u>

   $m_1 \times 0.358q_0(p_r-1) =10.039$

   p_r=1.414

b. <u>For failure in pull-out:</u>

   $m_1 \times 0.358q_0(p_r-1) =0.0753q_0p_r+10.039$
   p_r=1.79

   critical p_r=1.414

CASE-2: Two layers of reinforcement : $m_1+m_2=1$

a. For failure in rupture:
   $m_1 \times 0.358q_0(p_r-1) = 10.039$
   $m_2 \times 0.382q_0(p_r-1) = 9.196$
   $p_r=1.77$

b. For failure in pull-out:
   $m_1 \times 0.358q_0(p_r-1) = 0.0753q_0p_r+10.039$
   $m_2 \times 0.382q_0(p_r-1) = 0.0714q_0p_r+9.196$
   $m_1+m_2=1$

   $p_r = 2.936$

   critical $p_r = 1.77$

CASE-3: Three layers of reinforcement: $m_1+m_2+m_3=1$

a. For failure in rupture:

   $m_1 \times 0.358q_0(p_r-1) = 10.039$

   $m_2 \times 0.382q_0(p_r-1) = 9.196$

   $m_3 \times 0.294q_0(p_r-1) = 8.554$

   $p_r=2.19$

b. For failure in pull-out:
   $m_1 \times 0.358q_0(p_r-1) = 0.0753q_0p_r+10.039$
   $m_2 \times 0.382q_0(p_r-1) = 0.0714q_0p_r+9.196$
   $m_3 \times 0.294q_0(p_r-1) = 0.0645q_0p_r+8.554$
   $m_1+m_2+m_3=1$

   $p_r = 3.47$

   critical $p_r = 2.19$

Hence, pressure intensity corresponding to a settlement of 20mm, for different layered of reinforcement soil ($q=q_0p_r$)

a) Single layer of reinforcement: 67.585x1.414=95.585KN/m^2.
b) Two layers of reinforcement: 67.585x1.770 =119.625KN/m^2.
c) Three layers of reinforcement: 67.585x2.191 =148.281KN/m^2

For a particular given settlement, pressure taken by the strip footing on reinforced soil is higher than the corresponding pressure on the footing resting on unreinforced soil. Amount of the pressure taken by the footing increases significantly with the increase in the number of reinforcement layers.

1) Ultimate bearing capacity of reinforced sand bed

$q_{ur} = q_u p_{ru} + \Upsilon D_r N_{qE}$

where, p_{ru} =pressure ratio corresponding to the ultimate bearing pressure ,

q_u is ultimate bearing capacity of strip footing on unreinforced sand at a depth 0f 1.0m.

$q_u = \Upsilon D_r N_{qE} + 0.5*\Upsilon*BN_{\Upsilon E}S_{\Upsilon}$

where S_{Υ} =shape factor

=1(for strip footing having $L/B \geq 8$)

$q_u = 16.3 \times 1.0 \times 11.475 + 0.5 \times 16.3 \times 5.122$

$= 228.787 KN/m^2$

Safe ultimate bearing capacity, $q_s = 228.787/3$

$q_s = 76.262\ KN/m^2$

hence ,the limiting cases are given below:

$m_1 \times 0.358q_u(p_{ru}-1) = 0.0753q_u p_{ru} + 10.039$

$m_2 \times 0.382q_u(p_{ru}-1) = 0.0714q_u p_{ru} + 9.196$

$m_3 \times 0.294q_u(p_{ru}-1) = 0.0645q_u p_{ru} + 8.554$

CASE -1 **Single layer of reinforcement: $m_1 = 1, m_2 = 0$**

a. For failure in rupture:

$m_1 \times 0.358q_u(p_{ru}-1) = 10.039$

$p_{ru} = 1.123$

b. For failure in pull-out:

$m_1 \times 0.358q_u(p_{ru}-1) = 0.0753q_u p_{ru} + 10.039$

$p_{ru} = 1.279$

critical $p_r = 1.123$

CASE-2: **Two layers of reinforcement: $m_1 + m_2 = 1$**

a. For failure in rupture:

$m_1 \times 0.358q_u(p_{ru}-1) = 10.039$

$m_2 \times 0.382q_u(p_{ru}-1) = 9.196p_{ru}$

$p_{ru} = 1.228$

b. For failure in pull-out:

$m_1 \times 0.358q_u(p_{ru}-1) = 0.0753q_up_{ru}+10.039$

$m_2 \times 0.382q_u(p_{ru}-1) = 0.0714q_up_{ru}+9.196$

$p_{ru} = 2.036$

critical $p_{ru}=1.128$

<u>CASE-3:</u> Three layers of reinforcement : $m_1+m_2+m_3=1$

a. <u>For failure in rupture</u>:

$m_1 \times 0.358q_u(p_{ru}-1) = 10.039$
$m_2 \times 0.382q_u(p_{ru}-1) = 9.196$
$m_3 \times 0.294q_u(p_{ru}-1) = 8.554$
$p_{ru}=1.355$

b. <u>For failure in pull out:</u>

$m_1 \times 0.284q_u(p_{ru}-1) = 0.0501q_up_{ru}+10.466$
$m_2 \times 0.349q_u(p_{ru}-1) = 0.0562q_up_{ru}+9.919$
$m_3 \times 0.334q_u(p_{ru}-1) = 0.0498q_up_{ru}+9.242$

$p_{ru}=3.53$

critical $p_{ru}=1.355$

Ultimate bearing capacity of reinforced soil : $q_{ur} = q_up_{ru} + \Upsilon D_rN_{qE}$

a. Single layer reinforced soil:
Ultimate bearing capacity
$q_{ur1} = 228.787\times1.123+16.3\times(1+0.4)\times11.475=518.787KN/m^2$
Safe bearing capacity: $q_{sr1} = 172.929KN/m^2$

b. Double layered reinforced soil:
Ultimate bearing capacity
$q_{ur2} = 228.787\times1.228+16.3\times(1+0.5)\times11.475=561.514KN/m^2$
Safe bearing capacity: $q_{sr2} = 187.171KN/m^2$

c. Three layered reinforced soil:
Ultimate bearing capacity
$q_{ur3} = 228.787\times1.355+16.3\times(1+0.6)\times11.475=609.274KN/m^2$
Safe bearing capacity: $q_{sr3} = 203.091KN/m^2$

Safe bearing capacity of soil bed increases significantly on reinforcing. Safe bearing capacity of unreinforced soil (i.e.76.262KN/m^2) is increased to 172.9KN/m^2 for Single layer of reinforcing,187.171KN/m^2 for two layers of reinforcing, and 203.091 KN/m^2 for three layers of reinforcing.

Computation of bearing capacity at a_h=0.10, e=0.10 and inclination angle=20

TABLE 3.4: Calculation of forces for pressure ratio determination at different depths for a_h=0.10 , e=0.10 and inclination angle=20.

z/B	X_0/B	X_0'/B	J_{Zm}	I_{Zm}	M_{Zm}	f_e	ΔH	T_D	T_f
0.40	0.57	0.56	0.447	0.437	0.149	0.234	0.40m	$0.403q_0(p_r-1)$	$0.0697q_0p_r+9.932$
0.50	0.60	0.59	0.424	0.387	0.157	0.211	0.10m	$0.385q_0(p_r-1)$	$0.0663q_0p_r+9.299$
0.60	0.65	0.65	0.406	0.346	0.156	0.189	0.10m	$0.372q_0(p_r-1)$	$0.0588q_0p_r+8.357$

For 20mm settlement and ah=0.10 ,q_0=67.585KN/m^2(allowable soil pressure)

We know that , $mT_D=T_f$,where m is mobilization factor, we have three eqns from table as under

$m_1 \times 0.403q_0(p_r-1) = 0.0697q_0p_r+9.932$

$m_2 \times 0.385q_0(p_r-1) = 0.0663q_0p_r+9.299$

$m_3 \times 0.372q_0(p_r-1) = 0.0588q_0p_r+8.357$

and $m_1+m_2+m_3=1$

CASE-1: Single layer of reinforcement : $m_1=1, m_2=0$

a. For failure in rupture:
   $m_1 \times 0.403q_0(p_r-1) = 9.932$
   p_r=1.364
b. For failure in pull-out:
   $m_1 \times 0.403q_0(p_r-1) = 0.0697q_0p_r+9.932$
   p_r=1.649

critical p_r=1.364

CASE-2: Two layers of reinforcement : $m_1+m_2=1$

a. For failure in rupture:

$m_1 \times 0.403q_0(p_r-1) = 9.932$

$m_2 \times 0.385q_0(p_r-1) = 9.299$

$p_r=1.721$

b. For failure in pull-out:

$m_1 \times 0.403q_0(p_r-1) = 0.0697q_0p_r+9.932$

$m_2 \times 0.385q_0(p_r-1) = 0.0663q_0p_r+9.299$

and $m_1+m_2 = 1$

$p_r = 2.626$

critical $p_r = 1.721$

CASE-3: Three layers of reinforcement : $m_1+m_2+m_3=1$

(a) For failure in rupture:

$m_1 \times 0.403q_0(p_r-1) = 9.932$

$m_2 \times 0.385q_0(p_r-1) = 9.299$

$m_3 \times 0.372q_0(p_r-1) = 8.357$

and $m_1+m_2+m_3 = 1$

$p_r=2.053$

(b) For failure in pull-out:

$m_1 \times 0.403q_0(p_r-1) = 0.0697q_0p_r+9.932$

$m_2 \times 0.385q_0(p_r-1) = 0.0663q_0p_r+9.299$

$m_3 \times 0.372q_0(p_r-1) = 0.0588q_0p_r+8.357$

and $m_1+m_2+m_3 = 1$

$p_r = 4.132$

critical $p_r = 2.053$

Hence, pressure intensity corresponding to a settlement of 20mm, for different layered of reinforcement soil ($q=q_0p_r$)

(a) Single layer of reinforcement: 67.585x1.364=92.186KN/m^2.
(b) Two layers of reinforcement: 67.585x1.721 =116.314KN/m^2.
(c) Three layers of reinforcement: 67.585x2.053=138.750KN/m^2

For a particular given settlement, pressure taken by the strip footing on reinforced soil is higher than the corresponding pressure on the footing resting on unreinforced soil. Amount of the pressure taken by the footing increases significantly with the increase in the number of reinforcement layers.

Ultimate bearing capacity of reinforced sand

$q_{ur} = q_u p_{ru} + \Upsilon D_r N_{qE}$
where, p_{ru} =pressure ratio corresponding to the ultimate bearing pressure ,
q_u is ultimate bearing capacity of strip footing on unreinforced sand at a depth 0f 1.0m.
$q_u = \Upsilon D_r N_{qE} + 0.5 * \Upsilon * B N_{\Upsilon E} S_{\Upsilon}$
where S_{Υ} =shape factor
=1(for strip footing having $L/B \geq 8$)
q_u = 16.3 x 1.0 x11.475+0.5 x 16.3x 5.122
= 228.787KN/m^2
Safe ultimate bearing capacity, q_s =228.787/3
q_s =76.262 KN/m^2

hence ,the limiting cases are given below:

$m_1 \times 0.403 q_u (p_{ru}-1) = 0.0697 q_u p_{ru} + 9.932$

$m_2 \times 0.385 q_u (p_{ru}-1) = 0.0663 q_u p_{ru} + 9.299$

$m_3 \times 0.372 q_u (p_{ru}-1) = 0.0588 q_u p_{ru} + 8.357$

and $m_1 + m_2 + m_3 = 1$

CASE -1: Single layer of reinforcement : $m_1 = 1, m_2 = 0$

(a) For failure in rupture:
$m_1 \times 0.403 q_u (p_{ru}-1) = 9.932$
$p_{ru} = 1.108$

(b) For failure in pull-out:
$m_1 \times 0.403 q_u (p_{ru}-1) = 0.0697 q_u p_{ru} + 9.932$
$p_{ru} = 1.339$
critical $p_r = 1.108$

CASE-2: Two layers of reinforcement: $m_1 + m_2 = 1$

(a) For failure in rupture:
$m_1 \times 0.403 q_u (p_{ru}-1) = 9.932$

$m_2 \times 0.385q_u(p_{ru}-1) = 9.299$
and $m_1+m_2+m_3=1$
$p_{ru} = 1.124$

(b) For failure in pull-out:
$m_1 \times 0.403q_u(p_{ru}-1) = 0.0697q_u p_{ru}+9.932$
$m_2 \times 0.385q_u(p_{ru}-1) = 0.0663q_u p_{ru}+9.299$
and $m_1+m_2=1$
$p_{ru} = 1.853$
critical $p_{ru}=1.124$

CASE-3: Three layers of reinforcement : $m_1+m_2+m_3=1$

a. For failure in rupture:
$m_1 \times 0.403q_u(p_{ru}-1) = 9.932$
$m_2 \times 0.385q_u(p_{ru}-1) = 9.299$
$m_3 \times 0.372q_u(p_{ru}-1) = 8.357$
and $m_1+m_2+m_3=1$
$p_{ru}=1.312$

b. For failure in pull out:
$m_1 \times 0.284q_u(p_{ru}-1) = 0.0501q_u p_{ru} + 10.466$
$m_2 \times 0.349q_u(p_{ru}-1) = 0.0562q_u p_{ru} + 9.919$
$m_3 \times 0.334q_u(p_{ru}-1) = 0.0498q_u p_{ru}+9.242$
$p_{ru}=2.64$
critical $p_{ru}=1.312$

Ultimate bearing capacity of reinforced soil : $q_{ur} = q_u p_{ru} + \Upsilon D_r N_{qE}$

a. Single layer reinforced soil:
Ultimate bearing capacity,
$q_{ur1}=228.787\times1.108+16.3\times(1+0.4)\times11.475=515.355KN/m^2$
safe bearing capacity : $q_{sr1} = 171.785KN/m^2$

b. Double layered reinforced soil:
Ultimate bearingcapacity,
$q_{ur2} = 228.787\times1.124+16.3\times(1+0.5)\times11.475=558.311KN/m^2$
safe bearing capacity : $q_{sr2} = 186.104KN/m^2$

c. Three layered reinforced soil:
   Ultimate bearing capacity,
   q_{ur3} =228.787×1.312+16.3× (1+0.6)×11.475=599.436KN/m^2
   safe bearing capacity : q_{sr3} =199.812KN/m^2

Safe bearing capacity of soil bed increases significantly on reinforcing. Safe bearing capacity of unreinforced soil (i.e.76.262KN/m^2) is increased to 171.785KN/m^2 for Single layer of reinforcing tie, 186.104KN/m^2 for two layers of reinforcing tie, and 199.812 KN/m^2 for three layers of reinforcing tie.

TABLE SP1: Comparison of Pressure Ratios of strip footing for different layers of reinforcement and for different value of a_h and e/B values.

No.of Reinforcement layers	Static Case	Dynamic Case		Pressure Ratios	
	a_h=0.0, e/B=0.0	a_h=0.1,e/B=0.1	a_h=0.1,e/B=0.1 β=20°	$\frac{p_r(a_h=0.1, e/B=0.1)}{p_r(a_h=0.0, e/B=0.0)}$	$\frac{p_r(a_h=0.1, e/B=0.1, \beta=20)}{p_r(a_h=0.0, e/B=0.0)}$
	p_r	p_r'	p_r'	p_r' / p_r	p_r'/p_r
Single	1.465	1.414	1.364	0.935	0.902
Two	1.907	1.770	1.721	0.923	0.903
Three	2.291	2.191	2.053	0.957	0.897

TABLE SP2: **Comparison of Bearing Capacity Ratios of strip footing for different layers of reinforcement and for different value of a_h and e/B values.**

No.of Reinforcement layers	Static Case	Dynamic Case		Bearing Capacity Ratios	
	a_h=0.0, e/B=0.0	a_h=0.1,e/B=0.1	a_h=0.1,e/B=0.1 β=20	$\frac{q_{uE}\ (a_h=0.1,\ e/B=0.1)}{q_{us}\ (a_h=0.0,\ e/B=0.0)}$	$\frac{q_{uE}\ (a_h=0.1, e/B=0.1, \beta=20)}{q_{us}\ (a_h=0.0,\ e/B=0.0)}$
	q_{us}	q_{uE}	q_{uE}	q_{uE} / q_{us}	q_{uE} /q_{us}
None	527.308	228.287	228.787	0.434	0.434
One	1077.142	518.787	515.335	0.482	0.478
Two	1142.283	561.514	558.311	0.492	0.489
Three	1206.387	609.274	599.416	0.505	0.497

CHAPTER-4
CONCLUSION

From the analysis carried out to determine the seismic bearing capacity of strip footing resting on reinforced earth bed subjected to eccentric inclined loading followings conclusions are made --

- Non dimensional parameters (I_Z,J_Z,M_Z) plots have develop for obtaining seismic bearing capacity of strip footing resting on reinforced earth bed.
- For a given eccentricity,seismic acceleration and inclination angle for the strip footing,it was found that the pressure ratio increases with increase in the layers of reinforcement.
- For a given layer of reinforcing ties,pressure ratio decreases with increase in eccentricity ,seismic acceleration and inclination angle.
- It was found that the bearing capacity decreases with increase in eccentricity,seismic acceleration and inclination angle in camparison to vertical static loading.
- For a given value of seismic excitation (a_h),inclination angle(β) and eccentricity (e/B),single layer of reinforcement in the soil bed under the strip footing, bearing capacity increases significantly.On increasing the number of reinforcing ties (two or three layers) the seismic bearing capacity of strip footing resting on reinforced soil increases by lesser amounts.

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