ELASTIC-ANALYSIS OF SOIL-FOUNDATION INTERACTION

Dissertation submitted in partial fulfillment for the award of Degree

of

MASTER OF ENGINEERING

in

CIVIL ENGINEERING (STRUCTURAL ENGINEERING)

Submitted by

PANKAJ
(Roll No:-3505)

UNDER THE GUIDANCE OF

A. TRIVEDI

Professor

Department of Civil Engineering

DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

DELHI COLLEGE OF ENGINEERING

DELHI UNIVERSITY, DELHI

2003-2005

Prof. A.TRIVEDI

Department of Civil Engineering

Delhi College of Engineering

New Delhi-110042

India

CERTIFICATE

It is hereby certify that the content of the thesis entitled "**ELASTIC ANALYSIS OF SOIL-SURFACE INTERACTION"** is a bonafide work carried by **PANKAJ (Roll No-3505)** in partial fulfillment of the requirement for the award of the Master of Engineering with specialization in Structural Engineering at Delhi College of Engineering Delhi. He carries out the work under my supervision and guidance.

I therefore, recommended the acceptance of this dissertation work for the award of the degree of Master of Engineering with specialization in Structural Engineering.

Place: D.C.E. Delhi (Prof. A. Trivadi)

Date:

Mr. MANISH GUPTA

Senior Research Officer

Central Soil and Material Research Station (CSMRS)

New Delhi

India

CERTIFICATE

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Place: CSMRS, New Delhi (Manish Gupta)

Date:

ACKNOWLEDGEMENT

I would like to take this opportunity to thanks all those who have been a constant force of inspiration and have helped in the exercise of preparing the present study.

I express our sincere gratitude to **Prof. A. Trivadi**, (Department of Civil Engineering, Delhi College of Engineering Delhi), my project guide, for his constant inspiration, encouragement, guidance and constructive criticism and judicious evaluation that led to the compilation of this project work. It was due to his constant help and assistant that this minor project achieved its present shape.

I am thankful to **Prof. P.R.Bose**, HOD Civil Engineering Department, Delhi College of Engineering, for their valuable guidance, kind support and constant encouragement throughout the present study.

It is my duty to express my deepest gratitude and thanks to **Mr. Manish Gupta** Senior Research Officer, Central Soil and Material Research Station (CSMRS), New Delhi for their valuable guidance, kind support and constant encouragement throughout the present study.

I would like to extend my sincere thanks to the Officers of Soil discipline, Central soil and material research station, New Delhi for their kind co-operation in the laboratory testing on soil.

I express my sincere thanks to all my friends and staff of Civil Engineering Department of Delhi College of Engineering who left no stone unturned whenever i needed their assistance.

Place: D.C.E. Delhi

Date ....

(Pankaj)

(Roll No-3505)

Abstract

Over the years, many methods have been developed to predict the settlement of shallow foundations on cohesionless soil. However, methods for making such predictions with the required degree of accuracy and consistency have not yet been developed. Accurate prediction of settlement is essential since settlement, rather than bearing capacity, generally controls foundation design.

The settlement of foundations under working load conditions is an important design consideration. Well-designed foundations induce stress-strain states in the soil that are neither in the linear elastic range nor in the range usually associated with perfect plasticity. In this study, we analyze the load-settlement response of vertically loaded footings placed in sands using both the finite element method and the conventional elastic approach.

The interaction among structures, its foundations and the soil medium below the foundations alter the actual behavior of the structure considerably than what is obtained from the consideration of the structure alone. Thus, a reasonably accurate model for the soil-foundation-structure interaction system with computational validity, efficiency and accuracy is needed in improved design of important structures. The study makes an attempt to gather the possible alternative models available in the literature for this purpose.

In this study, a Conventional Elastic Method (CEM) is used in an attempt to obtain more accurate settlement prediction for non-linear behavoiur of the soil. The predicted settlements found by utilizing CEM are compared with a Finite Element Method (PLAXIS). The results indicate that CEM is a useful technique for predicting the settlement of shallow foundations on cohesionless soils, as they outperform the traditional methods for nonlinear behavoiur of the soil.

A numerical model is purposed to analyze the elastic behaviour of the foundation in cohessionless soils foundation is assumed to be constructed in a homogeneous and nonlinear medium. The purposed model is implemented in an axi-symmetry finite element code. The effect of the diameter of the footing on the settlement behaviour of the foundation-soil system also focused Finite Element Method based software namely PLAXIS has been used for the present study.

Plaxis is a finite element package that has been developed specifically for the analysis of deformation and stability in Geotechnical Engineering Projects. The simple graphical input procedures enable a quick generation of complex finite element models, and the enhanced output facilities provide a detailed presentation of computational results.

The study shows that stresses and settlement vary considerably with varying the elastic modulus of soil under the foundation, as well as with changing the footing size.

CONTENTS

CHAPTER-1

INTRODUCTION

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INTRODUCTION

1.1 GENERAL

Soil and foundation problems and solution were certainly not limited to any one era, geographic location, or particular people in history. While clear and specific detail are lacking for an accurate and comprehensive evolution of the state of the art, evidence exists of successful solutions to foundation problems faced by some of the earlier builders. For example, the many structures, aqueducts, bridges, roads, and do on built by the Romans 2000 years ago provides some proof of their mastery of the art in foundation design. Similarly, the Great Wall of China and the pyramid of Egypt are further evidence of the suitable foundation. Perhaps they may be credited as the early initiators of a basic approach to the solution of problems of soil mechanics and foundation.

Though the evidence cited above shows that some knowledge existed during ancient civilization of the interaction of a superstructure with the soil supporting it. There is insufficient evidence to suggest that these ancient people had a systematic approach to the solution of their foundation-related problems. In all probability, their basic knowledge of soil mechanics was rather skimpy, quote probably on structured, limited in scope and perhaps confined to the geography of a given region. During that era, lack of transportation and writing proved to be major obstacles to the dissemination of knowledge and ideas over wide areas, thereby imposing limitations to the propagation of new information.

Needless to say, the ancient designers experienced failures as well as successes, in some cases due to bad Engineering judgment and in others due to highly inadequate knowledge of soil properties. They certain lacked the technology of sampling, testing and evaluating the subsurface conditions. In all probability, their knowledge was derived though experience, through trial and error, and through commonsense approaches. It is quite possible that if the structure did not perform satisfactorily, it would be replaced with a new one that did, and so, on.

1.2 BEHAVIOUR OF SOIL MEDIA

The mechanical response of naturally occurring soil can be influenced by a variety of factors. These include, the shape, size, and mechanical properties of the individual soil particles, the configuration of the soil structure, the inte-rgranular stresses, stress history and the presence of soil moisture, the degree of saturation sand the soil permeability.

To solve a soil-foundation interaction problem taking into account all such material characteristics is clearly a difficult task. In order to obtain meaningful and reliable information for practical problems of soil-foundation interaction it becomes necessary to idealize the behavior of the soil by taking into account specific aspect of its behavior. The simplest type of idealized soil response assumes linear-elastic behavior of the supporting soil medium.

In the analysis of the soil-foundation interaction problems, it is general assumed that the soil medium could be adequately represented by an elastic medium occupying a half -space region. In practice, of course the foundation is usually located at some depth below the ground surface. The surface of the soil medium is assumed to form the soil-foundation interaction. The linear elastic idealizations of mathematical model, which exhibit the particular characteristics of soil behavior, several such idealizations have been developed. The simplest model of linear elastic behavior of supporting soil medium is generally attributed to Winkler. He assure that the surface displacement of the soil medium at every point is directly proportional to the stress applied to it at that point completely independent of stresses or displacements at other even immediately neighboring points of the soil-foundation interface.

1.3 SOIL-SURFACE INTERACTION

The mechanical behaviors of soil media is so complex that a mathematical simulation of the same is always a mammoth task to the Engineers. Soil is basically composed of particulate materials. The behaviour of soil, mainly the stress–strain–time property, influences the soil–structure interaction phenomenon.

Physically, when a load is applied on the soil mass (not completely saturated), the soil particles tend to attain such a structural configuration that their potential energy will be a minimum and hence stability is achieved. Up to a certain stress level, strain imparted to the soil mass in this process is elastic and then it may enter the plastic range depending on the magnitude of the applied load. This deformation is followed by a mostly viscoplastic deformation (dominant for fine-grained soil) due to viscous intergranular behaviour that implies strain with passage of time. This deformation occurs by the expulsion of the pore fluid and simultaneous transfer of excess pore pressure to the solid soil grains. Hence, the rate of such strain approaches a small value after a long time. The strain caused by the expulsion of water from the soil mass is identically equal to the strain of the soil skeleton. This is because soil skeleton is an aggregate of mineral particles, which together with bound water constitutes the soil mass. This process is known as primary consolidation. However, after primary consolidation of the soil-structure, continues to adjust to the load for some additional time and secondary compression occurs approximately following a logarithmic function of time. But it is to be noted that the settlement of any representative soil specimen may come to an end beforehand if the range of elasticity of soil is sufficient compared to the applied load. Then the strain will not be a function of time, but for such a fully saturated soil sample, strain will always be the function of time; since the pore fluid under such condition and then visco-elastic settlements will first share the external load will occur. It has been observed that the hardening of soil due to consolidation and the thixotropic processes must be taken into analysis as it causes manifold increase in the cohesion and angle of internal friction of soil. Thus well-selected rheological models in conjunction with the model to represent the phenomenomenologacal behaviour may offer some useful means to study the interactive system.

CHAPTER-2

LITERATURE REVIEW

CHAPTER-2

LITERATURE REVIEW

2.1 Interaction between elastic bodies

A complete analysis of the interaction problem for elastic bodies generally requires the determination of stress and strains within the soil and information regarding the distribution of displacement and stresses at the contact regions.

In this, deformable bodies of differing elastic characteristic are pressed together by external forces. The contact region between the bodies may be smooth or it may exhibit frictional characteristics giving rise to normal and shear traction at the contact surface. On addition the contact region may be advancing, receding or stationary. In problems with non-stationary contact, the extant of the contact region constitutes an additional unknown, which needs to be determined from the complete solution of the interaction problem. Considerable simplifications in the treatment of this elastic contact neighborhood of the area of contact to permit treatment by analytical methods available for elastic half-space regions.

2.2 Interaction between elastic bodies and structural elements

This interaction problem constitutes a special case of the general interaction problem between elastic bodies in which the mechanical behavior of one of the media is represented in terms of the behavior of a structural element such as a beam plate or shell. Idealized soil models prove to be particularly useful in the analysis of soil-foundation interaction problems. The relevant choice of an idealized behavior of the soil for a soil-foundation interaction problem is not necessarily unique, it will depend on a variety of factors including the type of the soil and soil condition, the type of foundation and the nature of external loading. In addition to this due consideration should be given to more purpose and life span of the structure and economical consideration.

2.3 Elastic models of soil under foundation

We shall consider here models of soil responded which exhibit purely elastic characteristics. From a physical point of view, an elastic material or an elastic medium will deform under the application of an external force system. On releasing of these external forces the material or the medium regains its original configuration. If the curve of unloading does not coincide with the loading curve, the material or the medium exhibits inelastic properties. In such media energy is dissipated during a loading cycle. We shall therefore restrict our attention to models of soil behavior for which the relationship between the applied forces and the resulting displacements are given by linear functions.

2.3.1 Winkler model

The idealized model of soil media proposed by Winkler assumes that the deflection of the soil medium at any point on the surface is directly proportional to the stress applied at that point and independent of stresses applied at other locations as shown in fig [1.1(a)].

Physically Winkler's idealization of the soil medium consists of system of mutually independent spring elements with spring constant k. one important feature of this soil model is that the displacement occurs immediately under the loaded area and outside this region the Winkler model the displacements of a loaded region will be constant whether the soil is subjected to an infinitely rigid load or a uniform flexible load.

Winkler's idealization represents the soil medium as a system of identical but mutually independent, closely spaced, discrete, linearly elastic springs. According to this idealization, deformation of foundation due to applied load is confined to load regions only shows the physical representation of the Winkler foundation. The pressure-deflection relation at any point is given by.

$P=kw$

Where p is the pressure, k is the coefficient of sub-grade reaction or sub-grade modulus, and w is the deflection as shown in fig [1.1(a)].

2.3.2 Limitation of Winkler model

The fundamental problem with the use of this model is to determine the stiffness of elastic springs used to replace the soil below foundation becomes two-fold since the numerical value of the coefficient of sub-grade reaction not only depends on the nature of the sub-grade, but also on the dimensions of the loaded area as well.

Modulus of sub-grade reaction or the coefficient of sub-grade reaction k is the ratio between the pressure p at any given point of the surface of contact and the settlement y produced by the load at that point. The value of sub-grade modulus may be obtained in the following alternative approaches.

[1] Plate load test, [2] Consolidation test, [3] Triaxial test, [4] CBR test

Following some suitable method mentioned to estimate k, a reasonable value of sub-grade modulus, the only parameter to dialyze soil stiffness, may be obtained in the absence of suitable test data, representative values for the same may be chosen following the guideline the basic limitations of Winkler hypothesis lies in the fact that this model cannot account for the dispersion of the load over a gradually increasing influence area with increase in depth. Moreover, it considers linear stress–strain behavior of soil. The most serious demerit of Winkler model is the one pertaining to the independence of the springs. So the effect of the externally applied load gets localized to the sub-grade only to the point of its application. This implies no cohesive bond exists among the particles comprising soil medium.

2.3.3 Elastic continuum model

Since the surface deflections that occur on a Winkler model are limited to the load region, this restricts its applicability to soil media, which possess the slightest amount of cohesion or transmissibility of applied forces. As per Korenev (1954,1960), It is common experience that in the case of soil media surface deflections will occur not only immediately under the loaded region but also within certain limited zones outside the loaded region. In attempts to account for this continuous behavior, soil medium have often been idealized as three-dimensional continuums elastic solids or elastic continua.

This is a conceptual approach of physical representation of the infinite soil media. Soil mass basically constitutes of discrete particles compacted by some inter-granular forces.

The genesis of continuum representation for the soil media is perhaps from the research work of Boussinesq (1885) and Gorbunov-Posadov (1941,1949), to analyze the problem of a semi-infinite, homogeneous, isotropic, linear elastic solid subjected to a concentrated force acting normal to the boundary, using the theory of elasticity. In this case, some continuous function is assumed to represent the behavior of soil medium. In fact, later on it has been concluded that the deflection line of its surface under a unit concentrated load can best describe the nature of supporting elastic medium of any type. In the continuum idealization, generally soil is assumed to be semi-infinite and isotropic for the sake of simplicity. However, the effect of soil layering and anisotropy may be conveniently accounted for in the analysis. This approach provides much more information on the stresses and deformations within soil mass than Winkler model. It has also the important advantage of simplicity of the input parameters, viz., modulus of elasticity and Poisson's ratio. Solutions for some practical problems idealizing the soil media as elastic continuum are available for few limited cases.

2.3.4 Limitation of continuum model

One of the major drawbacks of the elastic continuum approach is inaccuracy in reactions calculated at the peripheries of the foundation. It has also been found that, for soil in reality, the surface displacements away from the loaded region decreased more rapidly than what is predicted by this approach. Thus, this idealization is not only computationally difficult to exercise but often fails to represent the physical behavior of soil very closely, too.

2.3.5 Isotropic elastic continuum model

We consider the isotropic elastic continuum model, which can be effectively employed in the analytical treatment of soil-foundation interaction problems. The response function for the linear elastic half-space or characterized by the centered shape of the surface of the elastic medium subjected to a concentrated force or a uniform stress of finite extent as shown in fig [1.1].

Plane strain exists in the x, z plane. The displacement component v in the y-direction is zero and the displacements u and w in the z-directions respectively, are purely function of these coordinates.

The stress and displacement in an isotropic elastic half-plane to a concentrated normal line load P applied at its surface. The surface displacement in the x-direction is given by

$w(x, 0)=\{2P(1-\nu_s^2)/\pi\times E_s\}\times\log|x|+c$ ……………………..(1)

Where $|x|$ is the numerical value of x and c is an arbitrary constant.

E_s and ν_s are the modules of elastic and Poisson's ratio for the linear elastic material.

and for x=d,

$C=-2P(1-\nu_s^2)\log(d)/\pi\times E_s$……………………..(2)

Now stress and displacement in an isotropic elastic half-plane to a uniform distributed load applied at its surface. The surface displacement in the x-direction is given by.

$w(x, 0)=\{q(1-\nu)/\pi\times G_s\}[2b+(x-b)\log|x-b|-(x+b)\log|x+b|]$

And the displacement at the origin has a finite magnitude of

$w(0,0)=2qb(1-\nu)/\pi\times G_s\{1-\log b\}$

Here: w is the deflection, b is the width of loading, ν is the Poison's ratio and G_s is the Shear Modulus.

2.4 IMPROVED FOUNDATION MODEL

In order to take care of the shortcomings of then basic approaches, viz., Winkler's model and Continuum model, some modified foundation models have bee proposed in the literature. These modifications have generally be suggested following two alternate approaches, in the first approach the Winkler foundation is modified to introduce continuity through interaction amongst the spring elements by some structural elements and in the second approach, continuum model is simplified to obtain a more realistic picture in terms of expected displacement and/or stresses.

2.4.1 Improved versions of continuum model.

3.4.1.1 Vlasov model

Starting from continuum idealization this foundation model has been developed using variational principle by Filonenko-Borodich (1940,1945), This model imposes certain restrictions upon the possible deformations of an elastic layer. As per this model

The vertical displacement $w(x,z)=w(x)h(z)$, such that $h(0)=1$ and $h(H)=0$.this function $h(z)$ describe the variation of displacement in vertical direction.

The horizontal displacement $u(x,z)$ is assumed to be zero everywhere in the soil.

The function h (z) may be assumed to be linearly decreasing with depth for a classical foundation of finite thickness H. Hence, in this case, $h(z)-1-(z/H)$. For the foundation resting on relatively thick (or infinite thickness) elastic layer, the choice may be $h(z)=(\sinh[\nu(H-z)]/\sinh[\nu h]$, where ν is a coefficient depending on the elastic properties of the foundation defining the rate of decrease of displacements with depth. Then using the principle of virtual work, response function for this model is obtained.

Using the stress-strain relation for plane strain conditions by Pasternak (1954),

$\sigma_{xx}=\{E_0/(1-\nu^2_0)\}w(x)\times dh(z)/dz[\nu_0]$……………………(3)

$\sigma_{zz}=\{E_0/(1-\nu^2_0)\}w(x)\times dh(z)/dz[1]$ and ………………..(4)

$\tau_{xz}=\{E_0/(1+\nu^2_0)\}w(x)\times dw(z)/dx[h(z)]$…………………(5)

For linear variation of h(z) from (3),(4),(5) we get

$\sigma_{zz}=\{E_0/(1-\nu^2_0)\times H\}w(x)$…………………………………(6)

$\tau_{zx}=\{E_0/2\times(1+\nu^2_0)\}dw(z)/dx[1-z/H]$……………………..(7)

Here:

$E_0=E_s/(1-\nu^2_s)$.

$\nu_0=\nu_s/(1-\nu^2_s)$.

To examine the response of the Vlazov model in detail it is instructive to obtain the surface deflection profile for the plane strain problem of a soil layer, which is subjected to a line load P per unit length.

We have response function as

$q(x)=kw(x)-2td^2w(x)/dx^2$

Where:

$k=E_s/(1-\nu^2_0)\int(dh/dz)^2dz$ and

$t=E_s/4(1+\nu_s))\int(h)^2dz$.

k=coefficient of sub-grade reaction

t=transmissibility of applied force

Take linear variation of transverse displacement with depth we get

$w(x,z)=[3(1-\nu^2_s)P/\{6(1-\nu_s)^{1/2}\}E_0]e\text{-}\alpha x[1-z/H]$

Where:

$\alpha=\{6(1-\nu_0)^{1/2}\}/H(1-\nu_0)$

It is clear that the surface displacements of the elastic layer decrease rapidly with increasing distance from the point of application of the line load.

Similarly, in the case of the non-linear variation of w(x,z) with depth is

$w(x,z)=[3(1-\nu^2_s)P/\{6(1-\nu_s)\}^{1/2} E_0\times\psi_t\times\psi_\alpha]e^{-\alpha x}(\sinh[\gamma(H-z)/L]/\sinh[\gamma H/L]$

Where:

$\psi_\alpha=[\psi_k/\psi_\alpha]^{1/2}$

$\psi_\alpha=[\{6(1-\nu_0)\}^{1/2}/H(1-\nu_0)]\ \psi_\alpha$

2.5 APPLICATION OF THE MODELS

The various foundation models discussed herein utilize a number of parameters to represent the behaviour of the soil. Thus, the determination of the parameters that constitute the model is the basic requirement. Modulus of sub-grade reaction can be conveniently determined from plate load test. The values so obtained can be easily modified for the actual footing.

Studies have been reported in the area of soil–structure interaction replacing the soil in a number of different ways. Out of all the models available, Winkler foundation utilizes only a single parameter. This can be very conveniently determined and suitably modified for actual foundation size, shape, etc. to employ in actual analysis. The fundamental limitation of Winkler idealization lies with the independent behaviour of the soil springs. Since the degree of continuity of the structure is sufficiently higher than the soil media, this approximation may not be far from reality.

The validity of Winkler's assumptions has been strongly established for Gibson (1974) type soil medium, where shear modulus of soil varies linearly with depth. It is also recognized in the literature that even large error in the assessment of the values of the sub-grade modulus influences the response of the superstructure quite in-significantly. The present practice in design offices generally adopts a fixed base consideration for structural analysis and design. In this context, the Winkler model though oversimplified, seems adequate and suitable for computational purpose for its reasonable performance and simplicity.

2.6 Elastic half-space?

A soil medium with a horizontal ground surface extending laterally to infinite length, and downwards from the horizontal is called semi-infinite medium or semi-infinite half-space. Of such a medium is assumed to be homogeneous, isotropic and elastic, then it is called elastic half-space. Theoretical treatment for determining stresses on such a medium may be done using the theory of elasticity.

2.7 Elastic properties of soil

The relationship between deformation and strain with stress is important in understanding the behaviour of any material. For some building materials, Hooke's law provides a useful approximation between stress and strain. But this law does necessarily hold good for soils, as their behaviour in general is a non-linear and not perfectly elastic. It is observed that the entire stress-strain graph is a curve, unlike in steel where the initial portion is predominantly a straight line. The slope of the initial portion of the curve is defined as the stress-strain modulus (E).

2.8 Contact pressure

The pressure transmitted from the base of a foundation to the soil is termed the contact pressure. This depends on the rigidity of the foundation structure and the nature of the soil as shown in fig [1.2].

The presence of a thick compressible layer, like soft clay beneath a flexible foundation presents a bowl shaped settlement profile with more settlement at the center and almost zero at the edge. But the pressure distribution is uniform. This is the conventional distribution pattern used in the calculation of stresses and settlements.

An extremely rigid footing on the same clay will settle a uniform amount across its breadth. This compressible cohesive soil under a rigid footing has to transmit high contact pressure near the edges than at the center so as to maintain a uniform settlement. The contact pressure distribution is shown in fig [1.2].

For a flexible foundation resting on a non-cohesive soil, the distribution of contact Pressure is uniform, but the edges of the foundation experience a large settlement Because of lack of confining pressure at the edges, the foundation settles more. The settlement of a rigid footing on sand layer is uniform and the contact pressure increases from zero at the edge to a maximum at the center.

In actual practice, no foundation is perfectly flexible nor infinitely rigid, and hence the actual distribution of contact pressure is somewhere between the extreme values. Sufficient accuracy in the calculation of stresses and displacements can be obtained by assuming a uniform distribution of contact pressure.

2.9 METHOD OF FINDING THE STRESS IN SOIL UNDER FOUNDATION

2.9.1 Boussinesq's equations

In 1885 Joseph Valentin Boussinesq advanced theoretical expressions for determining stresses at a point within an ideal mass due to surface point loads as shown in fig [1.3(a)]. They are based on the assumption that the mass is an (1) Elastic (2) isotropic (3) homogeneous (4) semi-infinite medium that extends infinitely in all directions from a level surface.

$\sigma_z=3.Q.z^3/2.\pi.R^5$

$\sigma_x=(3.Q.z^3/2.\pi)\{x^2 z/ R^5+(1-2\mu)/3[1/R(R+z)-(2R+z).x^2/R^3(R+z)^2-z/R^3]\}$

$\sigma_y=(3.Q.z^3/2.\pi)\{x^2 z/ R^5+(1-2\mu)/3[1/R(R+z)-(2R+z).x^2/R^3(R+z)^2-z/R^3]\}$

$\tau_z=3.Q.z^2.X/2.\pi.R^5$

$\tau_{yz}=3.Q.z^2.y/2.\pi.R^5$

$\tau_{yx}=3.Q/2.\pi.[xyz/R^5-\{1-2\mu/3\}(2R+z)xy/R^3(R+z)^2]$

Note- Stress does not depend of the elastic or other properties of soil i.e. it is independent of the material content of the medium

2.9.2 Westergaard equation

Some fine-grained soil is interspersed with thin lenses of coarse-grained material that partially prevent lateral deformation of the soil. Such a situation represents the non-homogeneous condition. Westergaard (1938) suggested a solution to such a material by considering an elastic medium in which the lateral strain was assumed zero. As some of the soils are of this type, the Westergaard solution may be taken as a better approximation for such soils than that proposed by Bouddinrdq's for homogeneous soils. Westergaard expression for the vertical stress is

$\sigma_z=(Q./2.z^2.\pi)\{(1-2\nu)^{0.5}/(2-2\nu)\}/[(1-2\nu)(2-2\nu)+(r/z)^2]^{1.5}$.

2.9.3 Stress due to infinite length footing line load

Stress at a point A due to a line load q per unit length on the surface (fig [1.4(b)]) are given as

$\sigma_z = (2.q/\pi)[z^3/(x^2+z^2)]$

$\tau_{rz} = (2.q/\pi)[x.z^2/(x^2+z^2)^2]$

2.9.4 Stress due to Strip footing area caring uniform load

A strip of width B and infinite length with uniform pressure the stresses (fig [1.4(c)]) at point A are given as

$\sigma_z = (q/\pi)[\alpha+\sin\alpha.\cos(\alpha+2\beta)]$

$\tau_{xz} = (q/\pi)[\sin\alpha.\cos(\alpha+2\beta)]$

2.9.5 Vertical Stress due to circular footing caring uniform load

Two cases of stresses due to a uniform pressure (fig [1.5(d)]) on a circular area are available (1) stresses under the center of the circular area and (2) stresses at any point on the soil. The vertical stresses at a depth z under the center of a circular area of diameter 2a is:

$\sigma_z=q[1-\{1//(1+(a/z)^2\}^{1.5}]$

The vertical stresses at a depth z under the center and at any other point in footing of a circular area of diameter 2a is calculated by graph given below

m=z/r ,n=a/r

$\sigma_z = q.N_{ca}(m,n)$

Where

Nca= shape function of dimensionless variables (taken from graph)

r= radius of footing

a= horizontal distance from center of footing

Note: - Value of N_{ca} for different value of n and m are given in table [1.1] and shown graphically in fig.1.1

CHAPTER-3

CHARACTERSTIC

OF SOIL

CHAPTER-3

CHARACTERSTIC OF SOIL

3. INTRODUCTION TO SOIL MODULI

The modulus of a soil is one of the most difficult soil parameters to estimate because it depends on so many factors. Therefore when one says for example:" The modulus of this soil is 10,000 kPa", one should immediately ask: "What are the conditions associated with this number?" The following is a background on some of the important influencing factors for soil moduli. It is not meant to be a thorough academic discourse but rather a first step in understanding the complex world of soil moduli. First the modulus is defined, and then the factors influencing the modulus and related to the state of the soil are described, the factors related to the loading process are discussed, further some applications of soil moduli are presented.

Poisson's Ratio-A standard procedure for evaluation of Poisson's ratio for soil does not exist. Poisson's ratio for soil usually varies from 0.25 to 0.49 with saturated soils approaching 0.49. Poisson's ratio for unsaturated soils usually varies from 0.25 to 0.40. A reasonable overall value for Poisson's ratio is 0.40. Normal variations in elastic modulus of foundation soils at a site are more significant in settlement calculations than errors in Poisson's ratio.

The elastic modulus is sensitive to soil disturbance which may increase pore water pressure and, therefore, decrease the effective stress in the specimen and reduce the stiffness and strength. Fissures, which may have little influence on field settlement, may reduce the measured modulus compared with the in-situ modulus if confining pressures are not applied to the soil specimen.

3.1 How does one obtain a modulus from a stress strain curve?

In order to answer this question, the example of the stress strain curve obtained in a triaxial test is used. The sample is a cylinder; it is wrapped in an impermeable membrane and confined by an all around (hydrostatic) pressure. Then the vertical stress is increased gradually and the non-linear stress strain curve is obtained.

Elasticity assumes that the strains experienced by the soil are linearly related to the stresses applied. In reality this is not true for soils and there lies one complexity. The equations of elasticity for this axis-symmetric loading relate the stresses and the strains in the three directions as shown in Fig. 3.1.

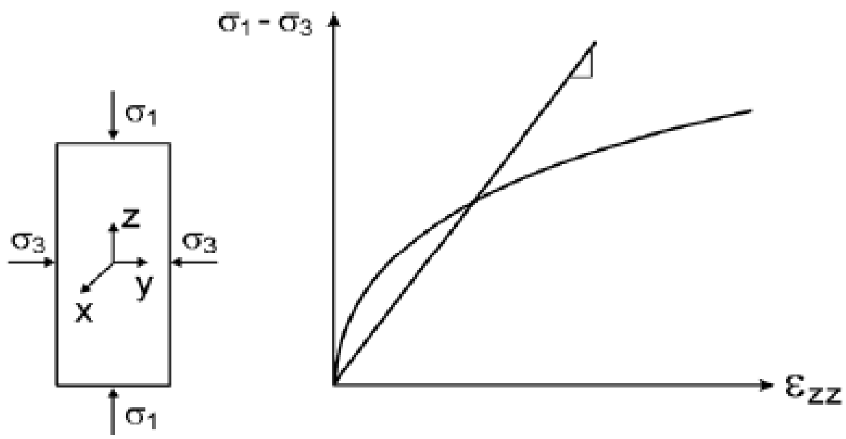


Fig.3.1 calculating a modulus

$$\varepsilon_{xx} = \frac{1}{E}\left(\sigma_{xx} - \nu(\sigma_{yy} + \sigma_{zz})\right) = \frac{1}{E}\left(\sigma_3 - \nu(\sigma_1 + \sigma_3)\right) \qquad (1)$$

$$\varepsilon_{yy} = \frac{1}{E}\left(\sigma_{yy} - \nu(\sigma_{xx} + \sigma_{zz})\right) = \frac{1}{E}\left(\sigma_3 - \nu(\sigma_1 + \sigma_3)\right) \qquad (2)$$

$$\varepsilon_{zz} = \frac{1}{E}\left(\sigma_{zz} - \nu(\sigma_{xx} + \sigma_{yy})\right) = \frac{1}{E}\left(\sigma_1 - \nu(\sigma_3 + \sigma_3)\right) \qquad (3)$$

$$E = \frac{\sigma_1 - 2\nu\,\sigma_3}{\varepsilon_{zz}} \qquad (4)$$

$$\frac{\varepsilon_{xx}}{\varepsilon_{zz}} = \frac{\sigma_3 - \nu\,(\sigma_1 + \sigma_3)}{\sigma_1 - 2\nu\,\sigma_3} \qquad (5)$$

In equations (1) and (3) there are two unknowns: the soil modulus E and the Poisson's ratio ν. In the triaxial test, it is necessary to measure the stresses applied in both directions as well as the strains induced in both directions in order to calculate the modulus of the soil. Indeed one needs two simultaneous equations to solve for E and ν. Note that **the modulus is not the slope of the stress strain curve**. An exception to this statement is the case where the confining stress is zero as it is for a typical concrete cylinder test or an unconfined compression test on clay. In order to calculate the Poisson's ratio, it is also necessary to measure the stresses applied in both directions as well as the strains induced in both directions. Note also that **the Poisson's ratio is not the ratio of the strains in both directions** (equation (5) on Fig.3.1). An exception to this statement is again the case where the confining stress is zero.

3.2 Which modulus? Secant, tangent, unload, reload, or cyclic modulus?

Because soils do not exhibit a linear stress strain curve, many moduli can be defined from the triaxial test results for example. In the previous paragraph, it was pointed out that the slope of the stress strain curve is not the modulus of the soil. However the slope of the curve is related to the modulus and it is convenient to associate the slope of the stress strain curve to a modulus. Indeed this gives a simple image tied to the modulus value; note however that in the figures the slope is never labeled as modulus **E** but rather as slope S. Referring to Figure 3.2.

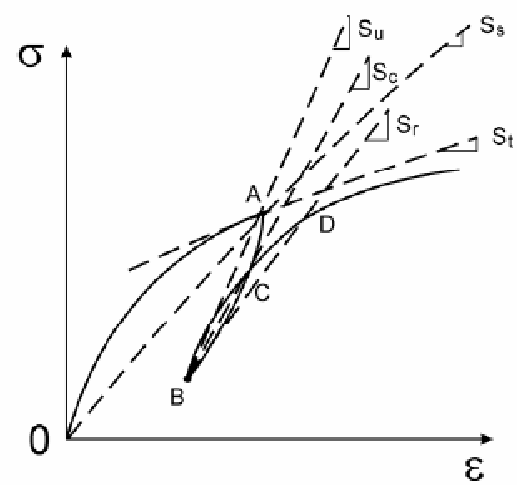


Fig.3.2 Definition of soil modulus

If the slope is drawn from the origin to a point on the curve (O to A on Figure 3.2), the secant slope S_s is obtained and the secant modulus E_s is calculated from it. One would use such a modulus for predicting the movement due to the first application of a load as in the case of a spread footing. If the slope is drawn as the tangent to the point considered on the stress strain curve then the tangent slope S_t is obtained and the tangent modulus E_t is calculated from it. One would use such a modulus to calculate the incremental movement due to an incremental load as in the case of the movement due to one more story in a high-rise building. If the slope is drawn as the line which joins points A and B on Fig. 3.2, then the unloading slope S_u is obtained and the unloading modulus E_u is calculated from it. One would use such a modulus when calculating the heave at the bottom of an excavation or the rebound of a pavement after the loading by a truck tire (resilient modulus). If the slope is drawn from point B to point D on Fig. 3.2, then the reloading slope S_r is obtained and the reload modulus E_r is calculated from it. One would use this modulus to calculate the movement at the bottom of an excavation if the excavated soil or a building of equal weight was placed back in the excavation or to calculate the movement of the pavement under reloading by the same truck tire. If the slope is drawn from point B to point C on Fig. 3.2, then the cyclic slope S_c is obtained and the cyclic modulus E_c is calculated from it. One would use such a modulus and its evolution as a function of the number of cycles for the movement of a pile foundation subjected to repeated wave loading. Whichever one of these moduli is defined and considered, the state in which the soil is at a given time will affect that modulus.

3.3 FACTORS ON WHICH MODULUS OF ELASTICITY DEPEND

3.3.1 how closely packed are the particles?

If they are closely packed, the modulus tends to be high. This is measured by the dry density (ratio of the weight of solids over the total volume of the wet sample) of the soil for example; it can also be measured by the porosity (ratio of the volume of voids over the total volume of the wet sample).

3.3.2 How are the particles organized?

This refers to the structure of the soil. For example a coarse grain soil can have a loose or dense structure and a fine grain soil can have a dispersed or flocculated structure. Note that two soil samples can have the same dry density yet different structures and therefore different soil moduli. This is why taking a disturbed sample of a coarse grain soil in the field and reconstituting it to the same dry density and water content in the laboratory can lead to laboratory and field moduli which are different.

3.3.3 What is the water content?

This parameter has a major impact because at low water contents the water binds the particles (especially for fine grained soils) and increases the effective stress between the particles through the suction and tensile skin of water phenomenon.

Therefore in this case low water contents lead to high soil moduli. This is why clay shrinks and becomes very stiff when it dries. At the same time at very low water contents the compaction of coarse grain soils is not as efficient as it is at higher water contents because the lubrication effect of water is not there. Therefore in this case very low water contents lead to low moduli. As the water content increases, water lubrication increases the effect of compaction and the modulus increases as well. However if the water content rises beyond an optimum value, the water occupies more and more room and gets to the point where it pushes the particles apart thereby increasing compressibility and reducing the modulus.

3.3.4 What has the soil been subjected to in the past?

This is referred to as the stress history factor. If the soil has been pre-stressed in the past it is called over consolidated. This pre-stressing can come from a glacier, which may have been 100 meters thick 10,000 years ago and has now totally melted. This pre-stressing can also come from the drying and wetting cycles of the seasons in arid parts of the world. If the soil has not been pre-stressed in the past, in other words if today's stress is the highest stress experienced by the soil and if the soil is at equilibrium under this stress, the soil is normally consolidated. An over-consolidated (OC) soil will generally have higher moduli than the same normally consolidated (NC) soil because the OC soil is on the reload part of the stress strain curve while the NC soil is on the first loading part. Some soils are still in the process of consolidating under their own weight. These are called under-consolidated soils such as the clays deposited offshore the Mississippi Delta where the deposition rate is more rapid than the rate, which would allow the pore water pressures, induced by deposition to dissipate. These clays have very low moduli.

3.3.5 What about cementation?

This refers to the "glue" which can exist at the contacts between particles. As discussed above, low water contents in fine-grained soils can generate suction in the water strong enough to simulate a significant "glue effect" between particles. This effect is temporary, as an increase in water content will destroy it. Another glue effect is due to the chemical cementation, which can develop at the contacts. This cementation can be due to the deposition of calcium at the particle-to-particle contacts for example. Such cementation leads to a significant increase in modulus.

These are some of the most important factors related to the state of the soil and influencing its modulus.

3.4 LOADING FACTORS ON WHICH MODULUS OF ELASTICITY DEPEND

3.4.1 what is the mean stress level in the soil?

The loading process induces stresses in the soil. These stresses can be shear stresses or normal stresses or a combination of both. At one point and at any given time in a soil mass there is a set of three principal normal stresses. The mean of these three stresses has a significant influence on the soil modulus. This is also called the confinement effect. Figure. 3.3(a) shows an example of two stress strain curves at two different confinement levels. As common sense would indicate, the higher the confinement is, the higher the soil modulus will be. A common model for quantifying the influence of the confinement on the soil modulus is given on Figure. 3.3(a) and is usually attributed to the work of Kondner. According to this model, the modulus is proportional to a power law of the confinement stress. The modulus E_0 is the modulus obtained when the confinement stress is equal to the atmospheric pressure Pa. A common value for the power exponent a in Figure. 3.3(a) is 0.5.

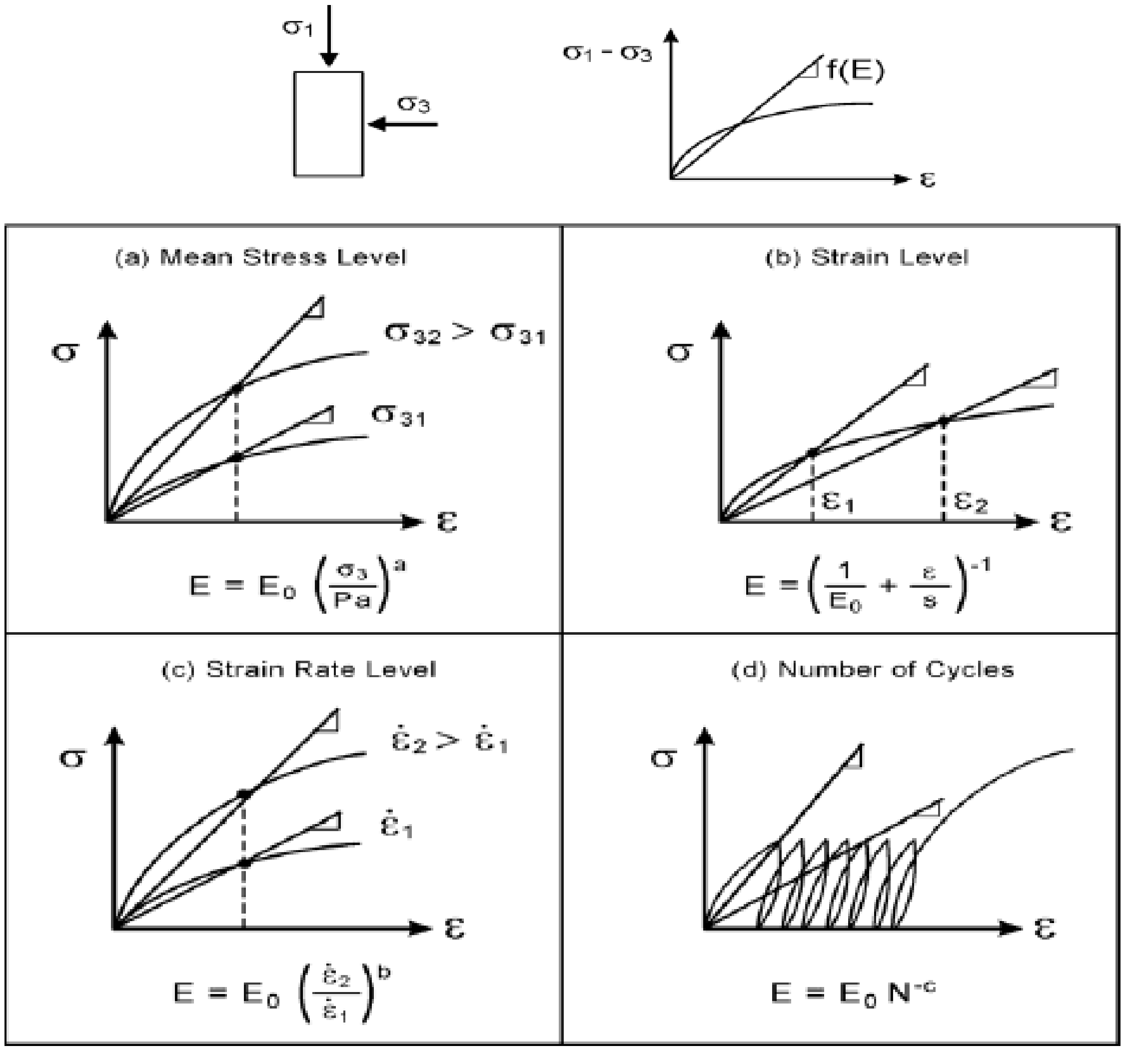


Fig. 3.3 Loading factors for soil moduli

3.4.2 What is the strain level in the soil?

The loading process induces strains in the soil mass. Because soils are nonlinear materials, the secant modulus depends on the mean strain level in the zone of influence. In most cases the secant modulus will decrease as the strain level increases because the stress strain curve has a downward curvature. Note that an exception to this downward curvature occurs when the results of a consolidation test is plotted as a stress strain curve on arithmetic scales for both axes. Indeed in this case the stress strain curve exhibits an upward curvature because the increase in confinement brought about by the steel ring is more influential than the decrease in modulus due to the increase in strain in the soil. In the triaxial test, the stress strain curve can be fitted with a hyperbola and the associated model for the modulus is shown on Fig. 3.3(b). This hyperbolic model is usually attributed to the work of Duncan. In this model (Fig. 3.3(b)), E_0 is the initial tangent modulus also equal to the secant modulus for a strain of zero. The parameter s is the asymptotic value of the stress for a strain equal to infinity. In that sense it is related to the strength of the soil.

3.4.3 What is the strain rate in the soil?

Soils like many other materials are viscous. This means that the faster a soil is loaded, the stiffer it is and therefore the higher the modulus is. In some instances the reverse behavior is observed. Fig. 3.3(c) shows an example of two stress strain curves obtained by loading the soil at two drastically different strain rates. The strain rate is defined as the strain accumulated per unit of time. The modulus usually varies as a straight line on a log-log plot of modulus versus strain rate. The slope of that line is the exponent b in Fig. 3.3(c). In clays, common values of this exponent vary from 0.02 for stiff clays to 0.1 for very soft clays. In sands common values of b vary from 0.01 to 0.03. The modulus E_0 is the modulus obtained at a reference strain rate.

3.4.4 What is the number of cycles experienced by the soil?

If the loading process is repeated a number of times, the number of cycles applied will influence the soil modulus. Again referring to the secant modulus, the larger the number of cycles the smaller the modulus becomes. This is consistent with the accumulation of movement with an increasing number of cycles. The model used to describe this phenomenon is shown on Fig. 3.3(d). The exponent c in the model is negative and varies significantly. The most common values are of the order of -0.1 to -0.3.

3.4.5 Is there time for the water to drain during the loading process?

Two extreme cases can occur: drained or undrained loading. The undrained case may occur if the drainage valve is closed during a laboratory test or if the test is run sufficiently fast in the field. The time required to maintain an undrained behavior or to ensure that complete drainage takes place depends mainly on the soil type. For example a 10-minute test in highly plastic clay is probably undrained while a 10-minute test in clean sand is probably a drained test. The Poisson's ratio is sensitive to whether or not drainage takes place. For example if no drainage takes place during loading in clay it is common to assume a Poisson's ratio equal to 0.5. On the other hand if complete drainage takes place (excess pore pressures are kept equal to zero), then a Poisson's ratio value of 0.35 may be reasonable. The difference between the two calculated moduli is the difference between the undrained modulus and the drained modulus. Note that the shear modulus remains theoretically constant when the drainage varies. Note also that the Poisson's ratio can be larger than 0.5 if the soil dilates during shear associated with compression.

3.5 MODULI FOR VARIOUS FIELDS OF APPLICATION

The modulus is useful in many fields of Geo-technical Engineering. It is clear by now that the modulus required for one field may be significantly different from the modulus for another field.

3.5.1 In the case of shallow foundations.

The mean stress level applied under the foundation is often between 100 and 200 kPa. The normal strain level in the vertical direction is about 0.01 or less and is typically associated with a movement of about 25 mm. The rate of loading is extremely slow because that strain occurs first at the construction rate and then the load is sustained over many years. The number of cycles is one unless cycles due to seasonal variations or other cyclic loading (such as compressor foundations) are included. Example values of the modulus in this case are 10,000 to 20,000 kPa.

3.5.2 In the case of deep foundations.

The mean stress level varies because the side friction on the piles occurs over a range of depth, while the point resistance occurs at a relatively large depth. The strain level at the pile point is usually smaller than in shallow foundations because a percentage of the load dissipates in friction before getting to the pile point. The strain rate is similar to the case of shallow foundations with rates associated with months of construction and years of sustained loads. High strain rates do occur however in the case of earthquake or wave loading. Cycles can be a major issue for earthquake loading of buildings and bridges or for wave loading of offshore structures. Because deep foundations are used in very different types of soils and for very different types of loading, the moduli vary over a much wider range of values than for shallow foundations.

3.5.3 In the case of slope stability and retaining structures

Movements are associated with the deformation of the soil mass essentially under its own weight. Therefore the stress level corresponds to gravity induced stresses. The strains are usually very small and the strain rate is again associated with the rate of construction at first and the long-term deformation rate during the life of the slope or of the retaining structure. Cycles may occur due to earthquakes or other cyclic phenomena. For properly designed slopes and retaining structures, the moduli tend to be higher than in foundation engineering because the strain levels tend to be smaller.

3.5.4 In the case of pavements.

The mean stress level in the sub-grade is relatively low. The pressure applied to the pavement is of the order of 200 kPa for car tires, 500 kPa for truck tires, and 1700 kPa for airplane tires. However, the vertical stress at the top of the sub-grade under a properly designed pavement may be only one tenth of the tire pressure applied at the surface of the pavement. The strain level is very low because the purpose of the pavement is to limit long-term deformations to movements measured in millimeters if not in tenths of millimeters. Typical strain levels are 0.001 or less at the top of the sub-grade. The rate of loading is very high and associated with the passing of a traveling vehicle. The loading time is of the order of milliseconds for a car at 100 km/h but is measured in hours for an airplane parked at the gate. The number of cycles is tied to the number of vehicles traveling on the pavement during the life of the pavement. This number varies drastically from less than a million of vehicle cycles for small roads to tens of millions for busy interstates. Typical modulus values range from 20,000 kPa to 150,000 kPa.

3.6 Effective stress analysis of strength and small strains behaviour of sand

Attention is focused on the influence of water content of the maximum elastic modulus, the decay of modulus with strain, the effect of the confining stress. The effective stress concept by a recent theoretical analysis by Coussy and Dangla (2002).

Despite the wide use of compacted materials, their stress-strain pre-failure behaviour, particularly under unsaturated conditions, has not yet been deeply investigated. There is still a lack of experimental data concerning unsaturated soils in the small strains domain and a need for a more rational analysis based of the general framework of soil mechanical behaviour.

3.6.1 Influence of water content on maximum modulus

For the unsaturated specimens, the variations of normalized modulus versus total vertical stress approximately follow a power law, with an exponent n=0.35-0.40. On the other hand, the lines for the dry and quasi- saturated specimens are nearly superimposed, with stiffer slope. For the same void ratio and under the same vertical stress, there is a general increase in the modulus when the water content decreases, as long as the water content is strictly larger than 0.

3.6.2 Influence of confining stress and water content on the elastic constants

Confining stress does not seem to have any influence on the reduction of elastic modulus. Initial water content of the specimen does not appear to influence the decay curve of the elastic constant. Initial water content of the specimens does not appear to influence the decay curve of the Young modulus.

3.6.3 Influence of loading rate on maximum modulus

It shows that only a slight increase in the modulus with the loading rate. Several investigations have show that the loading rate has little effect on the maximum modulus of stiff materials, but that this influence increase in the case of silty sands and clay.

3.7 stiffness degradation and shear strength of silty sands

Soil behaves non-linear from very early loading stage. When granular soil contain a certain amount of fines the degree of no linearity also changes, as stiffness and strength characteristics vary with fines content.

3.7.1 Degradation of elastic modulus for silty sands

In general the presence of no plastic fines in granular soils results in higher dilatancy because of increasing interlocking of particles, with fines wedging themselves between larger particles. This is true up to a certain percentage of fines. The upper limit of silt content up to which increasing dilatancy is on the order of 20%. For fines contents greater than this limit, the behaviour of the soil would be dominated by the fines rather than by the larger particles.

Soil parameter can be classified as either intrinsic or extrinsic or state variables. Intrinsic variables do not change with soil state and are only a function of soil particle mineralogy, shape, and size distribution. These variables include the friction angle at critical state; maximum and minimum void ratio and specific gravity. If the amount of fines in a soil changes, the values of the intrinsic soil variables also change. State variables on the other hands, are dependent on the soil state.

CHAPTER-4

NUMERICAL ANALYSIS OF SOIL-FOUNDATION

CHAPTER-4

NUMERICAL ANALYSIS OF SOIL-FOUNDATION

4.1 GENERAL

The settlement of foundations under working load conditions is an important design consideration. Well-designed foundations induce stress-strain states in the soil that are neither in the linear elastic range nor in the range usually associated with perfect plasticity. Thus, in order to accurately predict working settlements, analysis that is more realistic than simple elastic analyses are required. The settlements of footings in sand are often estimated based on the results of in-situ tests, particularly the standard penetration test (SPT) and the cone penetration test (CPT), we analyze the load-settlement response of vertically loaded footings placed in sands using both the finite element method with a nonlinear stress-strain model. Calculations are made for both normally consolidated sands with various relative densities. Based on these analyses, we propose a procedure for the estimation of footing settlement in sands.

Footings are often used to support structures at sites where the soils near the surface are sufficiently strong to serve as a bearing layer. There are two key calculations required for footing design: assessment of ultimate bearing capacity and estimation of settlement under working loads. The ultimate bearing capacity is usually calculated using the bearing capacity equation, which originated from the pioneering work by Prandtl (1921) and Reissner (1924). Although N_c and N_q could be rigorously determined for strip footings placed on the soil surface, values of *N* and shape and depth factors could not. Most efforts since then have been devoted to determining methods to estimate the values of these factors.

Settlements are considered tolerable if they do not impair the functionality or serviceability of foundations or the supported superstructures under the design loads. Most methods used in practice to estimate the settlements of footings in sand are based on the linear elastic approach. Results of this approach are strongly dependent on the reasonable selection of a representative set of elastic parameters. This is a difficult judgment call that can be made easier if a number of load test results or other forms of well-documented preexisting information are available. In fact, well-designed foundations induce stress-strain states in the soil that are neither in the linear elastic range nor in the range usually associated with perfect plasticity. This requires the consideration of the nonlinear stress-strain relationship of soils for accurate estimation of settlements under working loads. Estimated settlement should be no greater than the maximum tolerable settlement chosen on the basis of type and details of the superstructure, as well as its purpose and architectural finishing's. Footing settlement in sand deposits is often estimated using the results of *in-situ* tests, mainly the standard penetration test (SPT) and the cone penetration test (CPT).

In this approach, the soil stiffness is estimated from measured penetration resistance in terms of either the SPT blow count *N* or the cone resistance q_c. While the SPT is still widely used, the CPT has become popular for a number of reasons, including the much lower level of uncertainty associated with q_c than with *N*. There have been several methods proposed for the use of CPT results in footing settlement calculations. Most of these methods estimate representative soil compressibility or elastic moduli from cone resistance *qc*. Schmertmann's method is one of the most popular methods, in part due to its simplicity. It is based on linear elasticity and the observation that settlement results from strains that initially increase with depth (measured from the base of the footing), but then peak and drop toward zero. The soil elasticity modulus at different depths for use in calculations is determined by multiplying *qc* by an empirical factor. Here vertically loaded footings in sand are analyzed for various soil conditions using the finite element method. The analyses in this study take into account the non-linearity of sands, and the effects of footing size and relative density. The load-settlement responses obtained from these analyses are compared with those from Schmertmann's method and from field load tests. Based on these results, a new approach for estimating the settlement of footings in sand is presented. It is based on the elastic framework of Schmertmann, but allows more realistic accounting of factors such as the settlement level, footing size and relative density on the estimation of soil stiffness from CPT cone resistance.

4.2 METHODS OF SETTLEMENT ESTIMATION FOR FOOTINGS BEARING ON SAND

4.2.1 SPT-based methods

There are several methods available for the calculation of footing settlements using SPT results. Most of these methods are based on elasticity, and thus focus on determination of soil compressibility, with consideration of footing size. Meyerhof suggested the following relationship for the settlement of spread footings on sand:

$$s = \frac{0.203 L_R}{N_{45}} \left(\frac{q_b}{p_A}\right) \qquad \text{for} \quad B \leq 1.2 L_R$$

$$s = \frac{0.305 L_R}{N_{45}} \left(\frac{q_b}{p_A}\right) \left(\frac{B}{B + L_R/3.28}\right)^2 \qquad \text{for} \quad B > 1.2 L_R$$

Where s = footing settlement; q_b = unit load at base of footing; N_{45} = SPT blow count corrected for an energy ratio of 45%, following Skempton [42]; B = footing width; L_R = reference length = 1 m = 3.28 ft = 39.37 in; p_A = reference pressure = 100 kPa. In equations the SPT N_{45} values are not corrected for water table, vertical effective stress, or other factors. Peck and Bazaraa proposed the following relationship, a modification of above equation for estimating the settlements of footings on sand:

$$s = C_w C_d \left(\frac{0.051 L_R}{N_B}\right) \left(\frac{q_b}{p_A}\right) \left(\frac{2B}{B + L_R/3.28}\right)^2$$

$$N_B = \left(\frac{4N_{45}}{1 + 4\sigma'_v/p_A}\right) \frac{\sigma'_v}{p_A} \qquad \text{for} \quad \sigma'_v \leq 0.75 p_A$$

$$N_B = \left(\frac{4N_{45}}{3.25 + 4\sigma'_v/p_A}\right) \frac{\sigma'_v}{p_A} \qquad \text{for} \quad \sigma'_v > 0.75 p_A$$

Where C_w = groundwater correction factor; C_d = depth correction factor; q_b = unit load at base of footing; σ_v = vertical effective stress; N_{45} = measured SPT N values corrected for a 45% SPT energy ratio; N_B = stress-normalized SPT N value; B = footing width; L_R = reference length = 1 m = 3.28 ft = 39.37 in; p_A = reference pressure = 100 kPa. Equation is based on the assumption that settlements predicted by the Terzaghi and Peck [45] correlation produce excessively conservative results (i. e., excessively large settlements). Another method for estimating settlements of footings in sand or gravel was proposed by Burland and Burbidge [9]:

$$s = f_s \cdot f_I \cdot f_t \cdot I_c \left(q_b - \frac{2}{3}\sigma'_{v,p}\right)\left(\frac{100}{p_A}\right)\left(\frac{B}{L_R}\right)^{0.7}$$

Where s = footing settlement; f_s = shape factor; f_I = depth factor for the sand or gravel layer; ft = time factor; qb = unit load at footing base; σ_{vp} = maximum previous vertical stress; B = footing width; I_c = compressibility index; L_R = reference length = 1 m = 3.28 ft = 39.37 in; p_A = reference pressure = 100 kPa. Burland and Burbidge [9] presented values of the compressibility index I_c as a function of SPT blow count N. The use of σ_{vp} in equation allows for the effect of over consolidation at the footing base due to the excavation.

4.2.2 CPT-based methods

One of the most common methods for the calculation of footing settlement using CPT results are Schmertmann's method [40, 41]. In this method, the soil profile underneath the footing is divided into several sub-layers. For each sub-layer, the soil stiffness is determined based on the cone resistance q_c. As shown in Figure 4.1, the influence zone for settlement computations extends down to 2B for square footings and 4B for strip footings. The extent of the influence zone and the values of the influence factor are based on deformation profiles beneath footings obtained from analysis and experiments [40, 41]. Physically, stiffness increases with depth, and the stresses induced by the applied load decrease with depth, so the contribution of deeper layers to settlement should be less than that of shallower layers. This is observed to be true, except for the soil immediately below the footing base. The calculation of footing settlement in sands by Schmertmann's method is done using the following equations:

$$s = C_1 \cdot C_2 \cdot \left(q_b - \sigma'_v\big|_d\right) \cdot \sum \left(\frac{I_z \cdot \Delta z_i}{E_i}\right)$$

$$C_1 = 1 - 0.5\left(\frac{\sigma'_v\big|_d}{q_b - \sigma'_v\big|_d}\right)$$

$$C_2 = 1 + 0.2\log\left(\frac{t}{0.1 \cdot t_R}\right)$$

Where s = settlement caused by applied load; C_1 and C_2 = depth and time factors; q_b = unit load on footing base; $\sigma_{v/d}$ = vertical effective stress at footing base level; I_z = depth influence factor; Δzi = thickness of each sub-layer; E_i = representative elastic modulus of each sub-layer; t = time; t_R = reference time = 1 year = 365 days.

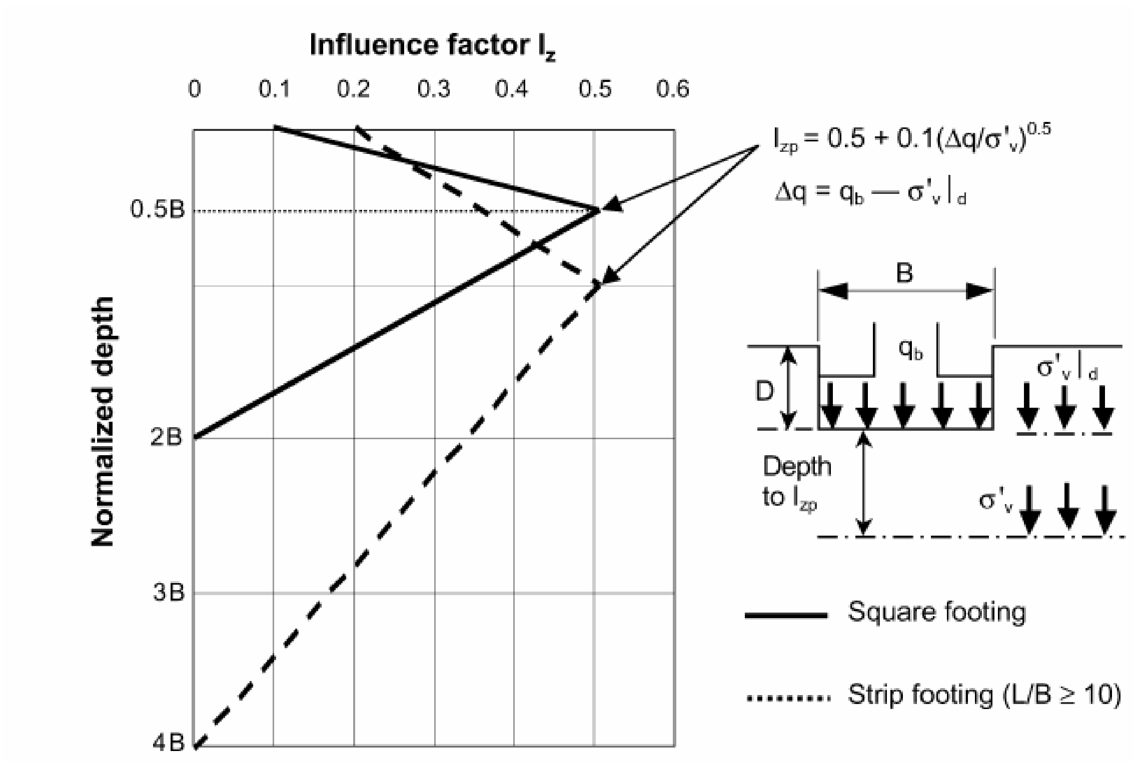


Fig. 4.1 Influence factor I_z vs. Depth

In Schmertmann's method, the elastic modulus E_i of each individual sub-layer is obtained from the representative cone resistance q_{ci} for that layer. The correlations between the elastic modulus E_i and the cone resistance q_{ci} that are most often used are those of Schmertmann et al. [41] and Robertson and Campanella [33], which may be summarized as

$$E_i = \begin{cases} 2.5q_{ci} & \text{for young normally consolidated silica sand} \\ 3.5q_{ci} & \text{for aged normally consolidated silica sand} \\ 6.0q_{ci} & \text{for overconsolidated silica sand .} \end{cases}$$

A different approach for footing settlement estimation using CPT results can be found in Berardi et al. [3]. In this approach, footing settlements in sands are calculated from the following equation from elasticity theory:

$$s = I_s \cdot \frac{q_b \cdot B}{E'}$$

Where s = footing settlement; I_s = influence factor depending on the shape and rigidity of the foundation; q_b = unit load on footing base; B = footing width; E_0 = drained Young's modulus, obtained from the cone resistance q_c as a function of the relative density D_R and the strain level. The procedure proposed by Berardi et al. [3] using (11) is iterative, because the strain level is one of the variables in the calculations. For the selection of q_c, Berardi et al. [3] suggested taking the representative q_c at a depth equal to B=2 below the footing base.

4.3. NUMERICAL SIMULATION OF FOOTING LOAD TESTS

4. 3.1 Stress-strain relationship for numerical analysis

For successful analyses to be performed, the behavior of soil should be modeled realistically. It is usually observed that soil shows a nonlinear stress-strain behavior from the early stages of loading. The hyperbolic family of soil models has been often used to describe the nonlinear soil behavior observed from the early stages of loading. Although hyperbolic soil models may not satisfy energy conservatism, they have been applied extensively and successfully in various nonlinear soil behavior problems. This is mainly due to their suitability for numerical implementation and to the clear relationship between soil parameters and observed stress-strain curves.

Based on the observed degradation of elastic modulus in sands, Lee and Salgado [23, 24] suggested the following modification to the conventional hyperbolic soil model for a general stress state:

$$\frac{G}{G_o} = \left[1 - f\left(\frac{\sqrt{J_2} - \sqrt{J_{2o}}}{\sqrt{J_{2\max} - \sqrt{J_{2o}}}}\right)^g\right]\left(\frac{I_1}{I_{1o}}\right)^{n_g} \qquad (12)$$

Where G = secant shear modulus; G_o = initial shear modulus; J_2 = second invariant of the deviatoric stress tensor; J_2; J_{2o} and $J_{2\max}$ represent the current, initial, and maximum shear stress in three dimensions; I_1 and I_{1o} are the first invariants of the stress tensor at the current and initial states; f, g, and n_g =material parameters. Equation (12) represents the degradation of shear modulus from its initial maximum value G_o as a function of the shear J_2 and confining stress I_1 levels. Lee and Salgado also proposed values of f and g as a function of the relative density D_R

Sand usually behaves as a linear elastic material with shear modulus G_o for shear strains up to approximately 10−5, after which the stress-strain relationship is strongly nonlinear. In this study, the following empirical equation, based on the work of Hardin and Black [17], was used to estimate the initial maximum shear modulus of sand:

$$G_o = C_g \frac{(e_g - e_o)}{1 + e_o} p_A^{1-n_g} \left(\sigma'_m\right)^{n_g}$$

Where C_g, n_g, and e_g = intrinsic material variables; e_o = initial void ratio; p_A = reference pressure = 100 kPa; and σ'_m = initial mean effective stress in the same unit as p_A. It should be noticed that the parameter n_g is the same as appears in both equations; n_g represents the dependence of the shear modulus on confinement.

The elastic stress-strain relationship may be expressed by two constants; the bulk modulus *K* and the shear modulus *G* are often used. The full description of nonlinear elastic response requires proper representation of the variation of *K* as well as *G* with changes in stress state. The bulk modulus depends mainly on the amount of confining stress. Based on the discussion of the *K-G* model by Naylor et al., the tangent bulk modulus K_t can be represented by the following equation:

$$K_t = D_s \cdot \left(\sigma'_m\right)^{n_k} \left(p_A\right)^{(1-n_k)}$$

Where p_A = reference pressure = 100 kPa; σ'_m = mean effective stress in the same units as p_A; *s* = material constant that can be calculated from the initial values of bulk modulus and confining stress; and n_k can be taken as 0.5.

It has been known that hyperbolic soil models with varying *G* and *K* cannot describe adequately the soil behavior near failure. In order to describe failure and post-failure soil, response, the rucker-Prager failure criterion was adopted. The rucker–Prager failure criterion is given by:

$$F = \sqrt{J_2} - (\alpha I_1 + \kappa) = 0$$

where

$$\alpha = \frac{2 \sin \phi_p}{\sqrt{3}\left(3 - \sin \phi_p\right)}$$

$$\kappa = \frac{6c \cdot \cos \phi_p}{\sqrt{3}\left(3 - \sin \phi_p\right)}$$

Where *c* = cohesion and ϕ_p = peak friction angle. The cohesive intercept *c* is zero in sands. The peak friction angle in sands can be expressed in terms of the friction angle at the critical state and the dilatancy angle [4, 39]. The critical-state friction angle for a given soil is independent of stress state and density. The dilatancy angle is a function of density and confinement, increasing with increases in density and decreasing with increases in confinement. Consequently, the envelope of the failure surface is nonlinear. Bolton [4] proposed the following equation to estimate the peak friction angle in sand:

$$\phi_p = \phi_c + 0.8 \cdot \psi_p$$

Where ϕ_p = peak friction angle, ϕc = critical state friction angle and p = peak dilatancy angle given by

$$\psi_p = \begin{cases} 6.25 I_R & \text{for plane – strain conditions} \\ 3.75 I_R & \text{for triaxial conditions} \end{cases}$$

The dilatancy index I_R that appears in (19) is given by

$$I_R = I_D \left[Q + \ln \left(\frac{p_A}{100 p'_P} \right) \right] - 1$$

Where I_D = relative density (as a number between 0 and 1); p_A = reference pressure = 100 kPa; p'_P = mean effective stress at peak strength in the same units as p_A; and Q = intrinsic soil variable, approximately equal to 10 for silica sands. Equations (18) through (20) were used to define the nonlinear =rucker–Prager failure surface.

4.3.2 Numerical modeling of footing load tests

Five footing load tests were performed at the Texas A&M University Riverside campus for the spread footing prediction symposium organized for the Settlement. Five square footings were tested: two 3 x 3 m footings (south and north sides), one 2:5 x 2:5 m footing, one 1:5 x 1:5 m footing, and one 1 x 1 m footing. The test site consists predominantly of sand down to a depth of 11 m. beneath this sand layer; there is a very stiff clay deposit extending down to a depth of approximately 33 m. The water table was observed at a depth of 4.9 m from the ground surface. Fig. 4.2 shows the sub-soil profile for the test site. Grain size analysis showed the amount of fines content to vary with depth, from 2 to 8% to 5 to 30% fines contents down to depths of 3 and 9 m, respectively.

All five footing load tests were modeled numerically using the finite element method. These analyses aimed at validating in a general way our numerical analysis by comparison with existing

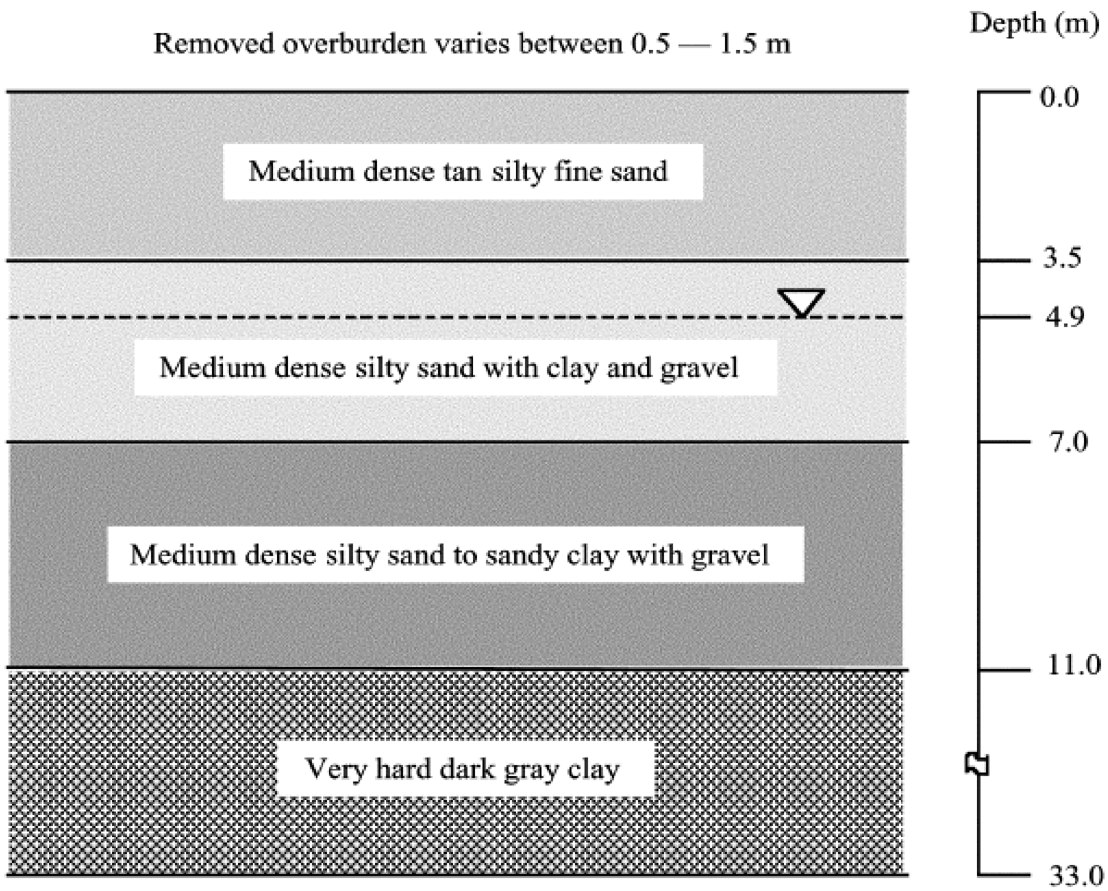


Fig.4.2 Sub-soil profiles

settlement observations. The commercial finite element program PLAXIS was used to model the footings, with a subroutine specifically written for the nonlinear elastic-plastic stress-strain model previously described. In the finite element analyses, the soil layers below the foundation level were modeled based on the soil profile (shown in Fig.4.2) determined from extensive site characterization done.

The bottom boundaries of the finite element models were located at a depth of 17 m. The lateral boundaries were located at a distance of 14 m (for the 1-m and 1.5-m footings), and 18 m (for the 2.5-m and 3-m footings) from the axis of the footings. The footings were modeled as circular footings with diameters equal to 1.13 m, 1.69 m, 2.80 m, and 3.40 m, with areas equivalent to those of the 1-m, 1.5-m, 2.5-m, and 3-m square footings, respectively. Differences in stresses at the same depths due to the use of circular rather than square footings were less than 2%, based on linear elastic calculations. 15-noded axi-symmetric elements were used in the finite element meshes to model the footings. The initial stress states for the analyses were set as geostatic, based on the soil profile shown in Fig.4.2 with unit weights estimated from the information obtained from the site characterization. The values of initial shear modulus G_o for the nonlinear stress-strain model were based on the results of the in-situ cross-hole test that was performed near the footings. The measured shear wave velocities and the values of the initial shear modulus G_o with depth are shown in Table 4.1.

A factor that is expected to have affected significantly the load-settlement response at this

TABLE 1

Small-Strain Shear Modulus from Cross-Hole Test (after Gibbens and Briaud [16])

Depth (m)	Time difference (ms)	Corrected separation (m)	Shear wave velocity (m/s)	G_o (MPa)
2	10	2.398	240	104
4	8	2.397	300	162
6	8.5	2.391	281	142
8	12	2.383	199	71
10	10	2.380	238	102

site is the over-consolidation ratio. As can be seen in Fig. 4.2, approximately 1.5 m of soil was removed before the load tests were conducted. Additionally, the geologic condition of the site, as a coastal plain, suggests considerable overconsolidation caused by desiccation of the fines. According to Lee and Salgado [23, 25], the elastic modulus degradation parameters *f* and *g* of normally consolidated sand are typically within the 0.94 to 0.98 and 0.17 to 0.3 ranges, respectively, depending on the relative density. Following the observations of Teachavorasinskun et al. the rate of elastic modulus degradation of over-consolidated sand is much lower than that of normally consolidated sand. In order to reflect the over-consolidated stress state of the site, *f* and *g* values equal to 0.96 and 0.6, respectively, were used. These values of *f* and *g* lead to load-settlement responses that are stiffer than those of normally consolidated sands.

4.4 PREDICTED AND MEASURED LOAD-SETTLEMENT RESPONSE OF FOOTINGS

It is instructive to compare the results of our finite element analyses with those obtained using the method of Schmertmann. The triangular influence area down to two times the footing width, as shown in Figure 4.1, was divided into 10 different layers with equal thickness. The elastic modulus E_i for each sub-layer was obtained using the relationship $E_i = 6{:}0 \text{x} q_{ci}$, appropriate for over-consolidated stress states according to (10). The cone resistance values were obtained from the cone penetration tests performed at the locations closest to each of the footing load tests.

Fig.4.3 shows the measured and predicted load-settlement response for each footing. As can be seen in the figure, the results predicted by the nonlinear finite element analyses are consistent with the measured results. Schmertmann's method is consistent with observed results for settlements of the order of 2 cm. The significant load under prediction resulting from application of Schmertmann's method to the 3-m footing (south side) was due to the very low cone resistance at a depth equal to about 3 m (= 10 ft) observed in the CPT test used in the analysis, which, we speculate, is not reflective of the true soil condition underneath the footing.

4.5. LOAD-SETTLEMENT RESPONSE OF VERTICALLY LOADED FOOTINGS

4.5.1. Load-settlement response for various soil states

Most existing procedures for the calculation of footing settlements in sands using *in-situ* test results consider footing size effects either implicitly or explicitly. For sands, the relative density is the most important factor controlling mechanical behavior. In most conventional elastic methods, the soil stiffness for footing settlement calculations is obtained from the SPT; N values or the cone resistance q_c without consideration of the relative density. This is because it was wrongly believed in the past that the density of sands was directly reflected in the SPT N values or q_c.

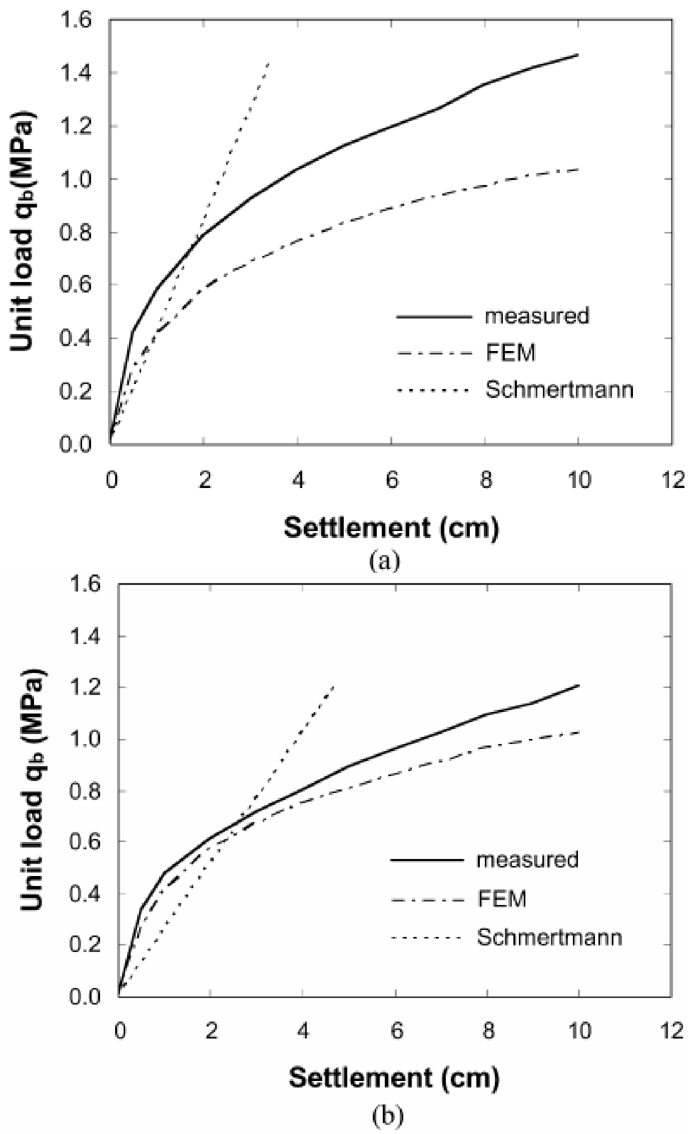
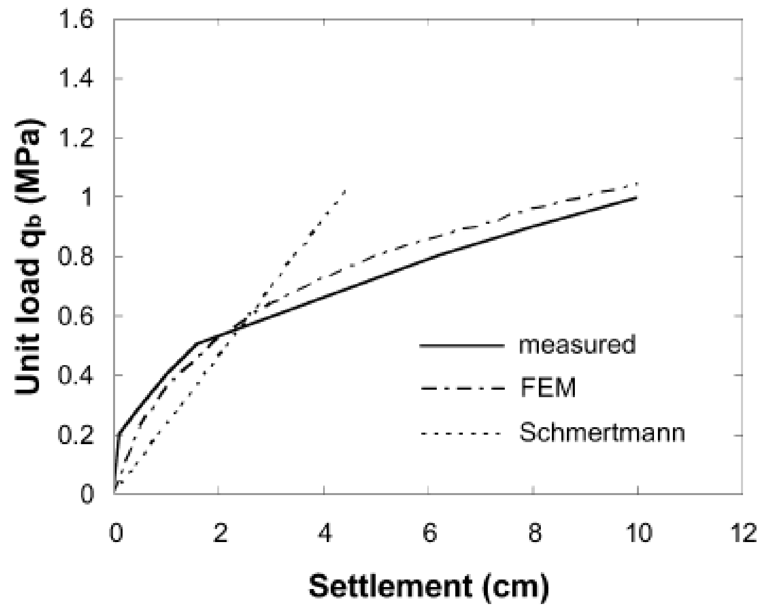


FIG.4.3 Measured and predicted load-settlement response for (a) 1-m footing and (b) 1.5-m footing.

When they were normalized with respect to vertical effective stress. It has been observed that the ratio of soil stiffness to penetration resistance of sands also changes with the relative density. This suggests that relative density should be a factor, together with footing size and stress-strain non-linearity, in the calculation of footing settlement in sands. In order to investigate the load-settlement response of footings bearing on sand, circular footings of different diameters (1, 2, 3 m), with stiffness much greater than that of soil, were modeled as resting on the soil surface with no surcharge. The soil was assumed to be normally consolidated sand. Table 2 shows the basic properties of sand, as given by Ghionna et al. [15]. Comparisons were made for four different relative densities (30, 50, 70, and 90%) The construction of the finite element models was done in a manner similar as for the Texas A&M footings. The lateral and bottom boundaries of the finite element mesh were located at 12 m horizontally and 15 m vertically from the center of the



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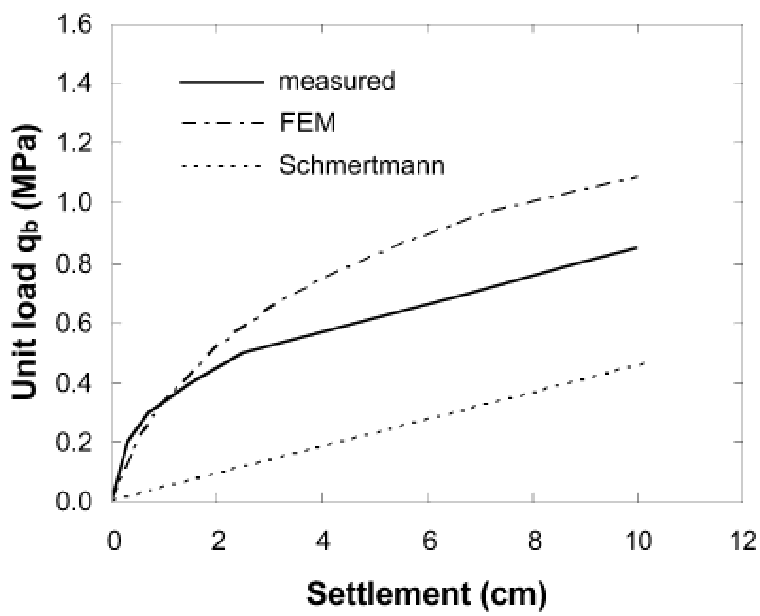
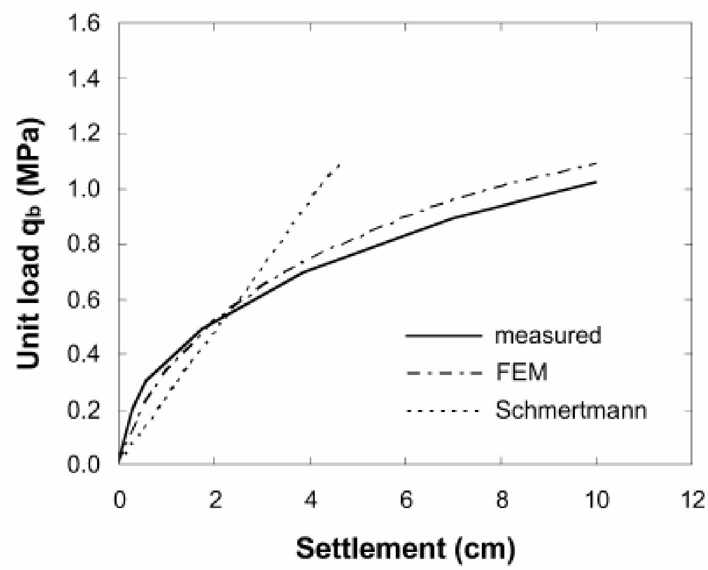


(d)

Fig. 4.3 Measured and predicted load-settlement response for (c) 2.5-m footing and (d) 3-m footing (south side).

footing base, respectively. Based on analyses with meshes of various sizes, it was found that the mesh size used in this study, extending laterally to more than 4 times the footing diameter and vertically to more than 5 times the footing diameter, is large enough to eliminate boundary effects. Interface elements were also used between the footing base and soil. Fig. 4.4 shows the finite element meshes for the 1-m, 2-m, and 3-m footings, respectively.

Fig. 4.5 shows results from the finite element analyses in terms of unit load at the footing base versus normalized settlement. The settlement was normalized with respect to the footing diameter. For the normalized load-settlement curves of footings on sand, Briaud and Jeanjean [6] suggested the use of a unique load-settlement curve, without consideration of the effects of footing size. The results obtained in this study, however, indicate that the effect of footing size on the



(e)

Fig. 4.3 Measured and predicted load-settlement response for (e) 3-m footing (north side).

TABLE 2

Basic Properties of Ticino Sand (after Ghionna et al. [15])

D_{10} (mm)	D_{50} (mm)	G_s^a	U^b	ϕ_c	e_{max}	e_{min}	γ_{max} (KN/m^3)	γ_{min} (KN/m^3)	C_g	n_g	e_g
0.36	0.54	2.623	1.5	34.8°	0.922	0.573	16.68	13.65	647	0.44	2.27

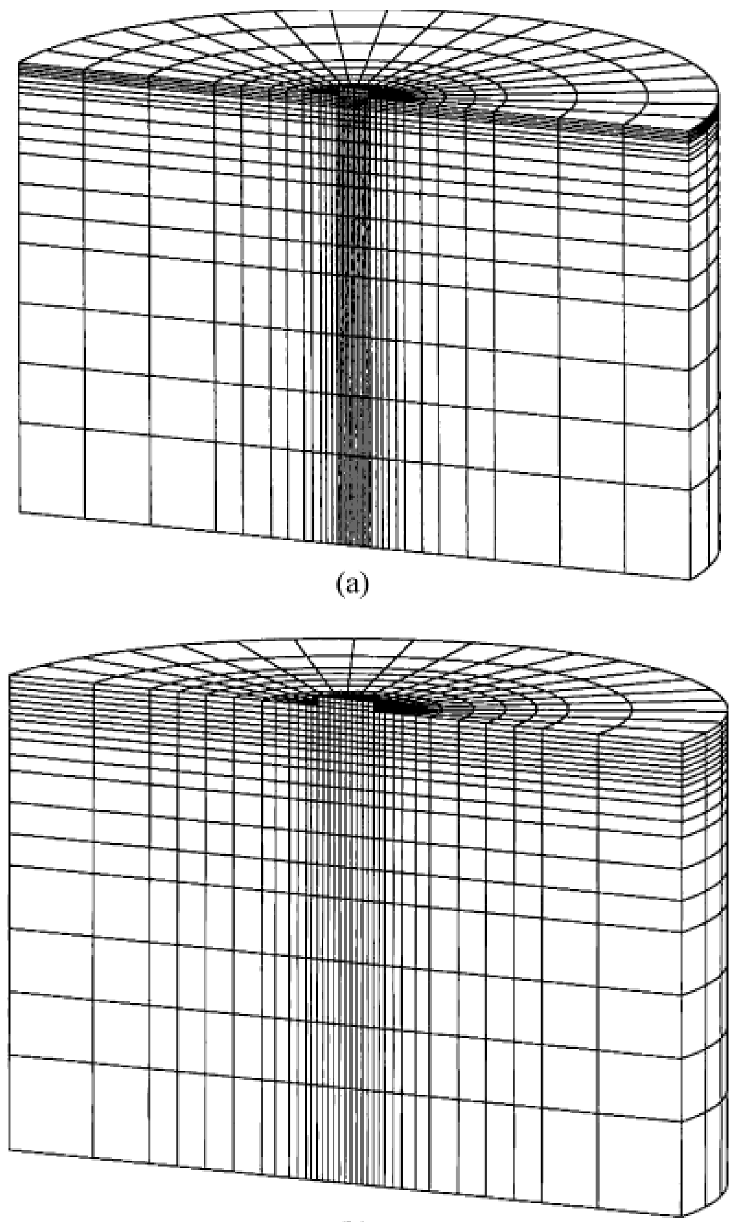
$^a G_s$ = specific gravity
$^b U$ = coefficient of uniformity

unit load for a given relative settlement cannot be neglected. As shown in Fig. 4.5, the unit load associated with a given relative settlement increases as the footing size increases, irrespective of the relative density. For D_R = 30%, the unit loads at the base of the 3-m footing are 0.656 Mpa and 0.991 MPa for relative settlements equal to 5% and 10%, respectively. These pressures are approximately twice as large as those observed for the 1-m footing. Similar results were found for other values of relative density. The effect of footing size on base resistance is thought to derive from the characteristics of granular soil deposits, for which soil stiffness is a function of confinement, increasing with depth. This increase of elastic modulus, as footing size increases, and thus the depth of the influence zone increases, results in a stiffer load-settlement response.

4.5.2. Footing load capacity at common design settlements

The tolerable settlement for shallow foundations has often been assumed to be 25 mm (or 1 in). Fig.4.6 shows the unit loads calculated from the finite element analyses for a footing settlement equal to 25 mm as a function of footing size and relative density. Because the unit loads in Fig. 4.6 correspond to the same absolute value of settlement, not the same relative settlement (S/B), smaller footings show higher values of unit loads.

As can be seen in Fig. 4.6(a), the differences between unit loads for footings with different diameters become more pronounced as the relative density increases. At D_R =90%, the unit loads at a settlement of 25 mm are 0.628 MPa, 0.493 MPa, and 0.456 MPa for the 1-m, 2-m,



(a)

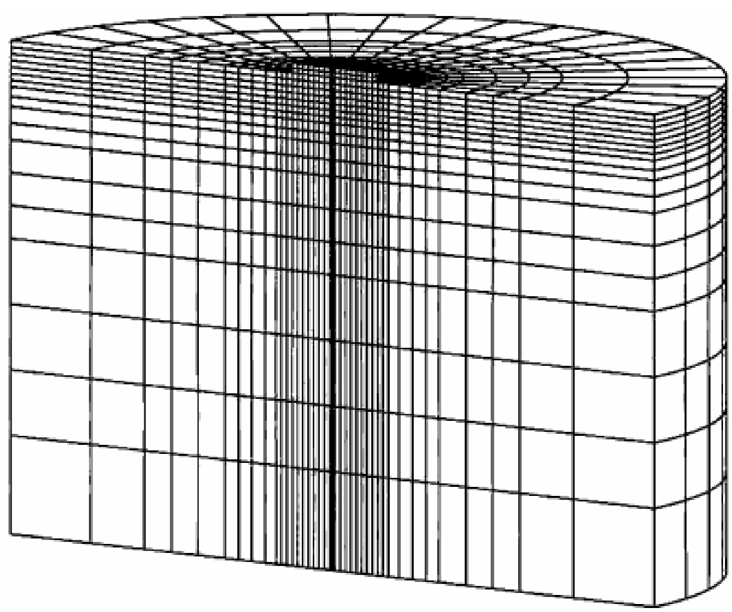
(b)

Fig. 4.4 Finite element models for (a) 1-m footing and (b) 2-m footing.

and 3-m footings, while at D_R = 30% the differences are small to negligible. These results compare favorably with the results obtained by Terzaghi and Peck [45] from actual footing load tests. Figure 4.6(b) shows the observed allowable soil pressure as a function of the footing size and different soil densities (as reflected in SPT blow counts) by Terzaghi and Peck [45]. Comparing Figures 6(a) and 6(b), it can be seen that both experimental and numerical results show the unit load to depend on footing size for high more than for low relative densities.

4.6. USE OF CONE RESISTANCE IN FOOTING DESIGN

Schmertmann's method is a pseudo-elastic method based on an assumed value of elasticity modulus. Because it is based on linear elasticity, the load-settlement response obtained by Schmertmann's method leads to unconservative results if the load or settlement level increases



(c)

Fig. 4.4 Finite element models for (c) 3-m footing.

beyond the range to which the assumed elasticity modulus applies. As discussed earlier, the elasticity modulus in Schmertmann's method is determined from the following relationship:

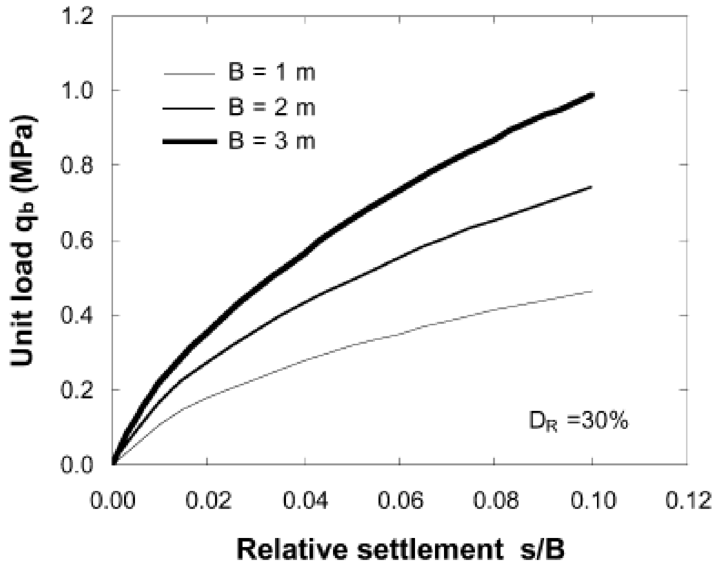
$$E_i = \varphi_i \cdot q_{ci}$$

in which E_i =elastic modulus for sub-layer i; φi = ratio of elastic modulus to cone resistance for sub-layer i; q_{ci} = representative cone resistance for sub-layer i. Fixed values of the parameter ' were given in (10) for different soil conditions. However, flexibility in setting tolerable settlements is desirable in design. For the same footing and soil conditions, setting different tolerable settlements implies different degrees of degradation of soil stiffness, and consequently different values of φi. The degradation of soil stiffness may also be different for different soil densities for the same tolerable settlement. The analysis and design of footings bearing on sand therefore require proper consideration of this stress-strain non-linearity of soils.

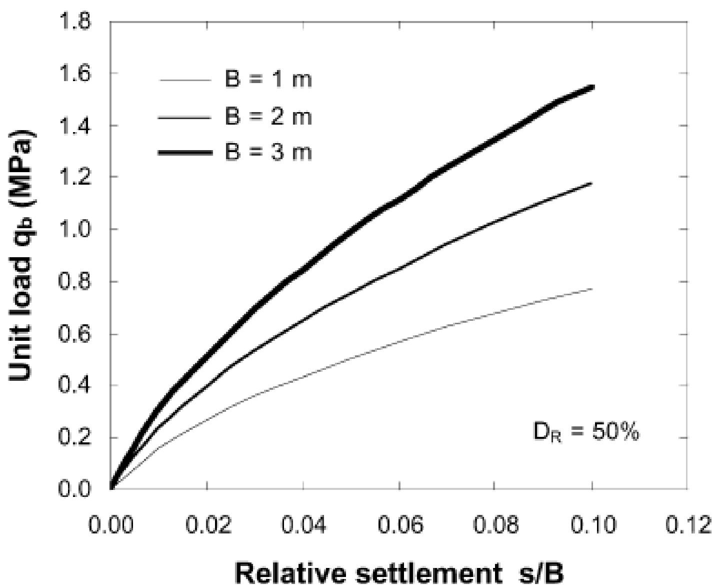
4.6.1. Isolated footings

Fig.-4.7 shows the load-settlement curves for the 1-m and 3-m circular footings and sand relative densities equal to 30 and 90%, obtained from both the nonlinear finite element analysis and Schmertmann's method. For Schmertmann's method, the triangular influence area down to two times the footing diameter below the footing base was divided into four different layers. Four layers are found to be sufficient for calculations involving uniform soil deposits. The representative elastic modulus of each layer was obtained based on the relationship E_i= 2.5xq_{ci} for normally consolidated sand deposits. The cone resistance q_c in each sub-layer was calculated using the cone penetration resistance analysis.

As can be seen in Fig.-4.7, the finite element analyses with nonlinear elastic-plastic model produce, for both the 1-m and 3-m footings, lower values of unit load for the same settlement level than those resulting from Schmertmann's method. The difference between the unit load calculated using the nonlinear finite element analysis and Schmertmann's method becomes more pronounced as the relative density increases. At the relative density of D_R = 30%, the load-settlement curves obtained from both approaches appear to be in reasonable agreement for the settlement range of interest in practice. At the relative density of D_R =90%, the nonlinear finite element analysis produces results that are conservative with respect to those of Schmertmann's method. These results indicate that different values of φi for use in (21) are needed for different settlement levels and relative densities if the pseudo-elastic approach is to produce realistic estimates of the footing settlement. As discussed earlier, the usual design criterion for footings on sand is that the settlement be no more than 25 mm (= 1 in). Fig. 4.8 shows the values of the ratio φ_{25} of cone resistance q_c to

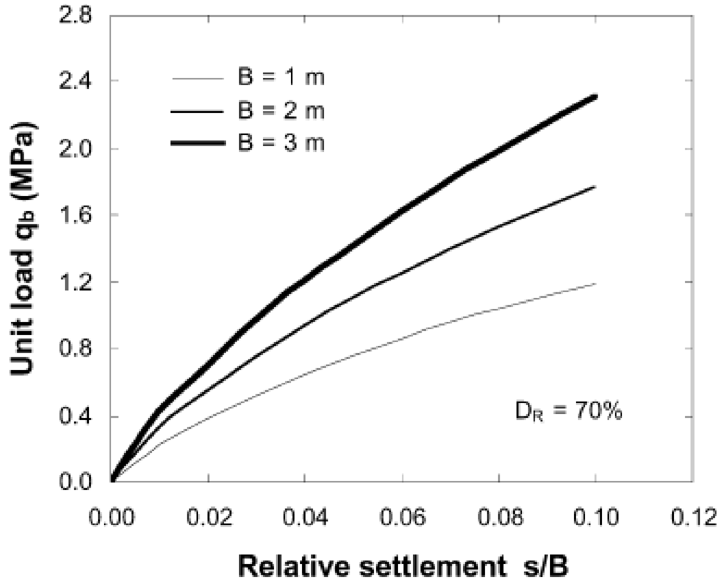


(a)

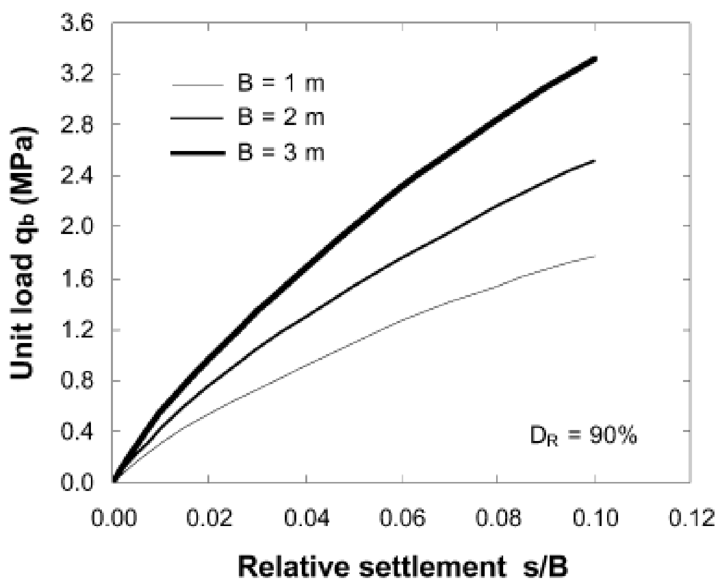


(b)

Fig. 4.5 Normalized load-settlement curves for (a) D_R = 30% and (b) D_R = 50%.



(c)



(d)

Fig. 4.5 Normalized load-settlement curves for (c) D_R =70% and (d) D_R =90%.

elastic modulus *E*, as a function of relative density, for which Schmertmann's method gives the same load for a settlement of 25 mm as the nonlinear finite element analysis. It is observed that the value of φ_{25} varies according to both the footing size and relative density. The larger the footing size, the higher the value of $\varphi_{25.}$ The influence of the relative density was even more substantial. At D_R =30%, the values of φ_{25} are higher than the 2.5 values originally proposed for normally consolidated sand. As the relative density increases, the value of φ_{25} decreases significantly, lying in the 1.1 to 1.7 range for D_R = 90%. It is interesting to note that φ_{25} = 2:5 is found to be a suitable value for a footing in the 2 to 3 m range bearing on a sand with D_R =50%, conditions that might be thought of as being "average."

The values of φ were further evaluated for a variety of settlements. Figure 4.9 shows different values of φ for different settlement levels, ranging from 2 to 10 cm. This settlement range is believed to cover most situations of interest in practice. As expected, $\varphi(=E/q_c)$ decreases with increasing settlement level, as the elastic modulus degrades with settlement. However, φ tends to stabilize near the upper limit (10 cm) of the settlement range considered. It is also observed that values of φ increase with footing size at the same settlement level and relative density (D_R). In addition, it is seen that the effect of D_R on φ decreases as the settlement level increases. For the 1-m footing, as an example, the ratio of the φ value for D_R = 90% to that for D_R =30% at a settlement of 2 cm is around 2.3 (i. e., φ= 1.2 and 2.7 at D_R = 30 and 90%, respectively), while the same ratio at a settlement of 10 cm is approximately 1.8.In Fig. 4.10, the values of φ were normalized with respect to the corresponding φ_{25} values given in Figure4.8 and plotted as a function of settlement. For each settlement points (one for each combination of footing size and relative density) are plotted in Figure 4.10. Since normalized values. φ/φ_{25} were used in the figure, all values of φ/φ_{25} fall within a certain range for each settlement. Due to the different rates of soil stiffness degradation with relative density, the φ/φ_{25} range becomes wider as the settlement level increases. The appropriate φi for each sub-layer *i* can be determined for a given settlement using Figures 8 and 10, estimation of footing settlement is then done following the procedure outlined next.

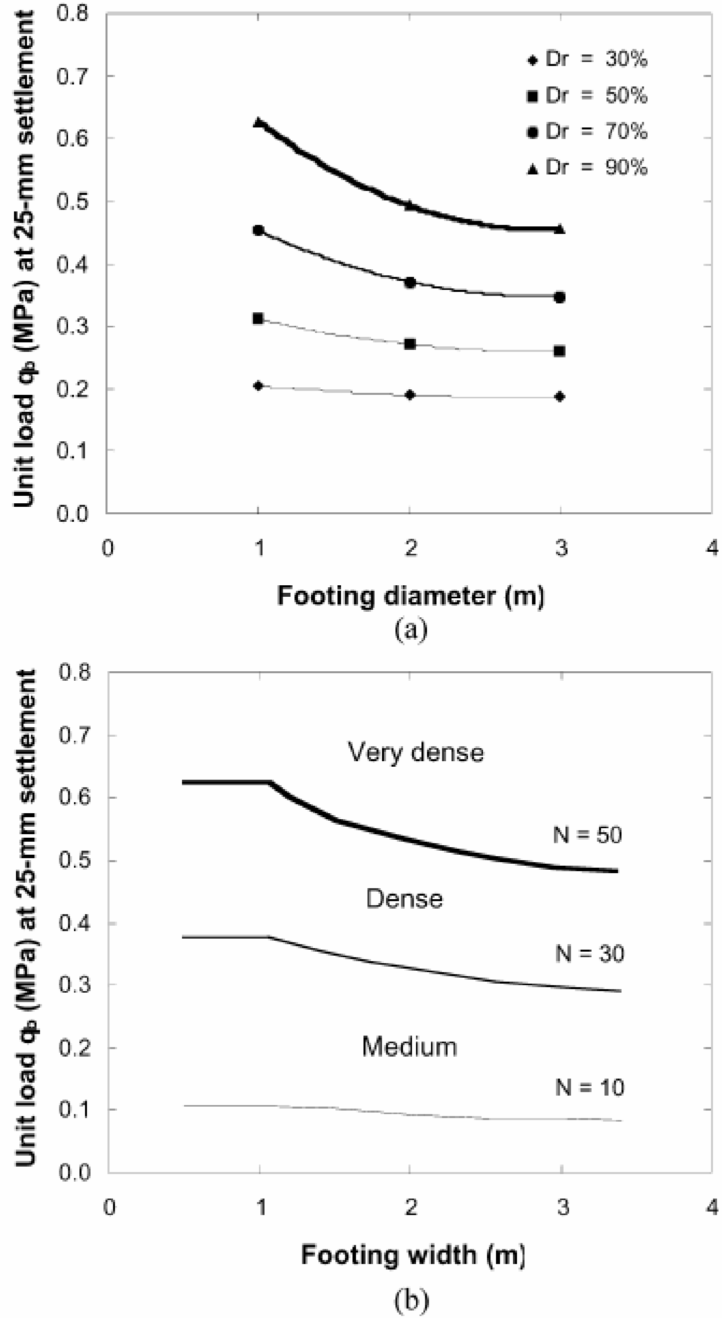
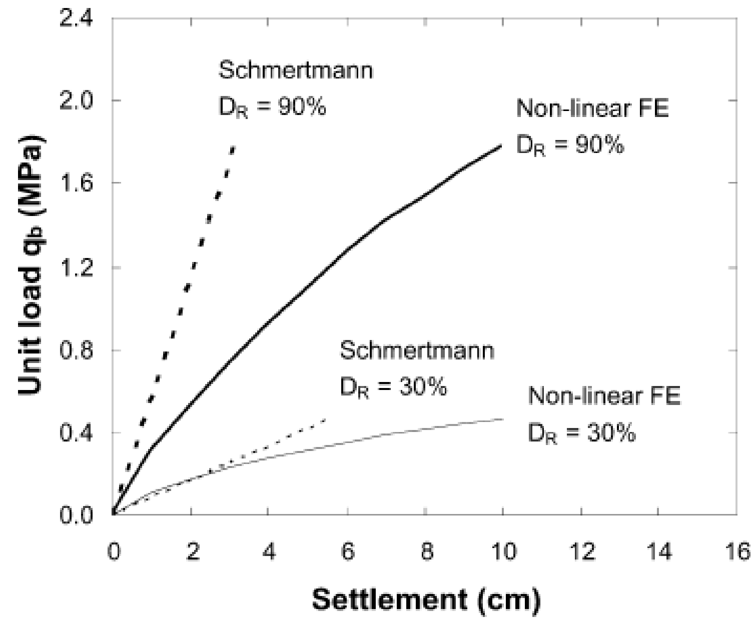
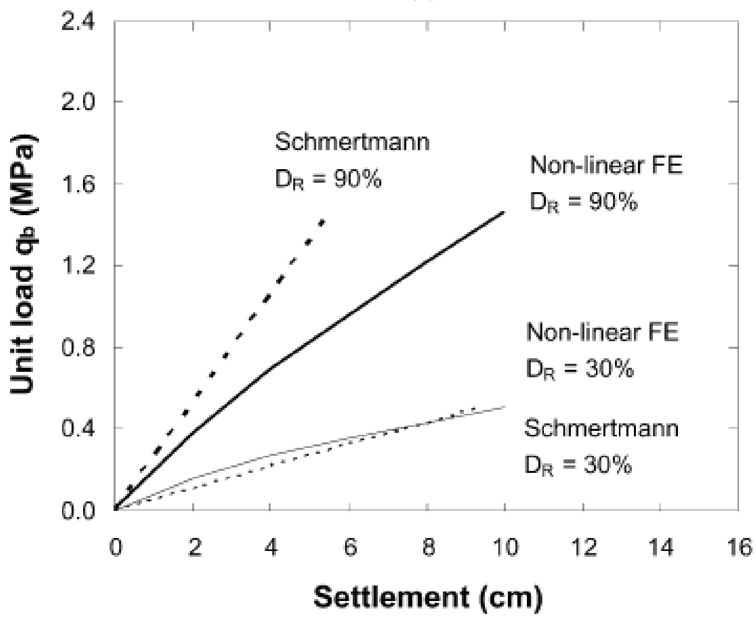


Fig. 4.6 Unit load at 25-mm settlement versus footing size from (a) FE analysis and (b) Terzaghi and Peck [45].



(a)



(b)

Fig. 4.7 Load-settlement curves at D_R =30% and 90% for (a) 1-m circular footing and (b) 3-m circular footing.

(a) Determine the tolerable settlement.

(b) Divide the influence zone in sub-layers with similar q_c values, and assigns a representative cone resistance q_{ci} to each sub-layer.

(c) Estimate the average relative density for each sub-layer.

(d) Determine φi for each sub-layer, for the given tolerable settlement, from either Figures 4.8 and 4.10 or Figure 4.9.

(e) Determine the elastic modulus E_i (from q_{ci}) for each sub-layer using the value of φi determined from (d).

(f) Calculate the allowable design load corresponding to the given tolerable settlement for the footing.

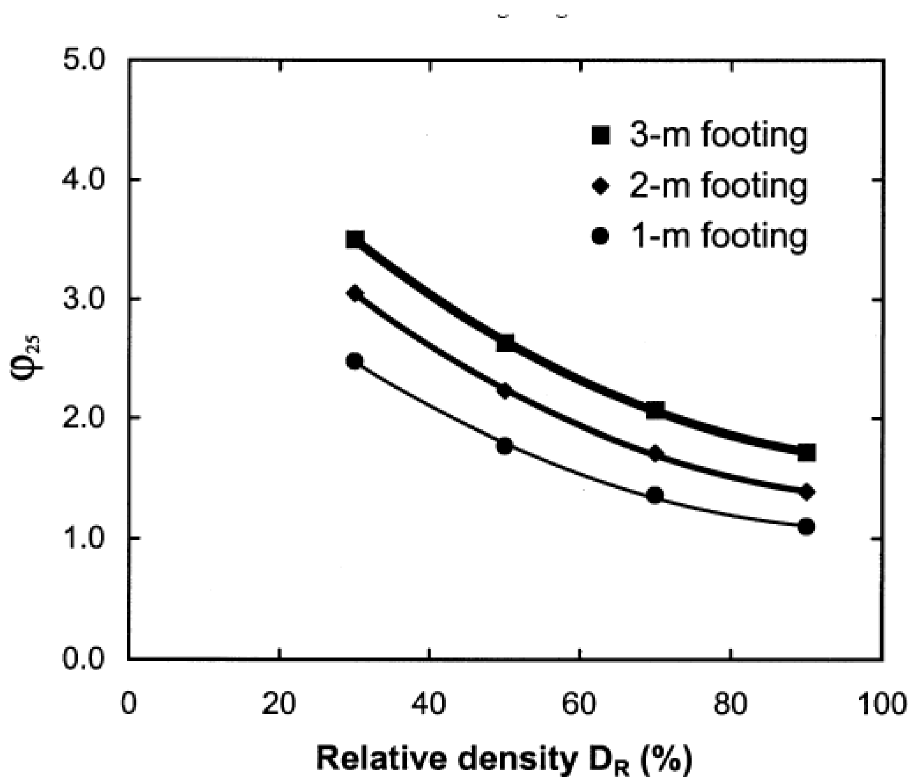


Fig. 4.8 Values of φ_{25} for circular footings with different sizes at different relative densities.

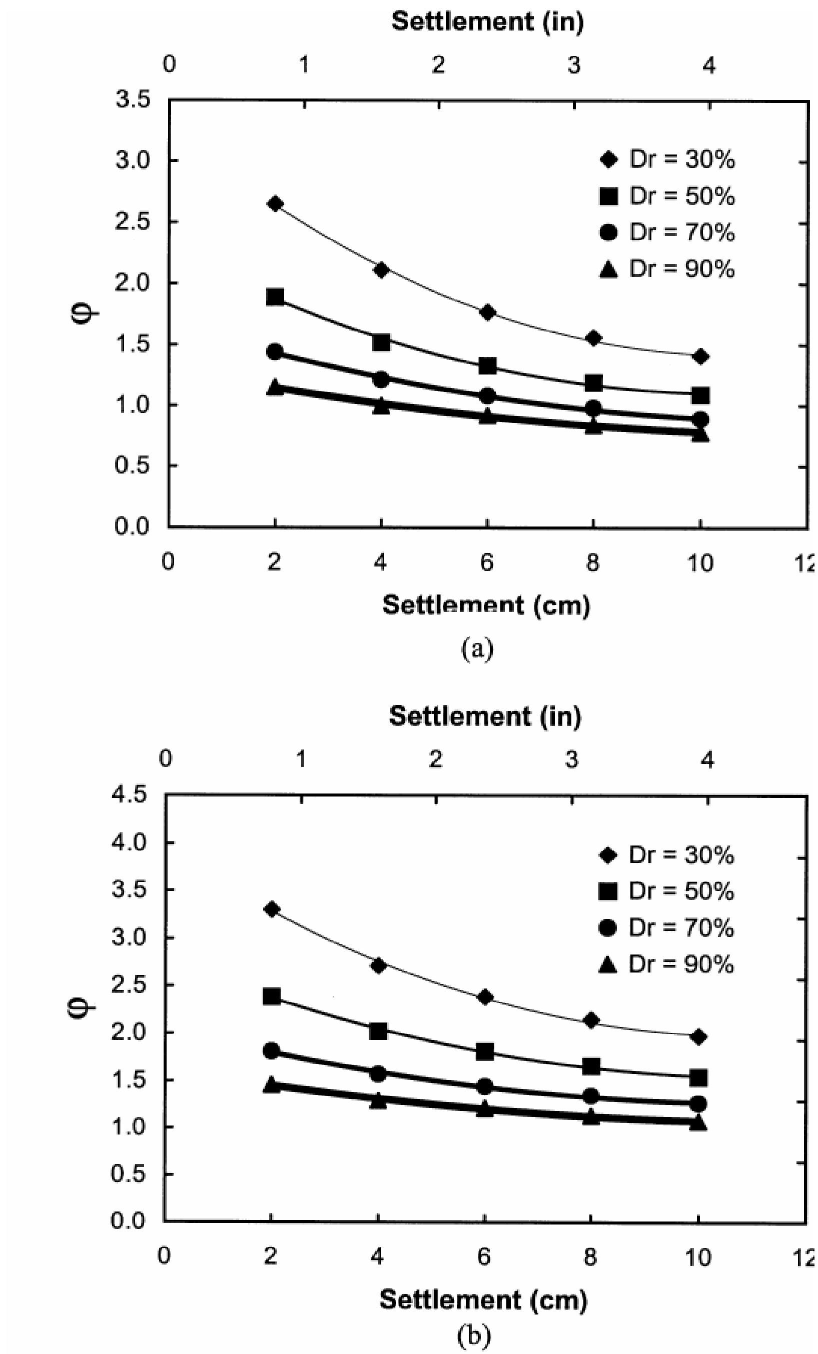
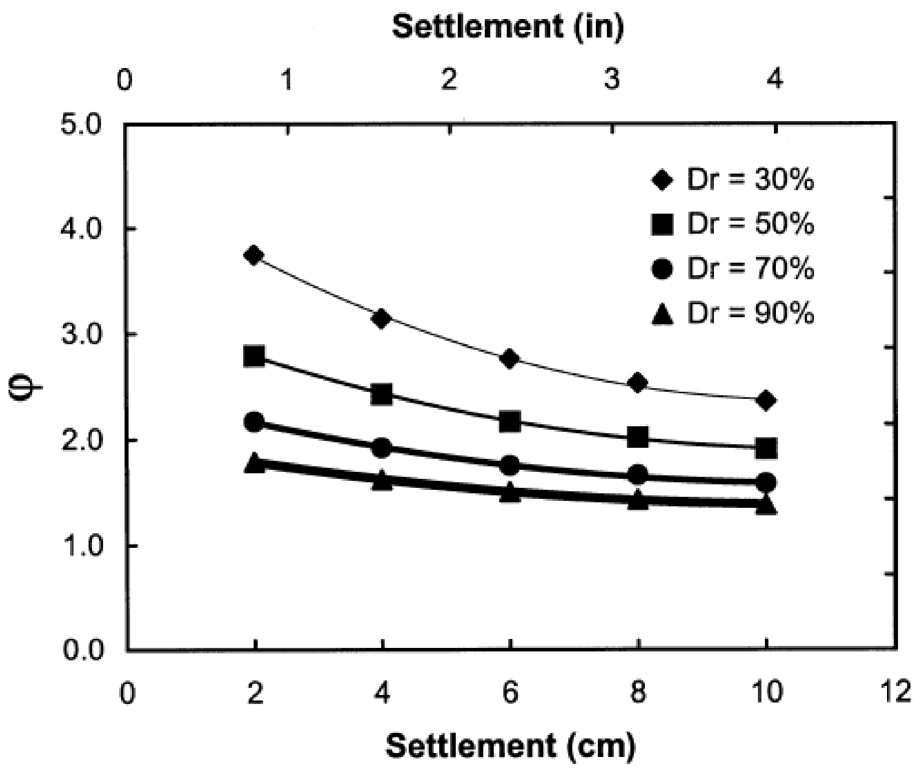


Fig. 4.9 Normalized φ values versus settlement for (a) 1-m circular footing and (b) 2-m circular footing.



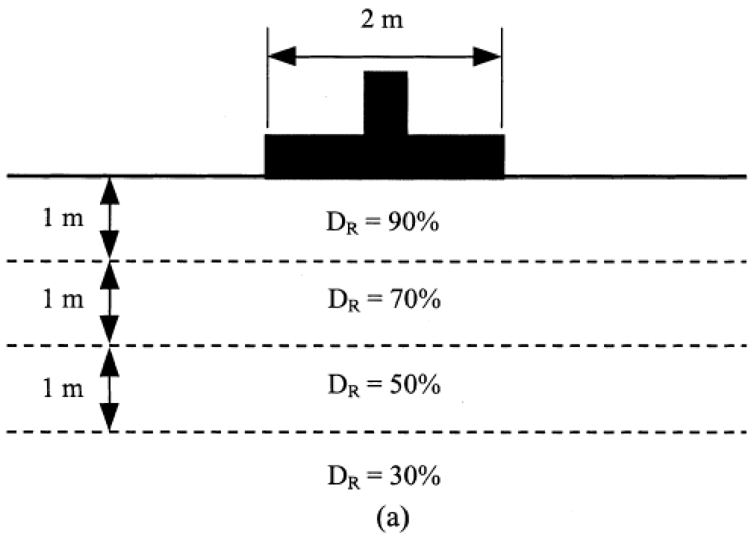
(c)

Fig. 4.9 Normalized φ values versus settlement for (c) 3-m circular footing.

4.7. EXAMPLES

In order to assess the applicability of the procedure proposed in this study to non-uniform soil profiles, some examples were prepared and analyzed using the finite element method, Schmertmann's method. Fig. 4.14 shows the footing and soil conditions considered. The footings were circular with a diameter equal to 2 m, bearing on layered sand, each layer having a different relative density. In the first soil profile, shown in Figure 4.14(a), the relative density decreases with depth, and in the second soil profile, shown in Figure 4.14(b), relative density increases with depth. Figure 4.15 shows the load-settlement responses for both cases obtained from the finite element analysis, the conventional Schmertmann's method, and the proposed procedure. For Schmertmann's method and the proposed method, the cone resistance q_{ci} was determined for each layer. As the soil was assumed to be normally consolidated, the value of the stiffness ratio (φ) used in Schmertmann's method was 2.5, while the stiffness ratios (φ) from Figure 4.9 were used for the proposed procedure. As can be seen in the figure, results obtained from the proposed method agree well with those from the finite element analysis for both cases. The difference between the proposed method and the FE analysis (in terms of calculated settlements for a given applied load) was never more than 8% for settlements up to 8 cm.

Schmertmann's method produces loads significantly higher than those from the finite element analysis for the case of decreasing density with depth. For the soil having increasing density with depth, Schmertmann's method is in good agreement with the loads from the finite element analysis up to about 5 cm.



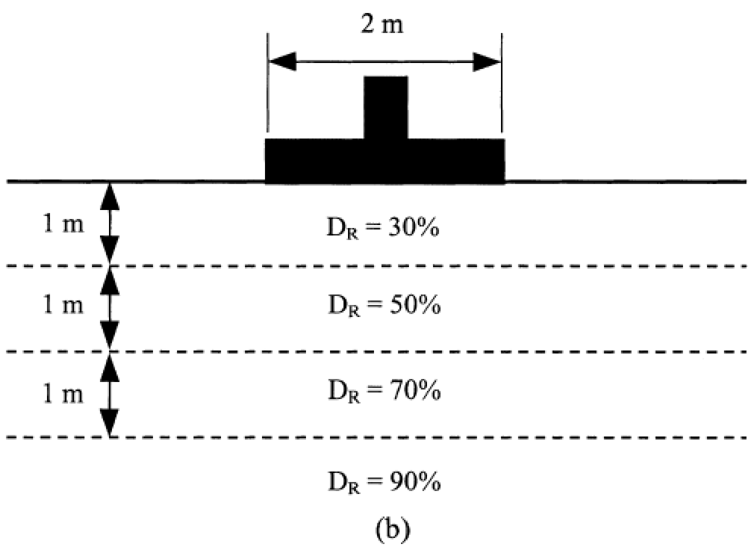


Fig. 4.14 Examples for layered soil profiles with (a) decreasing relative densities with depth and (b) decreasing relative densities with depth.

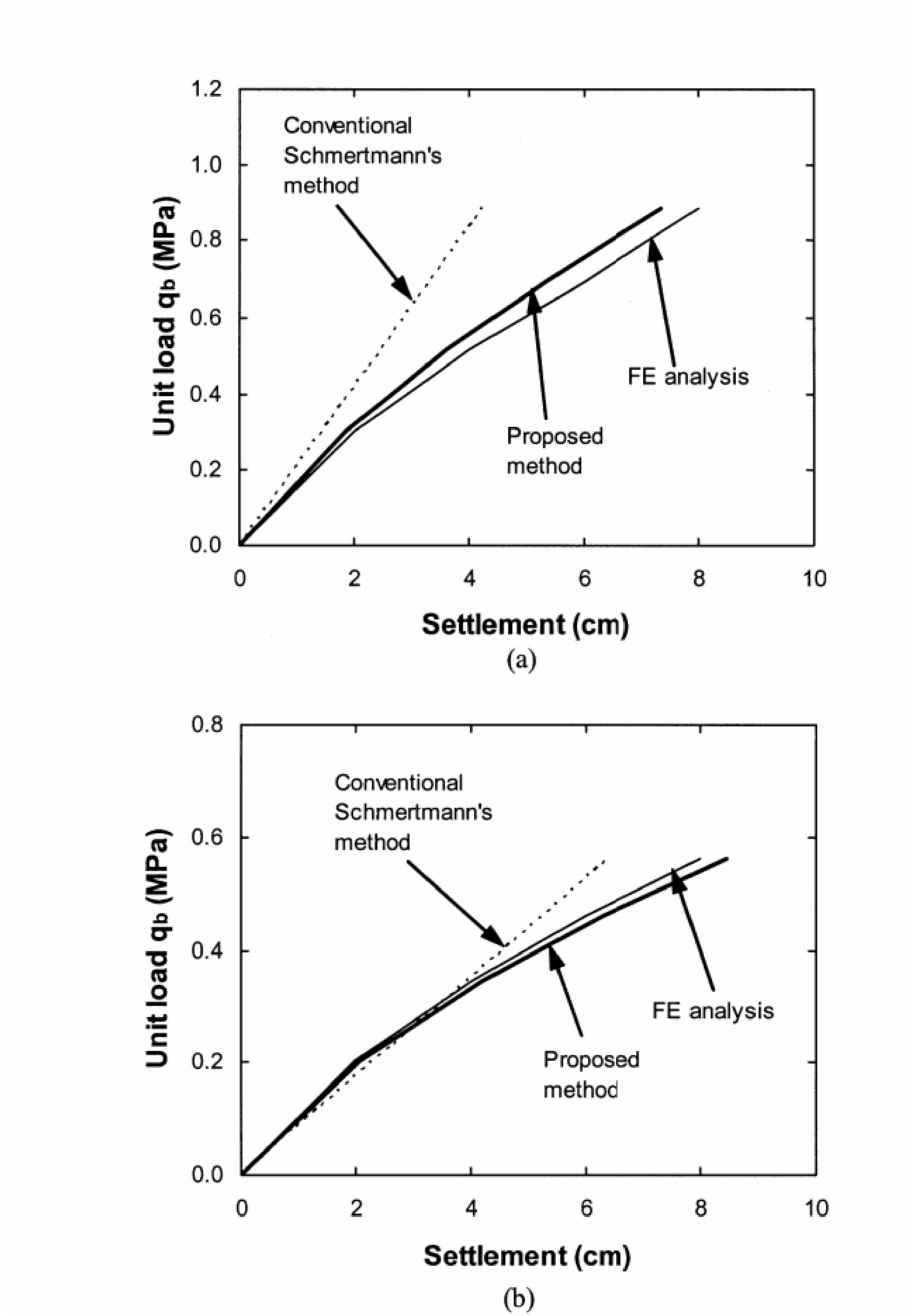


Fig. 4.15 Load-settlement responses for (a) the soil with decreasing relative densities with depth and (b) the soil with increasing relative densities with depth.

The load-settlement response of vertically loaded footings on sand was investigated using conventional elastic calculations and nonlinear finite element analysis. Based on the results of these analyses, a new settlement estimation procedure for footings on sands was developed for use when cone penetration resistance values are available. In order to validate the numerical analyses, field load tests performed on five footings. The load-settlement response predicted for each footing using the nonlinear finite element analysis was in reasonable agreement with the measured results. For analyzing the load-settlement response of footings for a variety of conditions, the loading of circular footings was modeled using the finite element method. Three different footing diameters (1, 2, and 3 m) were considered. The soil was assumed to be normally consolidated sand with relative densities equal to 30, 50, 70, and 90%. The normalized load-settlement curves for these footings show that the effect of footing size on base resistance is important. Relative density was also found to be an important factor in determining the design load of footings. The differences between the unit loads at 25 mm settlement for footings with different diameters become more pronounced as the relative density increases. These results were in good agreement with the allowable bearing pressures proposed by Terzaghi and Peck [45] with basis on load tests on footings in sand.

The most important step in the calculation of footing settlement using the conventional elastic approach is the estimation of a representative elasticity modulus of soil. In Schmertmann's method, the elasticity modulus is obtained from cone resistance by multiplying it by a number φ. We generalized this method for use in the calculation of settlements from small to large by obtaining the values of φ for use under various soil conditions and for a range of footing widths. The larger the footing size, the higher the value of φ was found to be. Relative density was also an important factor affecting the value of φ. As the relative density increases the value of φ decreases significantly. The results obtained in this study can be used to effectively calculate from CPT cone resistance either the footing settlement corresponding to a given load or the load corresponding to a design settlement for different footing sizes and soil conditions.

CHAPTER-5

NUMERICAL ANALYSIS USING PLAXIS SOFTWARE-AN OVERVIEW

CHAPTER-5

NUMERICAL ANALYSIS USING PLAXIS SOFTWARE

AN OVERVIEW

5.1GENERAL

Praxis version 8 is the finite element package for the two-dimensional analysis of deformation and stability on geo-technical engineering. PLAXIS is equipped with features to deal with various aspects of complex geo-technical structure. Real situation may be modeled either by a plane strain or as axisymmetric model. In a plane strain analysis, the calculated forced from prescribed displacements represent from per unit length on the out of plane direction (z-direction). On axisymmetric analyses, the calculated forces are those that act on the boundary of a subtending an angle of 1 radian. In order to obtain the forces corresponding to the complete problem therefore, these forces should be multiplied by a factor of 2*3.141

5.2 INPUT PROGRAM

To carry out finite element analysis using PLAXIS, the user has to create a finite element model and specify the material properties and boundary condition. This is done in the on put program. To set up a finite element model, the user must create a two-dimensional geometry model composed of points, lines and other components in the x-y plane. The generation of an appropriate finite mesh and the generation of properties and boundary conditions on an element level are automatically performed by the PLAXIS mesh generator based on the input of the geometry model. Uder may also customize the finite element mesh in water pressure and initial stresses to the initial state.

5.3 PREPARING MODE USING PLAXIS TOOLS

In principle, first draw the geometry contour, and then add the soil layers, then structural objects, then construction layers, then boundary conditions and then loadings. Using the geometry line option, the user may draw points and lines in the draw area. Plates are structural objects used to model slender structures in the ground with a significant flexural rigidity or normal stiffness. Plates can be used to simulate the walls shells or linings extending in z-direction. Geogrid are slender structures with a normal stiffness but no bending stiffness, are generally used to model reinforcements. To model the interaction between the sheet pile wall and the soil, the interfaces are used which is intermediate between smooth and fully rough.

5.4 MODELING OF SOIL BEHAVIOUR

In PLAXIS, soil properties and material properties of structure are stored in material data sets from the database sets are assigned to the soil clusters or to the corresponding structural objects in the geometry model. PLAXIS supports various model to simulate the behaviour of soil and other continua such as Linear Elastic model, Mohr-Coulomb Model, Jointed rock Model, Hardening soil model, soft model, soft soil creep model and user defined models. Once the geometry model has been created and finite element mesh has been generated, the initial stress state and the initial configuration must be specified. This is done by initial conditions part of the input program.

5.5 CALCULATIONS

After this, the actual finite element calculations can be executed. Therefore it is necessary to define which types of calculations are to be performed and which type of loadings or construction stages are to be activated during the calculations. PLAXIS Allows for different types of finite element calculations in the engineering practice, a project are divided into project phases. Similarly, a calculation process in PLAXIS is also divided into calculation phases. Examples of calculation phases are the activation of a particular loading at a certain time, the simulation of a construction stage, the introduction of a consolidation period, the calculation of safety factors etc.

5.6 OUTPUT PROGRAM

The main output quantities of a finite element calculation are the displacement at the nodes and the stresses at the stress points. In addition, when a finite element model involves structural elements, structural forces are calculated in these elements. Extensive ranges of facilities exist within PLAXIS to display the results of a finite analysis. The curves program can be used to draw load-displacement curves, stress-strain diagrams and stress or strain paths of pre-selected points in the geometry. These curves visualize, and this gives an insight into the global and local behaviour of the soil. When subsequentially clicking on the output button, the results of all selected phases are displayed on separate windows in the output program. In this way, results of different phases can easily be compared.

CHAPTER-6

PROBLEM DEFINATION AND FORMULATION

CHAPTER-6

PROBLEM DEFINATION AND FORMULATION

6.1 **General**

For the present study, a soil sample clean sand is collected. In order to characterize the soil under study the soil samples were subjected to following laboratory tests.

1.Grain size analysis

2. **Specific gravity test**

3. Consolidated Undrained Triaxial test

The above tests are conducted as per the Bureau of Indian Standard Code

6.1.1 **Grain size analysis**

For grain size analysis sieve of different size are used and results are shown in tabular form.

6.1.2 **Specific gravity test**

Specific gravity test was conducted by density bottle method. The results are tabulated below

6.1.3 **Consolidated Undrained Triaxial test**

Consolidated Undrained Triaxial test was conducted by an upgraded computer controlled Triaxial shear equipments were used for this test. The soil samples were compacted at 98% of maximum dry density. The test was conducted under three stages viz. Saturation, Consolidation and Shear. The soil samples were consolidated and sheared under different constant confining pressure of 1,2,3,and 4 kg/cm^2 respectively after achieving full saturation. The graphs are plotted as Strain vs. Deviator stress for different confining pressure and p vs. q graph for calculating effective and total shear strength parameter. The test results are tabulate below.

Table 6.1 Test results of sand samples performed in laboratory

Si. No.	Name of the test and detail	Valve
1	Specific gravity	2.65
2	Grain size analysis (Mechanical analysis) Silt (0.075 to 0.002 mm) Fine sand (0.425 to 0.075 mm) Medium sand (2 to 0.425 mm) Coarse sand (4.75 to 2 mm)	 3% 26% 34% 47%
3	Maximum void ratio Minimum void ratio Relative density (%)	0.78 0.48 74.0%
4	Consolidated Undrained Triaxial test Coefficient of cohesion C, kg/cm^2 Angle on internal friction (ϕ),	 0.06 29.5^0

6.2 **CASE OF STUDY**

Here define the problem handled for the present study and its formation by finite element method. Present study has been carried out to:

(1) Find the variation of the elastic modulus (E) with settlement in soil, under foundation.

(2) Evaluate the effect of variation of the elastic modulus (E) on settlement of soil under circular footing.

(3) Study the effect of the change of size of footing on settlement of soil under circular foundation.

(4) Analysis the effect of the change of percentage of silt content in sand on settlement of soil under circular footing.

(5) Compare the calculated result with finite element method (FEM) using software PAXIS.

6.2.1 **Circular foundation**

It was found that the mesh size used in this study, extending laterally to more than 4 times the footing radius and vertically to more than 3 times the footing radius, is large enough to eliminate boundary effects.

The ground water table was taken to be at a depth of 4m below ground level to decrease the effective of the pore water pressure on stresses in soil sample. Applied load over the foundation was taken as 200.00kPa so that soil can stress up to the elastic limit. The complete analysis has been carried out axisymmetrically, foundation of diameter 1m, 2m and 3m is taken for the analysis, Foundation is considered as flexible in nature and soil below the foundation is taken as having constant value of elastic modulus and elastic modulus changes with change in effective stress in soil.

6.2.**2 Details of study**

In this study Conventional Elastic Method (CEM) of calculations and finite element method (FEM) for numerical analysis is used for analysis, following cases is taken during study.

Case 1:

Soil mesh of designed size is taken for analysis. Properties of soil are taken as given in the table (1) for soil and footing of diameter 1m, 2m and 3m respectively. Find the value of the stresses and settlement as per the methods.

Case 2:

In the second case of study Properties of soil are taken as given in the table (1) for soil have Young modulus a function of stress, have footing of diameter 1m, 2m and 3m respectively. Find the value of the stresses, value of elastic modulus (E) and settlement as per the methods.

Case 3:

A foundation of given area is taken for analysis, by changing the percentage of silt content (0,5,10 and 15%) in sand, find the value of the stresses and settlement in soil under footing.

Case 4:

Footings are analyzed for different value of loading and find the settlement in soil and elastic range of soil under consideration.

6.2.3 **Properties of soil**

Properties assign to soil in the above cases are as below.

Table 6.2 Table of soil properties

Properties		Valves	
		Sand	Silty sand
Density (kN/m^3)	γ_{unsat}	18.00	16.50
	γ_{sat}	20.00	18.00
Young's modulus (kPa)		50000.00	50000.00
e_{max}		0.78	0.70
e_{min}		0.48	0.42
Relative density (%)		74.0	80.3
Poison's ratio		0.30	0.15
Dilatancy angle ψ		0.00	0.00
Angle of shearing resistance,ϕ		29.5^0	31.0^0
Cohesion, c (kN/m^2)		0.06	0.12
Permeability (m/day)	k_x	0.001	0.001
	k_y	0.001	0.001

PROPERTIES		SILTY SAND			
		Silt (%)			
		0.00	5.00	10.00	15.00
Density (kN/m^3)	γ_{unsat}	16.5	16.5	16.5	16.5
	γ_{sat}	18.00	18.00	18.00	18.00
Young's modulus (kPa)		20000.00	25000.00	27000.00	30000.00
e_{max}		0.8	0.70	0.65	0.63
e_{min}		0.48	0.42	0.36	0.32
Relative density (%)		75.0	80.3	74.3	77.4
Poison's ratio		0.15	0.15	0.15	0.15
Dilatancy angle ψ		0.00	0.00	0.00	0.00
Angle of shearing resistance,ϕ		29.5^0	31.6^0	33.0^0	33.0^0
Cohesion, c (kN/m^2)		0.06	.010	0.50	2.24
Permeability (m/day)	k_x	0.001	0.001	0.00	0.00
	k_y	0.001	0.001	0.001	0.001

Experimental Program

ASSUMPTION CONSIDERATION	NOTATION	REMARKS
Modulus of elasticity is constant with depth, lateral distance.	Soil type A	A constant value of elastic modulus is taken for soil. (E=50Mpa).
Modulus of elasticity is a function of effective stress.	Soil type B	An average value of elastic modulus is taken for every 0.5m depth of soil.
Water content	Constant	
Poisons ratio’s	Constant	
Water table	At depth of 4m	Constant

Foundation details

PARAMETERS	DETAILS	REMARKS
Radius (r) of circular foundation.	0.5m, 1.0m, 1.5m	Diameter=1m, 2m, 3m
Depth of each soil layer for soil type B.	0.5m	At depth 0.0m, 0.5m, 1.0m, 1.50m, 2.0m, 2.5m, 3.0m.
Depth of soil Analyzed.	6r(Radius) or 3.0m	Distance is taken from the soil-foundation interface.
Lateral distance of soil Analyzed.	8r(Radius) or 4.0m	Distance is taken from the center of foundation in each direction.
Lateral Intervals at which soil is analyzed.	0.25m each	0.0m, 25m, 0.50m, 0.75m, 1.0m, 1.25m,1.50m,1.75m , 2.0m, 2.25m,2.5m, 3.0m.

For the soil type B, the deformation modulus (elastic modulus) of the soil was varied with depth because below ground level, the effective stress varies linearly with depth. And also effective stress controls the compressibility and shear strength characteristics. So the elastic modulus of the soil varied with depth. An expression for the variation of the modulus of the soil as a function of the depth was derived based on the earlier work by Hardin and Drmevich (1972) as

$E_s = E_0 (p_v / p_a)^{0.64}$

Where

E_s =young modulus at any depth (kPa)

p_v = effective vertical stress in soil (kPa)

p_a=atmospheric pressure(kPa)

E_{o}=Young modulus under atmospheric pressure (kPa)

6.3 **Problem analysis using plaxis software**

An axisymmetric grid is first generated using plaxis software to simulate the foundation and surrounding soil. Then, constitutive parameters were assigned to the different zone of elements. Loads and other boundary conditions are imposed. Because of the axisymmetric of the geometry, only half the problem needs to be modeled. Restricting the vertical displacement at the bottom simulated end-bearing conditions. Soil modeled as linear-elastic model

6.4 **Closure**

The results obtained form the analysis are presented in the form of deformed shape, X and Y displacement values, loads vs. displacement curves.

CHAPTER-7

RESULTS AND DISCUSSION

CHAPTER-7

RESULTS AND DICUSSIONS

7.1 GENERAL

The results and discussions of all the cases are analyzed and presented in this chapter. Settlement, effective stresses and elastic modulus at the desired location are presented in the tables and graphs.

In this chapter tables shows the values of the effective stresses under foundation at different depth and lateral intervals, for different radius and silt content in soil. Tables and Fig. Shows the variation of the elastic modulus (Young modulus) with depth and lateral distance in the soil, Fig. also shows the influence of the radius on the elastic modulus of the soil under foundation.

Chapter also contains tables and Fig. shows the variation of the settlement with depth and lateral distance in the soil, and influence of the change of the size of the footing on the settlement of foundation.

Output of the PLAXIS software shows: Value of the settlement at different depth under the foundation in the form of principal direction and shading, for different value of diameter. In plaxis we got in tabular and graphical form the total vertical stresses, effective stresses, increment in stresses, increment in settlement, increment in strain, total strain etc.

Table 7.1 Vertical stresses vs. Depth for soil type A (r=0.5m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Vertical stresses (kPa)										
1	182.111	112.4	66.36	25.28	8.34	3.02	1.24	0.58	0.3	0.1	0.04
2	129.289	51.76	39.18	25.28	14.66	8.12	4.5	2.56	1.5	0.58	0.26
3	84.793	27.68	23.6	18.38	13.32	9.2	6.18	4.12	2.76	1.28	0.62
4	56.8916	16.8	15.2	12.96	10.52	8.2	6.22	4.62	3.4	1.84	1.02
5	39.9178	11.16	10.44	9.36	8.1	6.78	5.54	4.44	3.52	2.16	1.32
6	29.237	7.92	7.54	6.98	6.28	5.5	4.72	4	3.32	2.24	1.5

Table 7.2 Vertical settlements vs. Depth for soil type A (r=0.5m)

n=(a/r)	0	0.5	1	1.5	2	2.5	3	3.5	4	5	6
m=(z/r)	Vertical settlements (mm)										
0	3.624	3.214	2.451	1.745	0.725	0.675	0.345	0.228	0.158	0.101	0.1
1	2.52283	2.2772	1.6232	0.9824	0.6122	0.4082	0.284	0.2032	0.148	0.082	0.048
2	1.22993	1.1532	0.9596	0.7296	0.5288	0.378	0.2716	0.1974	0.145	0.081	0.047
3	0.66102	0.6356	0.5678	0.4768	0.3822	0.2968	0.2266	0.1718	0.13	0.075	0.045
4	0.36865	0.3588	0.3318	0.293	0.249	0.2048	0.1648	0.1306	0.102	0.062	0.038
5	0.1948	0.1908	0.1798	0.1634	0.1438	0.1228	0.1026	0.0844	0.068	0.044	0.028
6	0.08053	0.0792	0.0754	0.0698	0.0628	0.055	0.0472	0.04	0.033	0.022	0.015

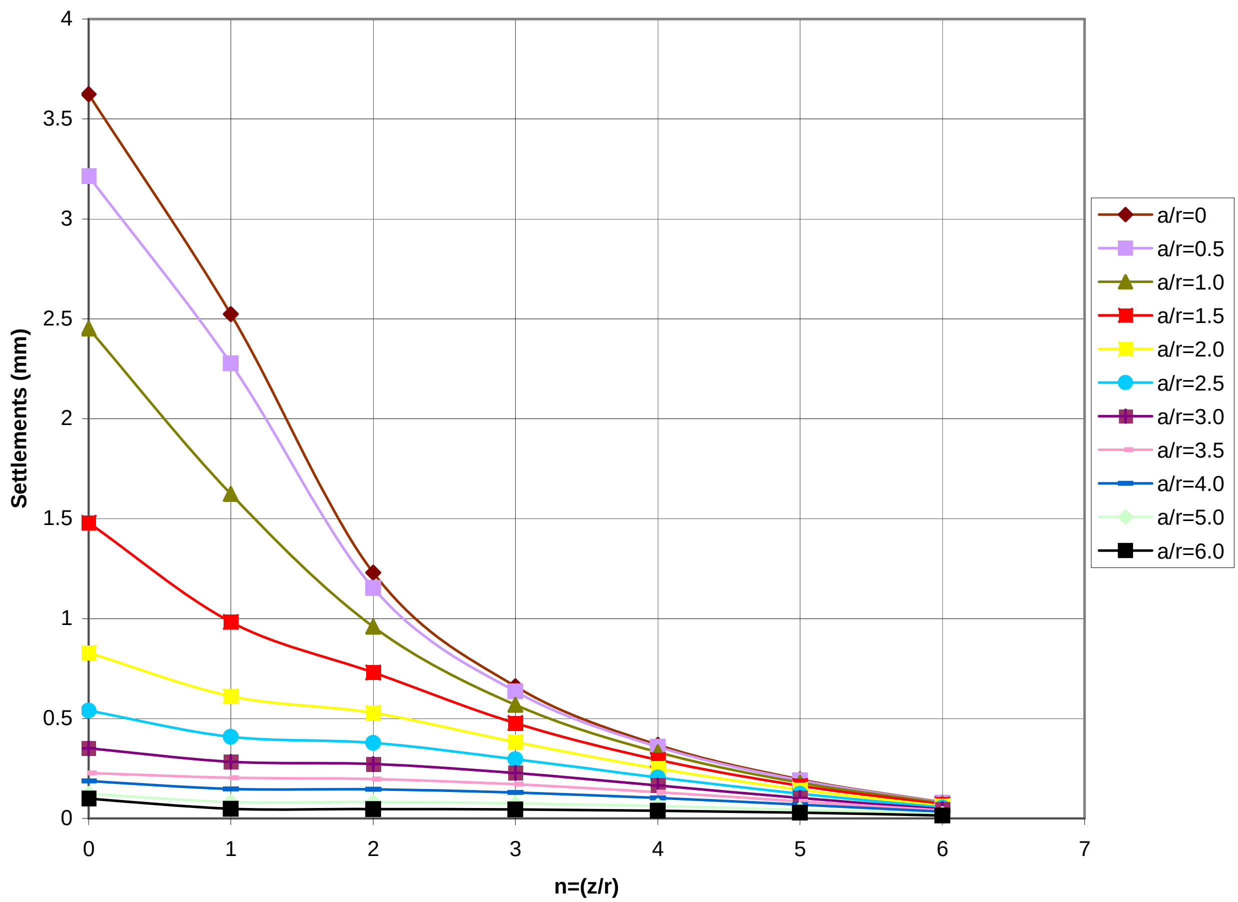


Fig. 7.2 Vertical settlements vs. Depth for soil type A (r=0.5m)

Table 7.3 Vertical stresses vs. Depth for soil type A (r=1.0m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Vertical stresses (kPa)										
0.5	198.4	179.22	167.86	138.26	83.14	39.66	12.02	4.7	2.08	0.56	0.2
1	175	125.08	112.4	91.68	66.36	41.26	25.28	14.48	8.34	3.02	1.24
1.5	133.637	82.24	74.98	64.04	51.2	38.5	27.56	19.08	12.98	6.02	2.9
2	97.6	55.54	51.76	46.04	39.18	32.02	25.28	19.42	14.66	8.12	4.5
2.5	71.8685	39.2	37.16	14.04	30.18	25.98	21.8	17.88	14.42	9.08	5.6
3	54.2	28.82	27.68	25.88	23.6	21.04	18.38	15.76	13.32	9.2	6.18

Table 7.4 Vertical settlements vs. Depth for soil type A (r=1.0m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Vertical settlements (mm)										
0	9.421	6.421	5.671	4.661	3.528	2.715	1.757	1.254	0.701	0.537	0.225
0.5	7.30705	5.101	4.7184	3.7994	2.9366	1.9846	1.3032	0.9132	0.658	0.36	0.206
1	5.32305	3.3088	3.0398	2.4168	2.1052	1.588	1.183	0.8662	0.637	0.354	0.204
1.5	3.57305	2.058	1.9158	1.5	1.4416	1.1754	0.9302	0.7214	0.554	0.324	0.192
2	2.23668	1.2356	1.166	0.8596	0.9296	0.7904	0.6546	0.5306	0.424	0.264	0.163
2.5	1.26068	0.6802	0.6484	0.3992	0.5378	0.4702	0.4018	0.3364	0.277	0.183	0.118
3	0.542	0.2882	0.2768	0.2588	0.236	0.2104	0.1838	0.1576	0.133	0.092	0.062

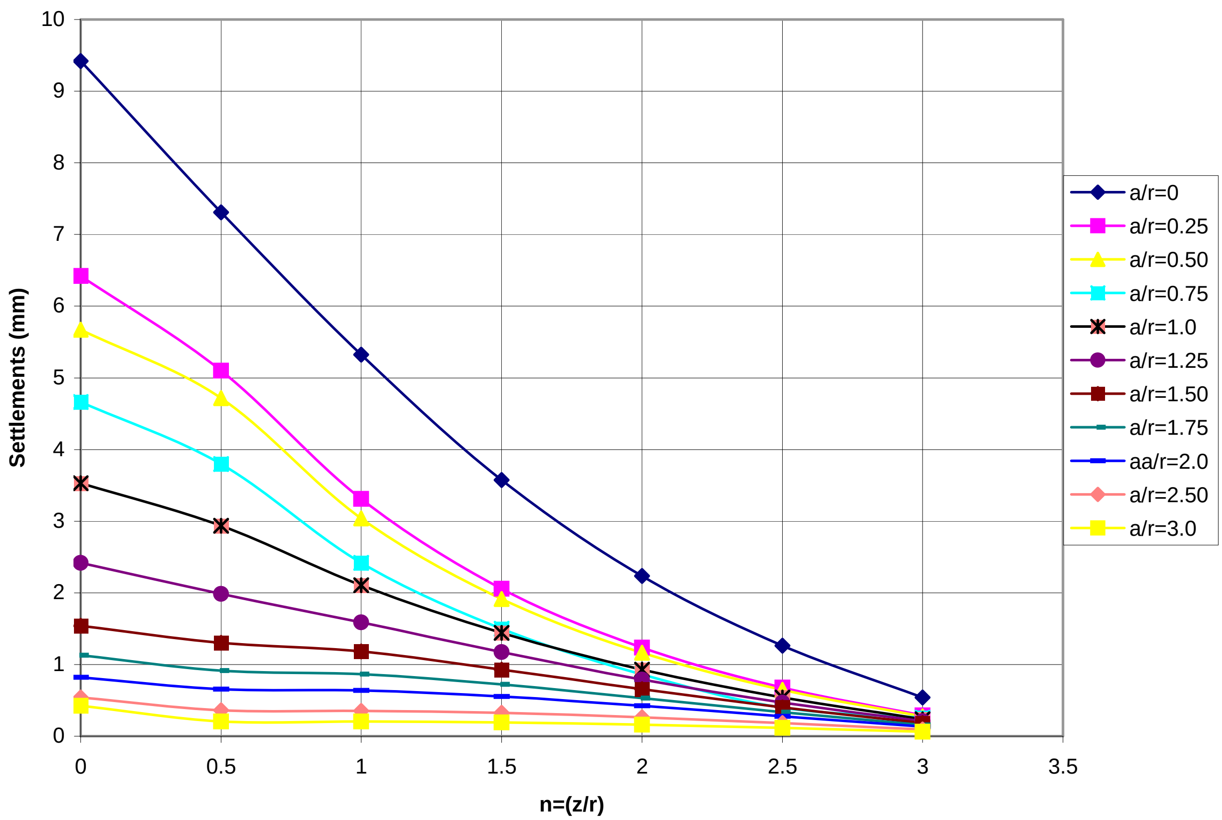


Fig. 7.4 Vertical settlements vs. Depth for soil type A (r=1.0m)

Table 7.5 Vertical stresses vs. Depth for soil type A (r=1.5m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Vertical stresses (kPa)										
0.3	199.8	190	178	172	162	142	84	20	6	0.4	0.28
0.7	194.174	162	148	150	116	88	76	60	20	16	3
1	175	128	116	112.4	96.02	76	74.74	52	34	18.8	8.34
1.3	147.571	106	98	91.5	81	66	56	48	28	24.2	12.04
1.7	120.492	76	78	74.98	62	48.2	46	42	26	26.2	13.6
2	97.6	61	52	51.76	48	42	39.18	33	24.2	20.2	14.66

Table 7.6 Vertical settlements vs. Depth for soil type A (r=1.5m)

N=(A/R)	0	0.16667	0.3333	0.5	0.6667	0.8333	1	1.167	1.333	1.667	2
M=(z/r)	Vertical settlements (mm)										
0	11.455	9.112	8.554	7.654	6.589	6.001	4.558	2.745	1.543	1.245	0.765
0.3	9.34637	7.23	6.7	6.5264	5.6502	4.622	3.7592	2.55	1.382	1.058	0.519
0.7	7.34837	5.33	4.92	4.8064	4.0302	3.202	2.9192	2.35	1.322	1.054	0.516
1	5.40663	3.71	3.44	3.3064	2.8702	2.322	2.1592	1.75	1.122	0.894	0.486
1.3	3.65663	2.43	2.28	2.1824	1.91	1.562	1.4118	1.23	0.782	0.706	0.403
1.7	2.18092	1.37	1.3	1.2674	1.1	0.902	0.8518	0.75	0.502	0.464	0.283
2	0.976	0.61	0.52	0.5176	0.48	0.42	0.3918	0.33	0.242	0.202	0.147

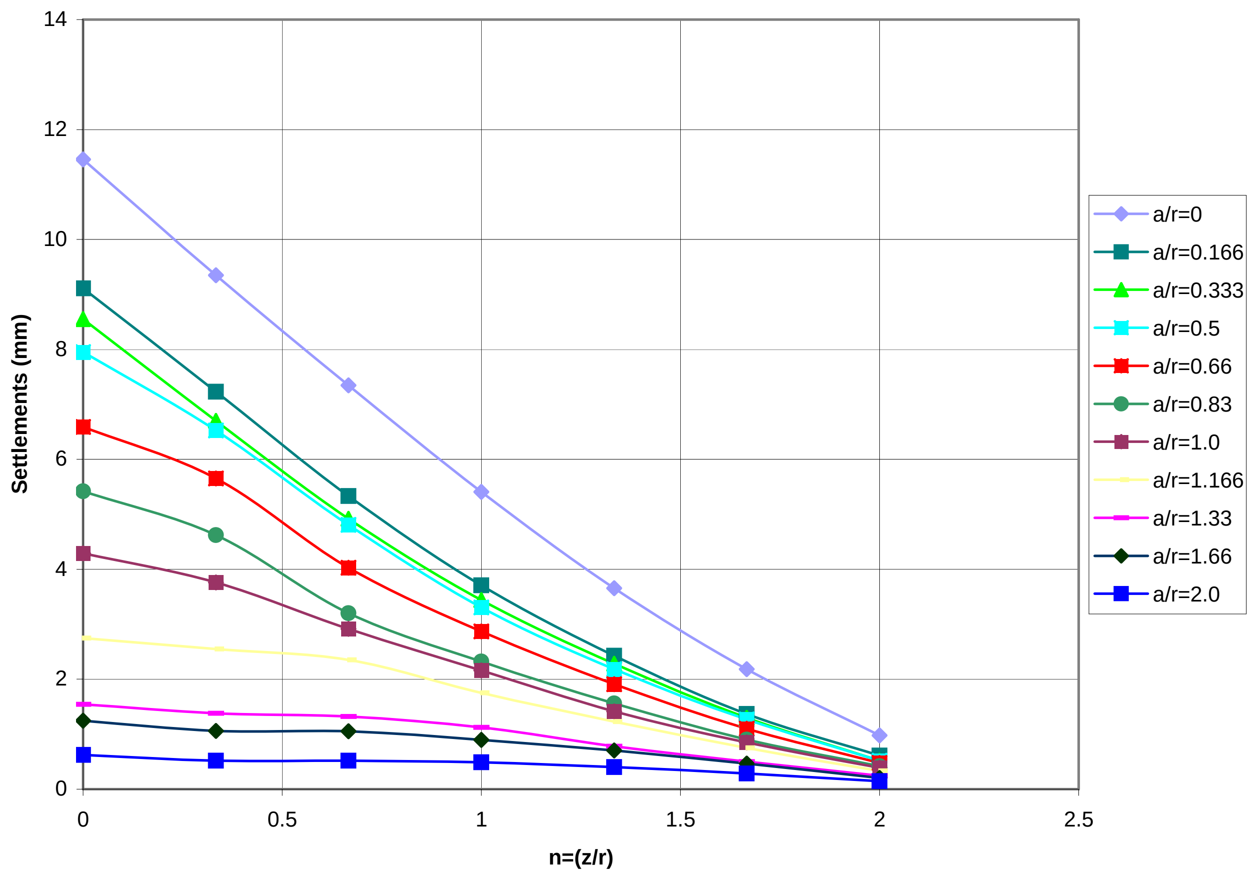


Fig. 7.6 Vertical settlements vs. Depth for soil type A (r=1.5m)

Table 7.7 Vertical stresses vs. Depth for soil type B (r=0.5m)

n=(a/r)	0	0.5	1	1.5	2	2.5	3	3.5	4	5	6
m=(z/r)	Vertical stresses (kPa)										
1	129.289	112.4	66.36	25.28	8.34	3.02	1.24	0.58	0.3	0.1	0.04
2	56.8916	51.76	39.18	25.28	14.66	8.12	4.5	2.56	1.5	0.58	0.26
3	29.237	27.68	23.6	18.38	13.32	9.2	6.18	4.12	2.76	1.28	0.62
4	17.3849	16.8	15.2	12.96	10.52	8.2	6.22	4.62	3.4	1.84	1.02
5	11.4268	11.16	10.44	9.36	8.1	6.78	5.54	4.44	3.52	2.16	1.32
6	8.05307	7.92	7.54	6.98	6.28	5.5	4.72	4	3.32	2.24	1.5

Table 7.8 Young modulus vs. Depth for soil type B (r=0.5m)

N=(A/R)	0	0.5	1	1.5	2	2.5	3	3.5	4	5	6
M=(z/r)	Young modulus (kPa)										
1	59543.3	54136.6	37833	19627	9233.4	4627.8	2526.37	1507	962.5	456	244.6
2	34072.4	31951.1	26440	19627	13550	9067.1	6069.52	4136	2875	1507	873.3
3	21667.3	20875.8	18731	15802	12695	9870.6	7530.82	5716	4353	2582	1577
4	15215.6	14865.5	13887	12461	10813	9127.7	7563.93	6179	5016	3304	2212
5	11438.2	11255.9	10757	9987.1	9051.9	8020.6	6991.28	6014	5136	3685	2636
6	9016.21	8914.63	8621.5	8180.7	7613.5	6956.9	6269.75	5602	4936	3777	2875

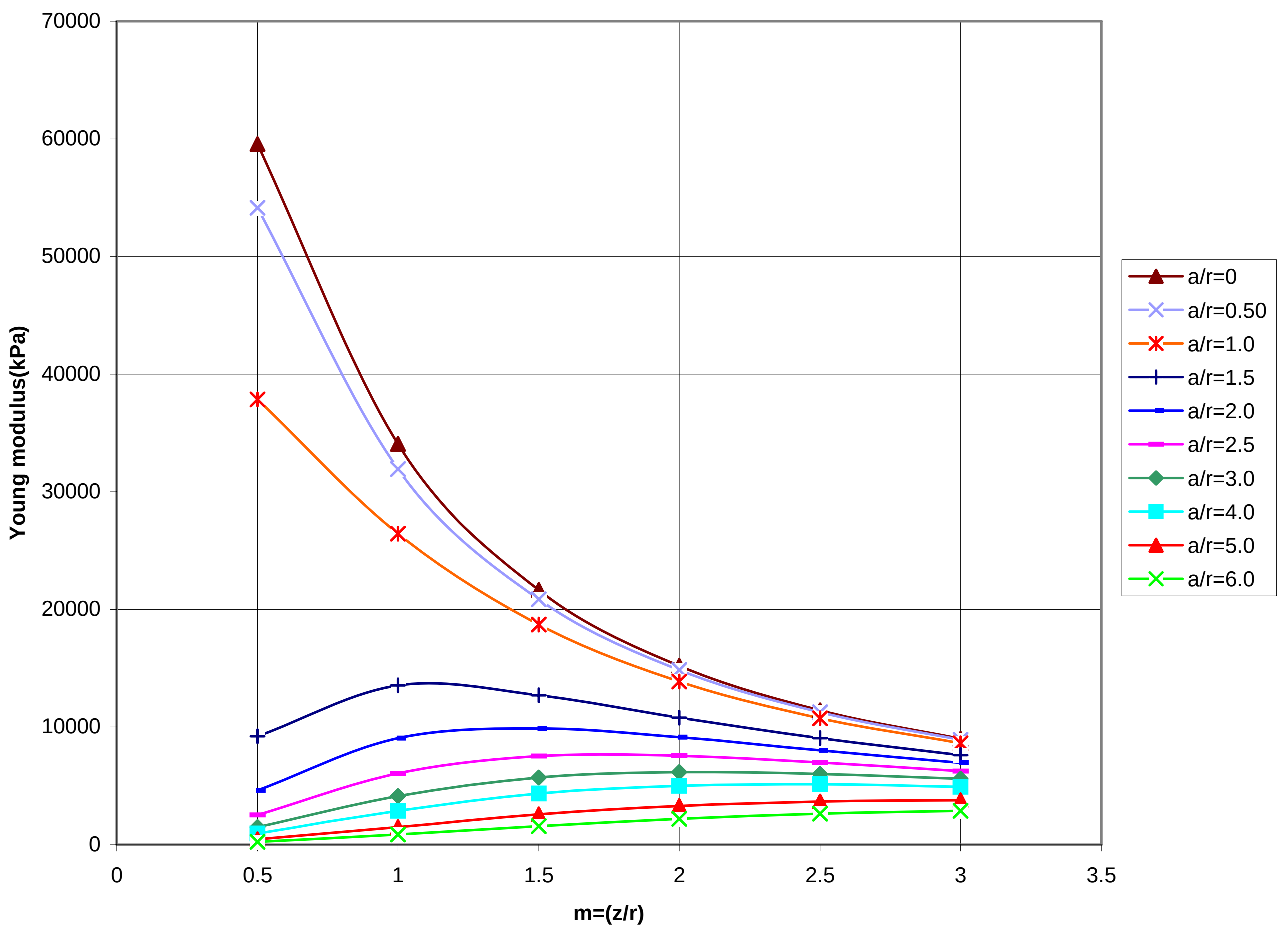


Fig. 7.8 Young modulus vs. Depth for soil type B (r=0.5m)

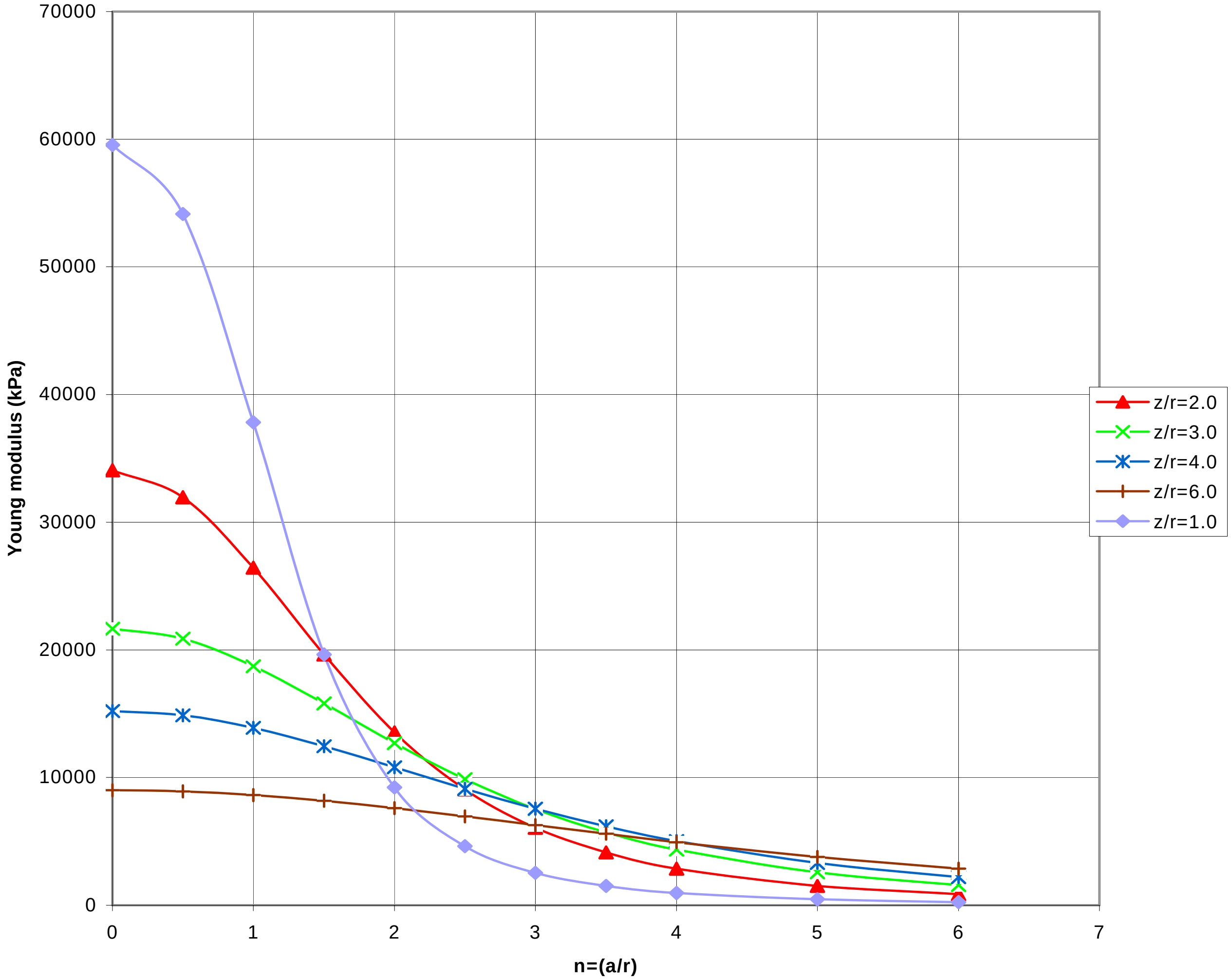


Fig. 7.9 Young modulus vs. Lateral distance for soil type B (r=0.5m)

Table 7.10 Vertical settlements vs. Depth for soil type B (r=0.5m)

N=(A/R)	0	0.5	1	1.5	2	2.5	3	3.5	4	5	6
M=(z/r)	Vertical settlements (mm)										
0	4.985	4.652	4.228	3.624	3.241	2.947	2.556	2.141	1.982	1.627	1.352
1	4.1126	4.01609	3.7177	3.2848	2.8635	2.5072	2.21021	1.962	1.752	1.418	1.169
2	3.02692	2.97797	2.8407	2.6408	2.4119	2.1809	1.9648	1.77	1.596	1.308	1.087
3	2.19206	2.16799	2.0998	1.9968	1.8709	1.7332	1.59409	1.46	1.335	1.116	0.938
4	1.51738	1.50502	1.4698	1.4153	1.3463	1.2671	1.18378	1.1	1.018	0.868	0.742
5	0.94609	0.93995	0.9226	0.8952	0.8598	0.818	0.77262	0.726	0.679	0.59	0.511
6	0.44659	0.44421	0.4373	0.4266	0.4124	0.3953	0.37641	0.357	0.336	0.297	0.261

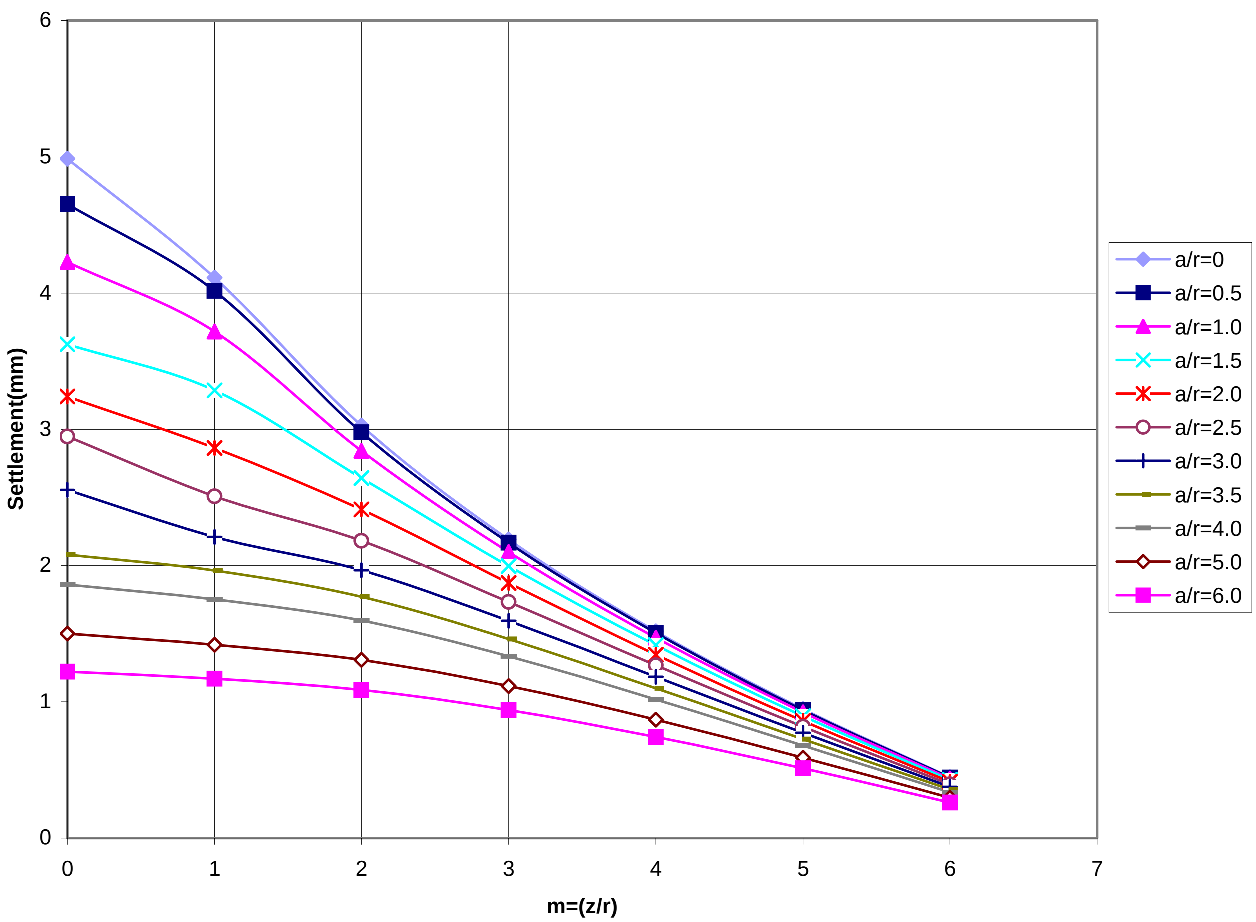


Fig.7.10 Vertical settlements vs. Depth for soil type B (r=0.5m)

Table 7.11 Vertical stresses vs. Depth for soil type B (r=1.0m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Vertical stresses (kPa)										
0.5	198.4	179.22	167.86	138.26	83.14	39.66	12.02	4.7	2.08	0.56	0.2
1	175	125.08	112.4	91.68	66.36	41.26	25.28	14.48	8.34	3.02	1.24
1.5	133.637	82.24	74.98	64.04	51.2	38.5	27.56	19.08	12.98	6.02	2.9
2	97.6	55.54	51.76	46.04	39.18	32.02	25.28	19.42	14.66	8.12	4.5
2.5	71.8685	39.2	37.16	14.04	30.18	25.98	21.8	17.88	14.42	9.08	5.6
3	54.2	28.82	27.68	25.88	23.6	21.04	18.38	15.76	13.32	9.2	6.18

Table 7.12 Young modulus vs. Depth for soil type B (r=1.0m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Young modulus (kPa)										
0.5	79670.6	74348.6	71111	62322	44100	26659	11838.7	6252	3591	1471	730.6
1	73153.6	58218.1	54137	47132	37833	27386	19627.2	13437	9233	4628	2526
1.5	60897.5	43775	41109	36928	31716	26127	20814.2	16209	12474	7398	4502
2	49180.8	33519.8	31951	29505	26440	23049	19627.2	16405	13550	9067	6070
2.5	39940.7	26448.7	25505	13158	22140	19995	17746.8	15509	13399	9783	7043
3	32967.8	21456.6	20876	19943	18731	17324	15802.5	14233	12695	9871	7531

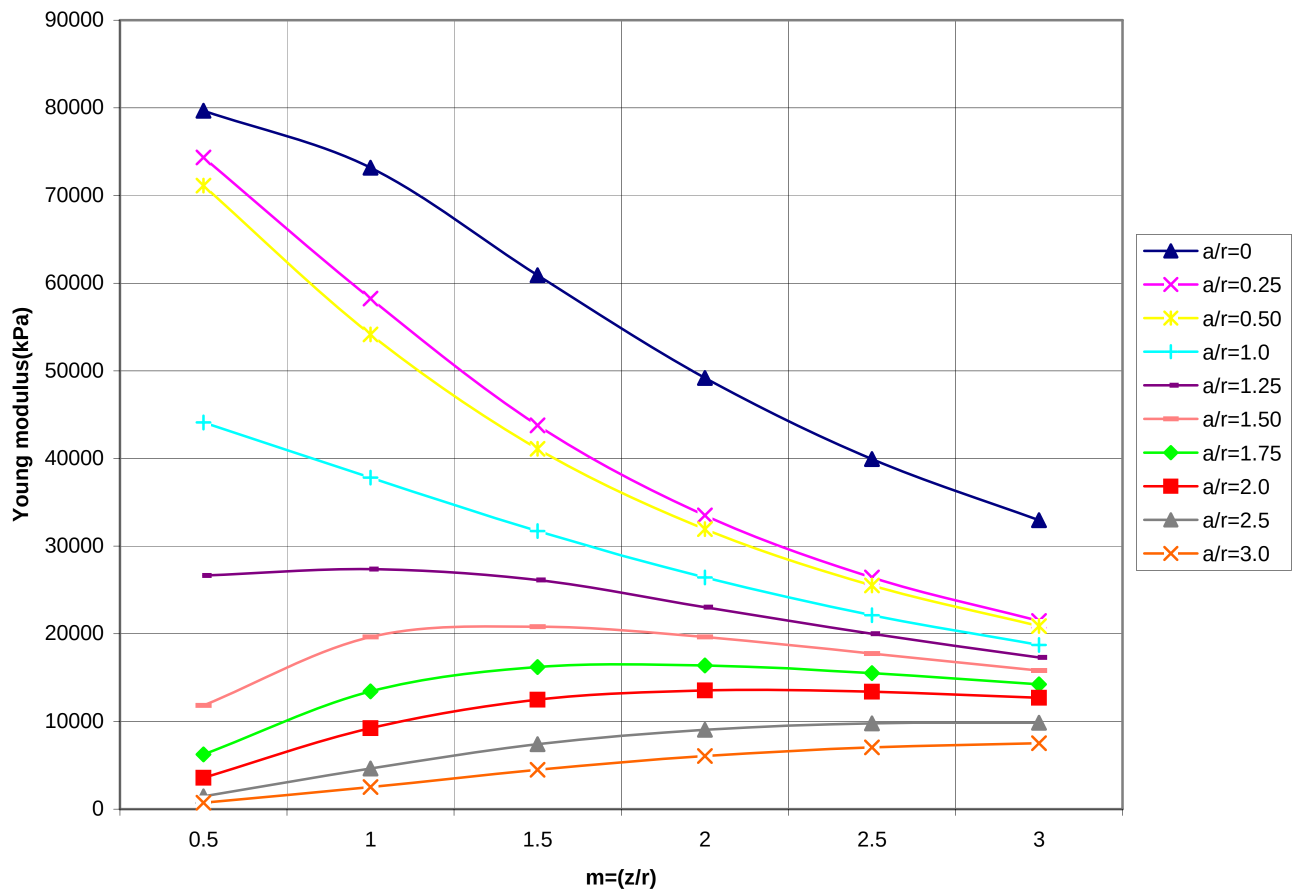


Fig.7.12 Young modulus vs. Depth for soil type B (r=1.0m)

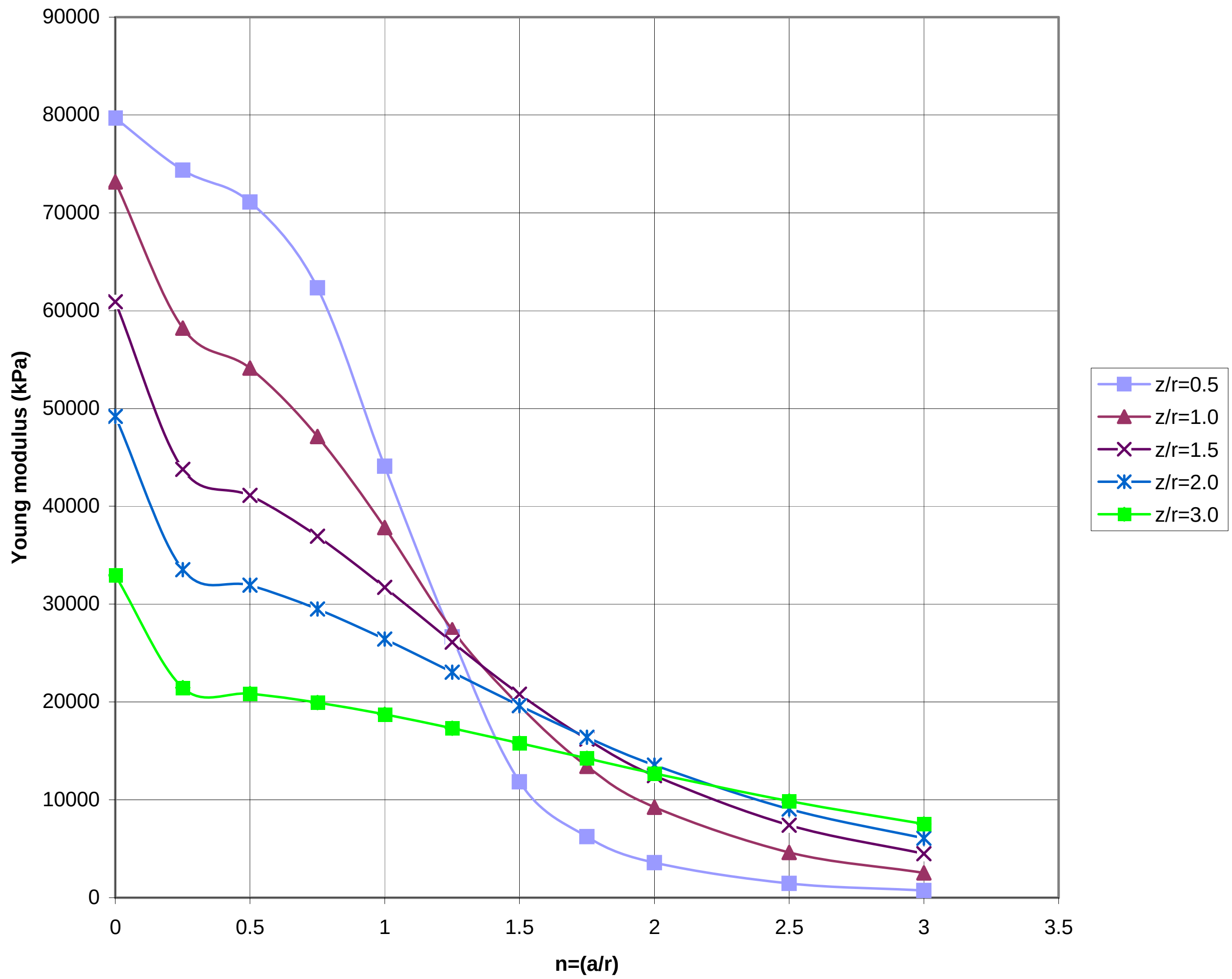


Fig. 7.13 Young modulus vs. lateral distance for soil type B (r=1.0m)

Table 7.17 Vertical Settlement vs. Depth for soil type B (r=1.0m)

n=(a/r)	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	3
m=(z/r)	Vertical Settlement (mm)										
0	7.569	6.712	6.252	5.685	5.221	4.627	4.111	3.556	3.01	2.655	2.021
0.5	6.25243	5.45996	5.3318	4.9115	4.6793	4.1854	3.65346	3.225	2.865	2.301	1.883
1	5.0073	4.25469	4.1515	3.8023	3.7367	3.4416	3.14581	2.849	2.576	2.111	1.746
1.5	3.81119	3.18046	3.1134	2.8297	2.8597	2.6883	2.5018	2.311	2.124	1.785	1.501
2	2.71396	2.24111	2.2014	1.9626	2.0525	1.9515	1.83975	1.722	1.604	1.378	1.179
2.5	1.7217	1.41264	1.3915	1.1824	1.3116	1.2569	1.19575	1.13	1.063	0.93	0.808
3	0.82202	0.67159	0.663	0.6489	0.63	0.6073	0.58155	0.554	0.525	0.466	0.41

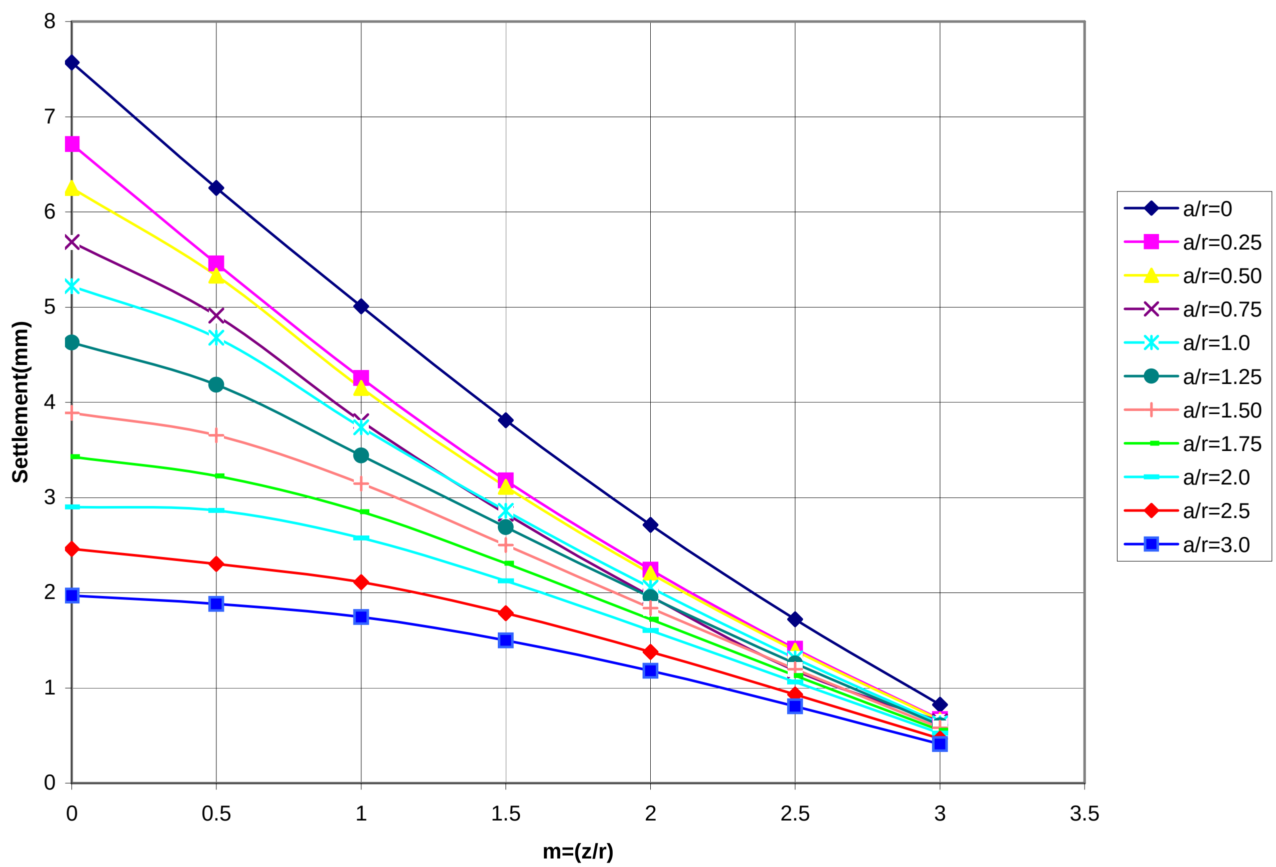


Fig.7.17 Vertical Settlement vs. Depth for soil type B (r=1.0m)

Table 7.14 Vertical stresses vs. Depth for soil type B (r=1.50m)

n=(a/r)	0	0.16667	0.3333	0.5	0.6667	0.8333	1	1.167	1.333	1.667	2
m=(z/r)	Vertical stresses (kPa)										
0.33	199.8	190	178	172	162	142	84	20	6	0.4	0.28
0.67	194.174	162	148	150	116	88	76	60	20	16	3
1	175	128	116	112.4	96.02	76	74.74	52	34	18.8	8.34
1.33	147.571	106	98	91.5	81	66	56	48	28	24.2	12.04
1.67	120.492	76	78	74.98	62	48.2	46	42	26	26.2	13.6
2	97.6	61	52	51.76	48	42	39.18	33	24.2	20.2	14.66

Table 7.15 Young modulus vs. Depth for soil type B (r=1.5m)

n=(a/r)	0	0.16667	0.3333	0.5	0.6667	0.8333	1	1.167	1.333	1.667	2
m=(z/r)	Young modulus (kPa)										
0.33	80052.5	77361.1	74004	72299	69413	63464	44409.9	16737	7381	1170	918.4
0.67	78512.7	69412.9	65275	65874	55310	45837	41488.1	35327	16737	14380	4607
1	73153.6	59138.8	55310	54137	48638	41488	41019.1	32052	24009	16047	9233
1.33	65146.5	52020.9	49318	47069	43325	37693	33708.4	30354	21040	19053	11852
1.67	56757.1	41488.1	42227	41109	36124	30440	29488	27719	20006	20110	12876
2	49180.8	35726.8	32052	31951	30354	27719	26439.5	23527	19053	16850	13550

Table 7.17 Vertical Settlement vs. Depth for soil type B (r=1.50m)

N=(A/R)	0	0.16667	0.3333	0.5	0.6667	0.8333	1	1.167	1.333	1.667	2
M=(z/r)	Vertical Settlement (mm)										
0	8.242	7.52	6.953	6.614	6.304	5.922	5.387	4.927	4.112	3.342	2.634
0.33	6.86695	6.26558	6.1133	6.0601	5.7863	5.4194	5.12427	4.507	3.662	3.199	2.507
0.67	5.61902	5.03758	4.9106	4.8706	4.6193	4.3007	4.17853	3.91	3.256	3.028	2.354
1	4.38245	3.87065	3.777	3.7321	3.5707	3.3407	3.26261	3.061	2.658	2.472	2.029
1.33	3.18633	2.78845	2.7283	2.6939	2.5836	2.4248	2.35157	2.25	1.95	1.886	1.577
1.67	2.05372	1.76963	1.7348	1.722	1.6488	1.5493	1.52091	1.459	1.285	1.251	1.069
2	0.99226	0.8537	0.8112	0.81	0.7907	0.7576	0.74094	0.701	0.635	0.599	0.541

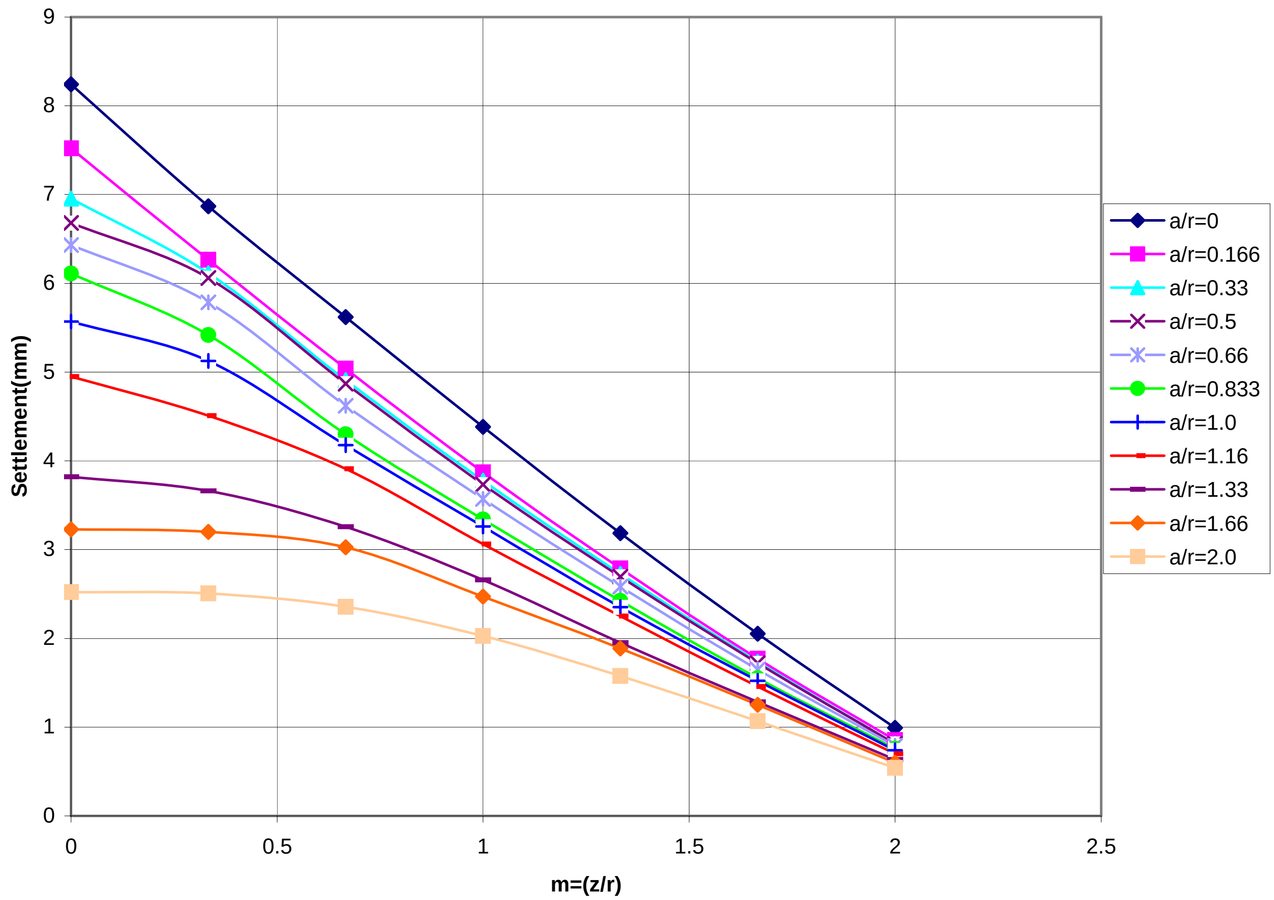


Fig.7.17 Vertical settlement vs. Depth for soil type B (r=1.50m)

Table 7.18 Vertical settlements vs. Silt (%) for soil type A (r=0.5m)

SILT %	0	5	10	15
M=(z/r)	Settlements (mm)			
0	7.451	5.884	5.425	4.854
0.5	6.307	5.045	4.671	4.304
1	3.07	2.459	2.277	2.04
1.5	1.6525	1.322	1.224	1.101
2	0.9216	0.737	0.682	0.614
2.5	0.487	0.389	0.36	0.324
3	0.2013	0.161	0.149	0.133

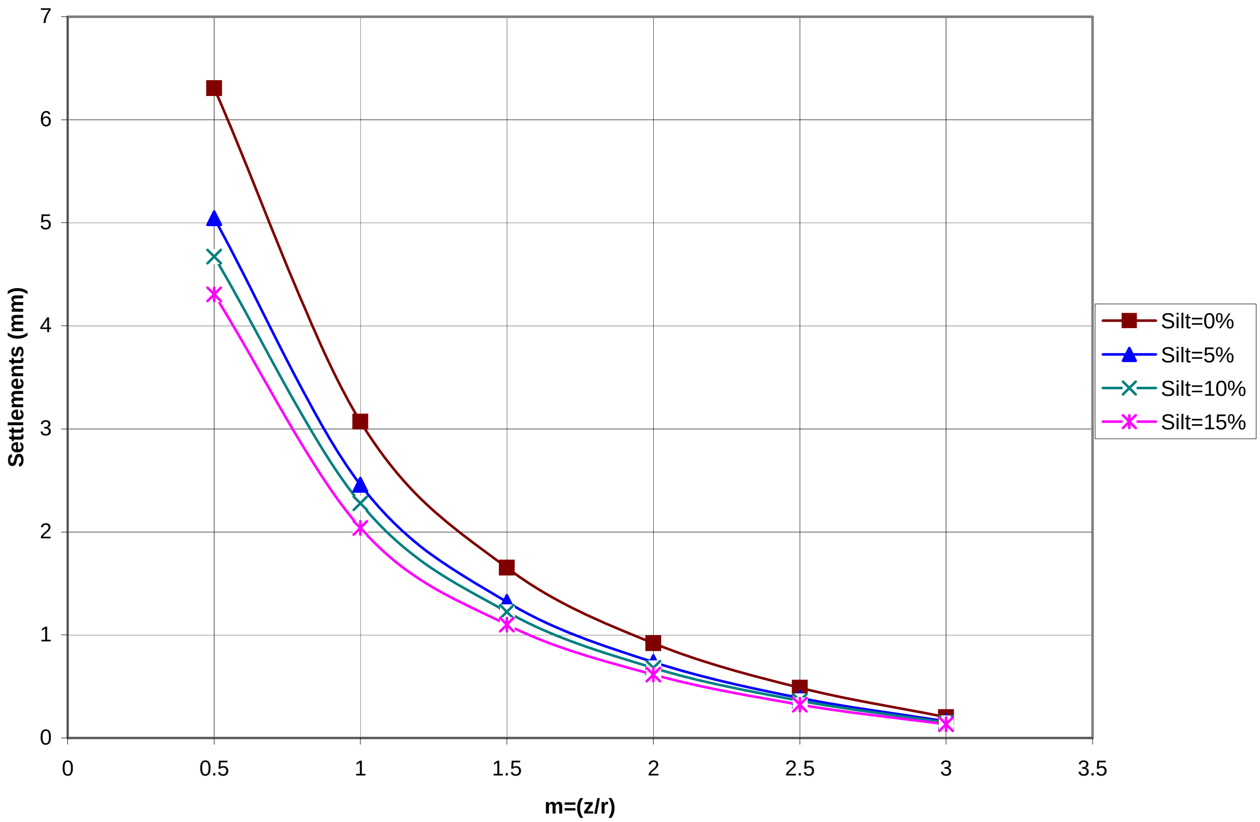


Fig.7.18 Vertical settlements vs. Silt (%) for soil type B (r=0.5m)

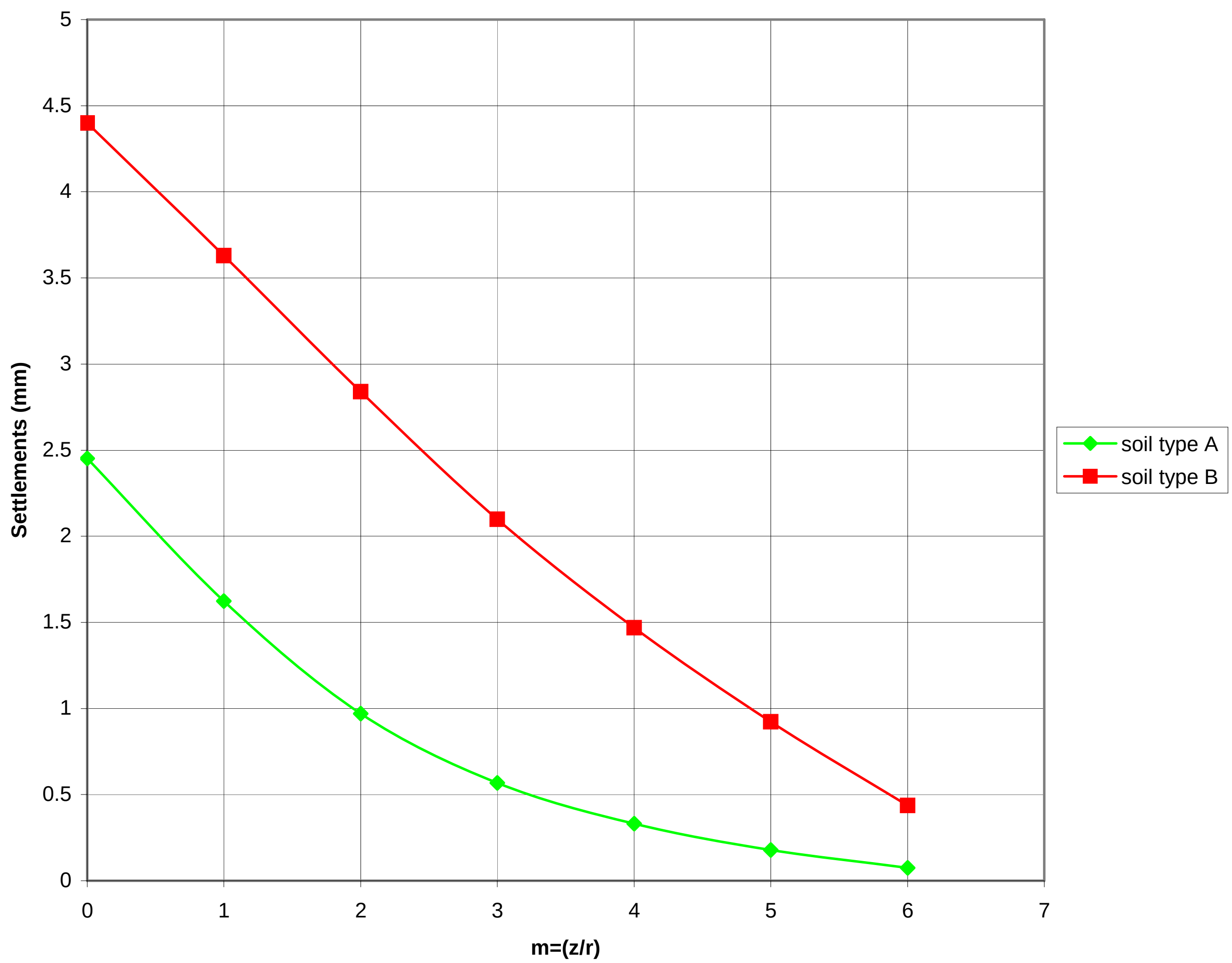


Fig.7.19 Vertical Settlement vs. Depth for soil type A & B (r=0.50m)

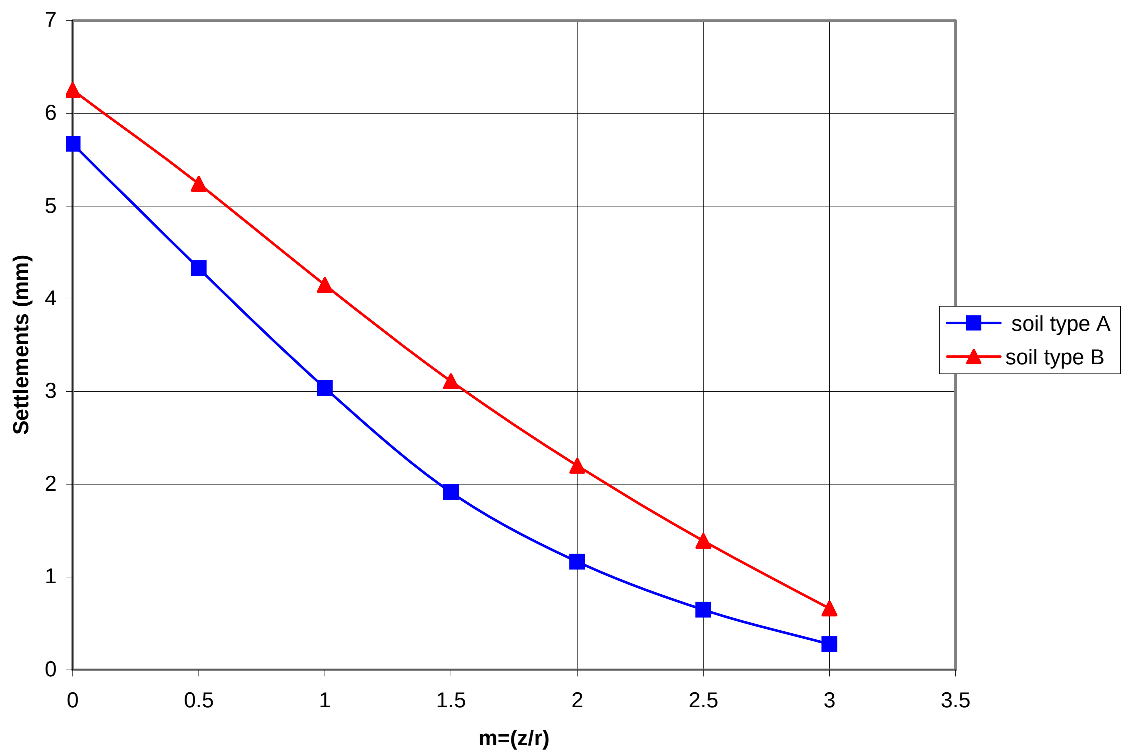


Fig.7.20 Vertical Settlement vs. Depth for soil type A & B (r=1.0m)

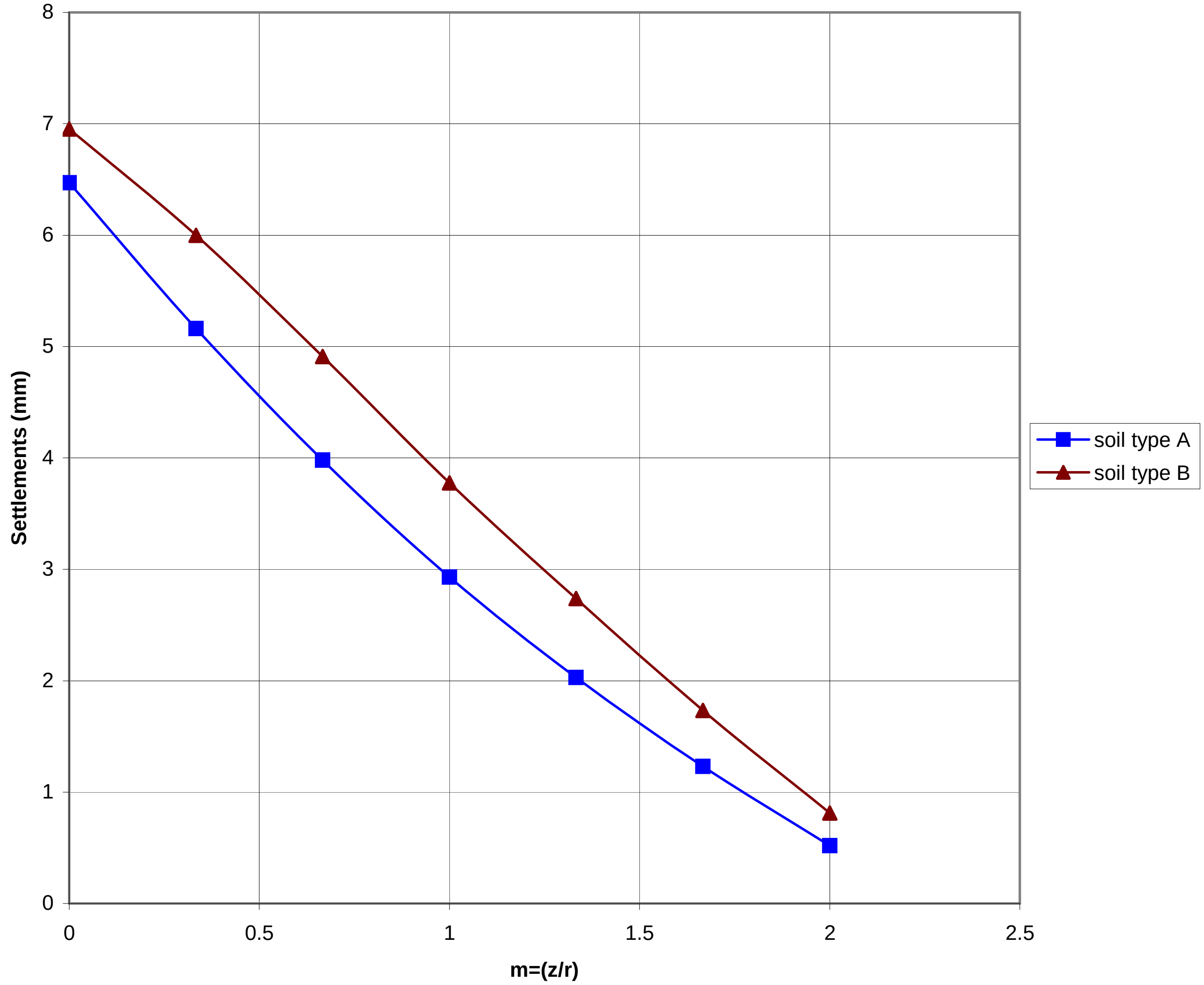


Fig.7.21 Vertical Settlement vs. Depth for soil type A & B (r=1.5m)

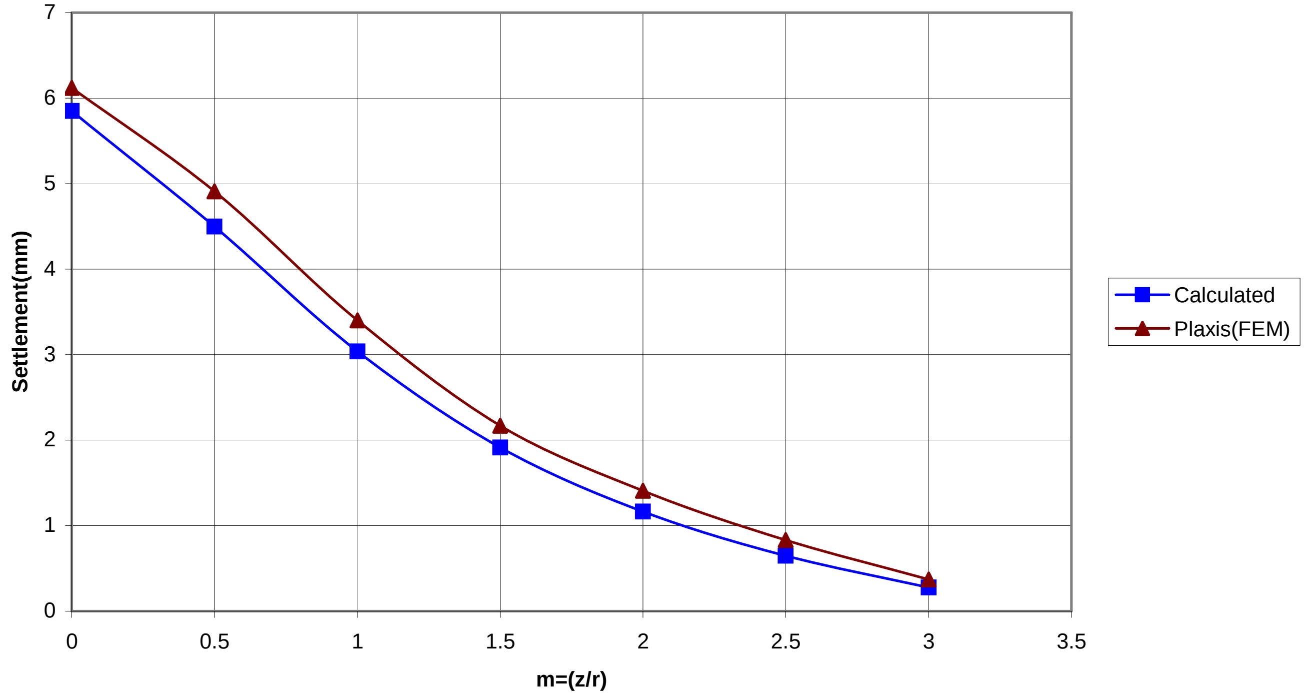


Fig. 7.22 Vertical Settlement vs. Depth for soil type A (r=1.0m)

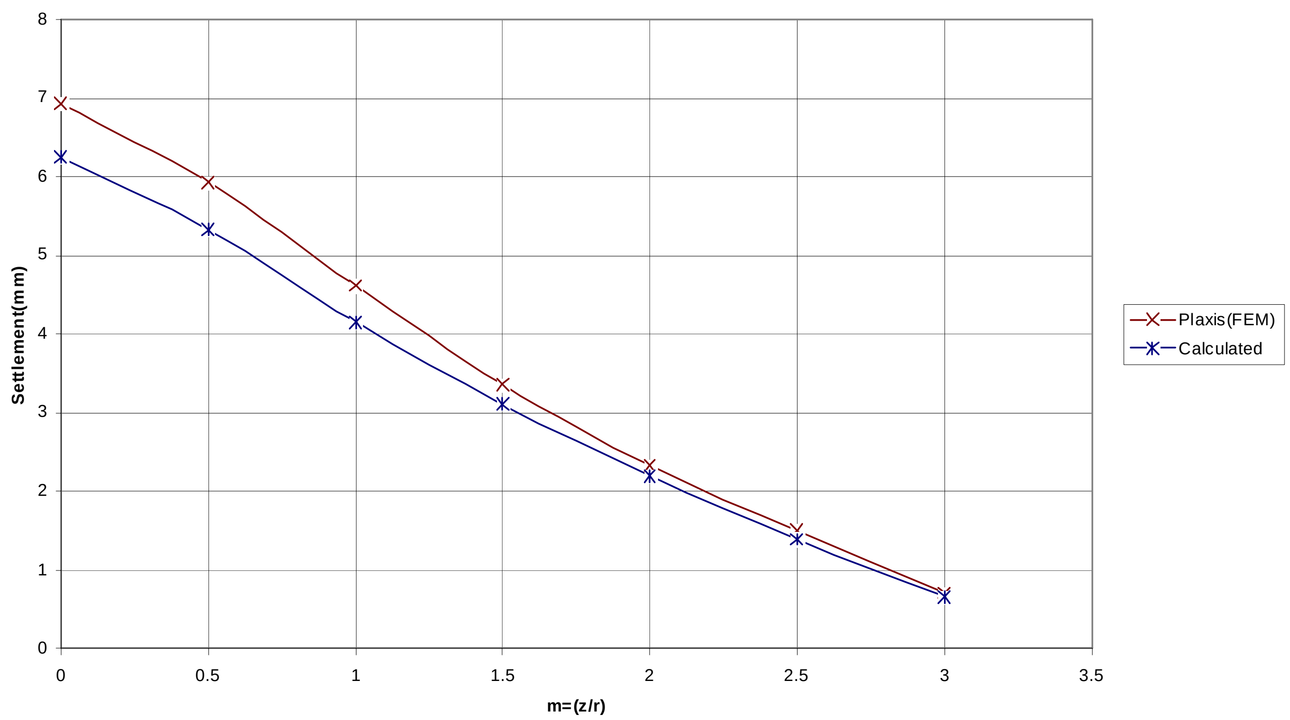


Fig. 7.23 Vertical Settlement vs. Depth for soil type B (r=1.0m)

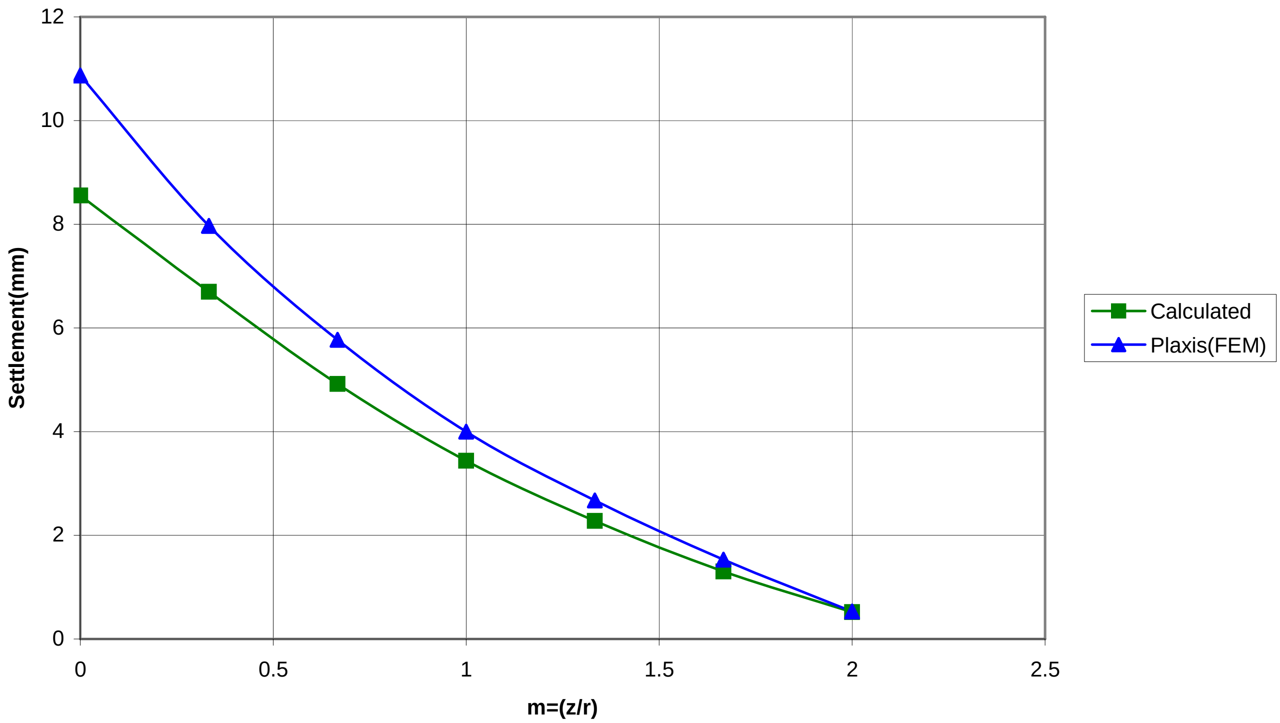


Fig. 6.24 Vertical Settlement vs. Depth for soil type A (r=1.5m)

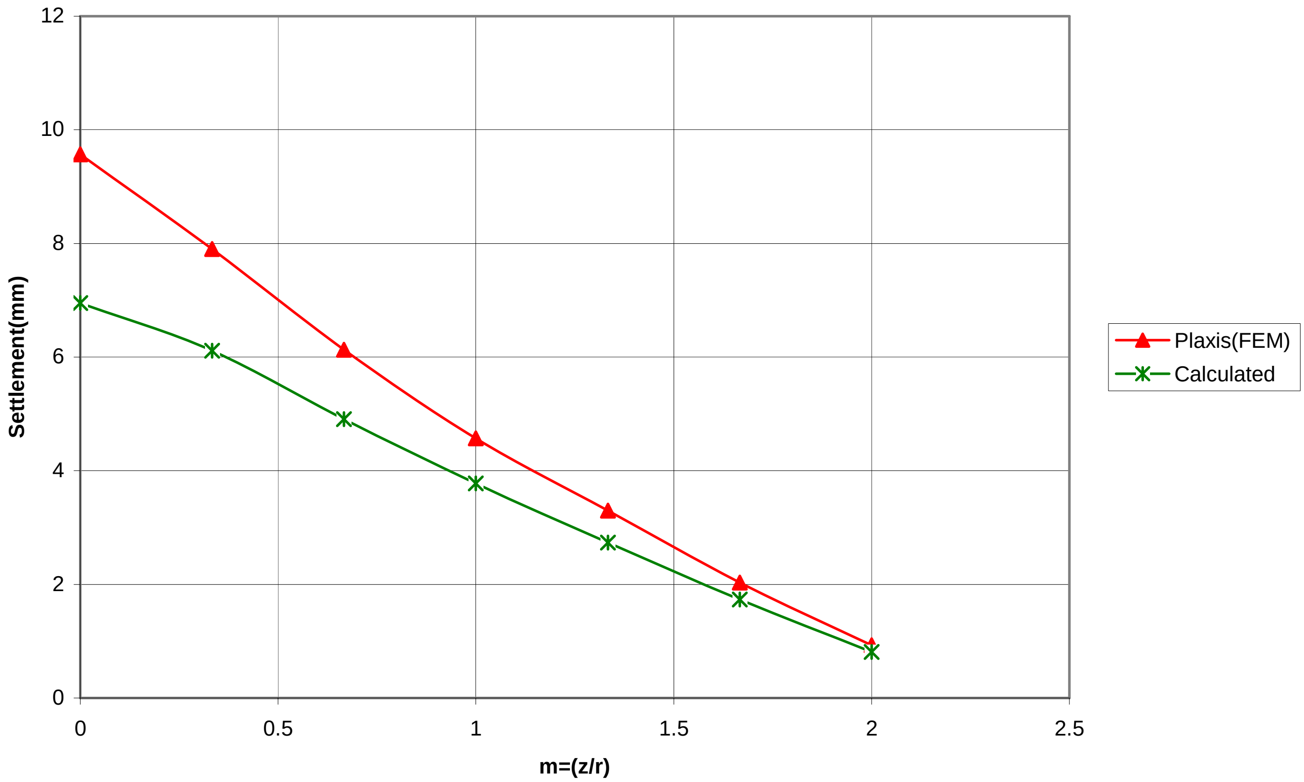


Fig. 7.25 Vertical Settlement vs. Depth for soil type B (r=1.5m)

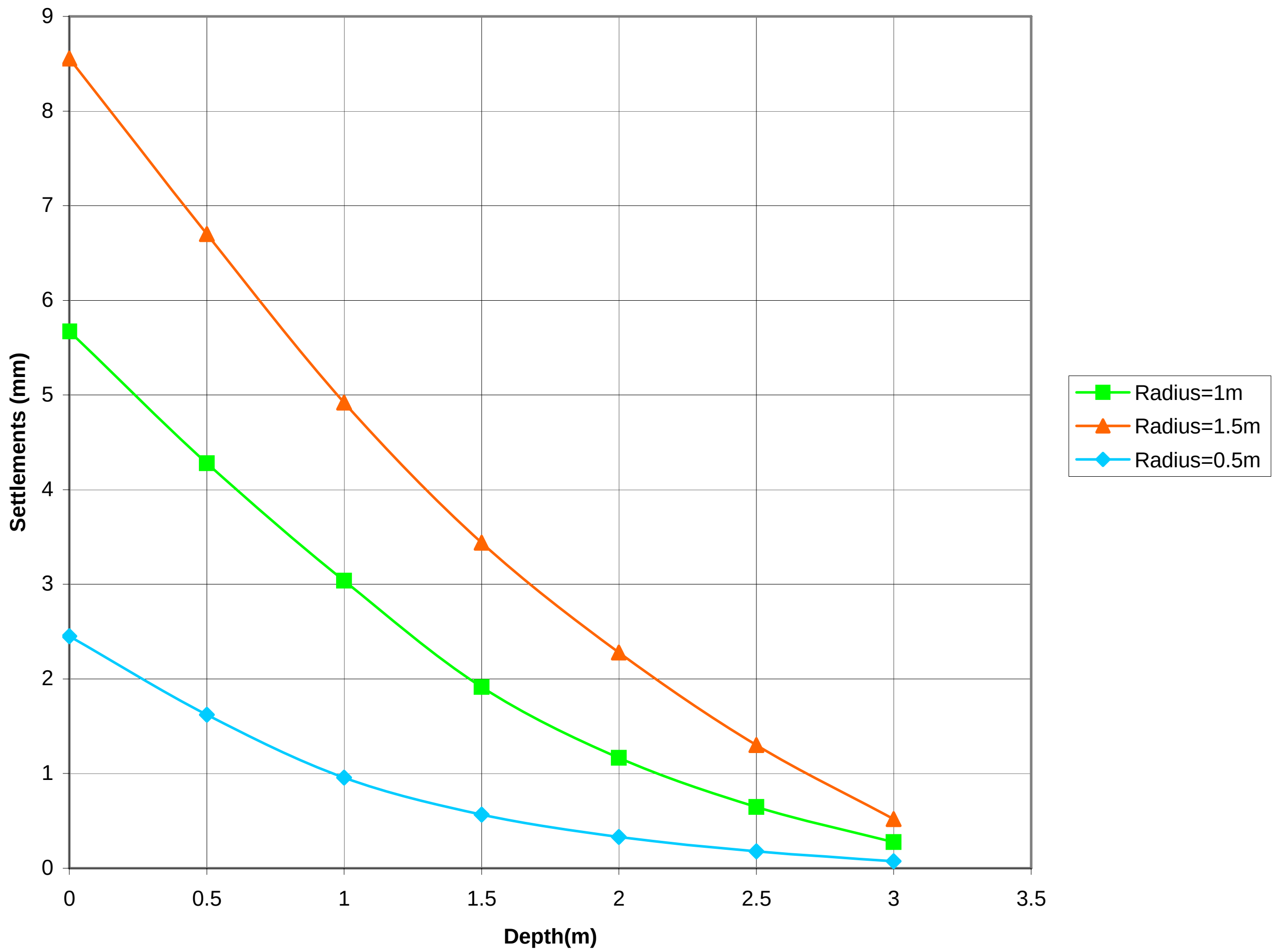


Fig. 7.26 Vertical Settlement vs. Depth for soil type A

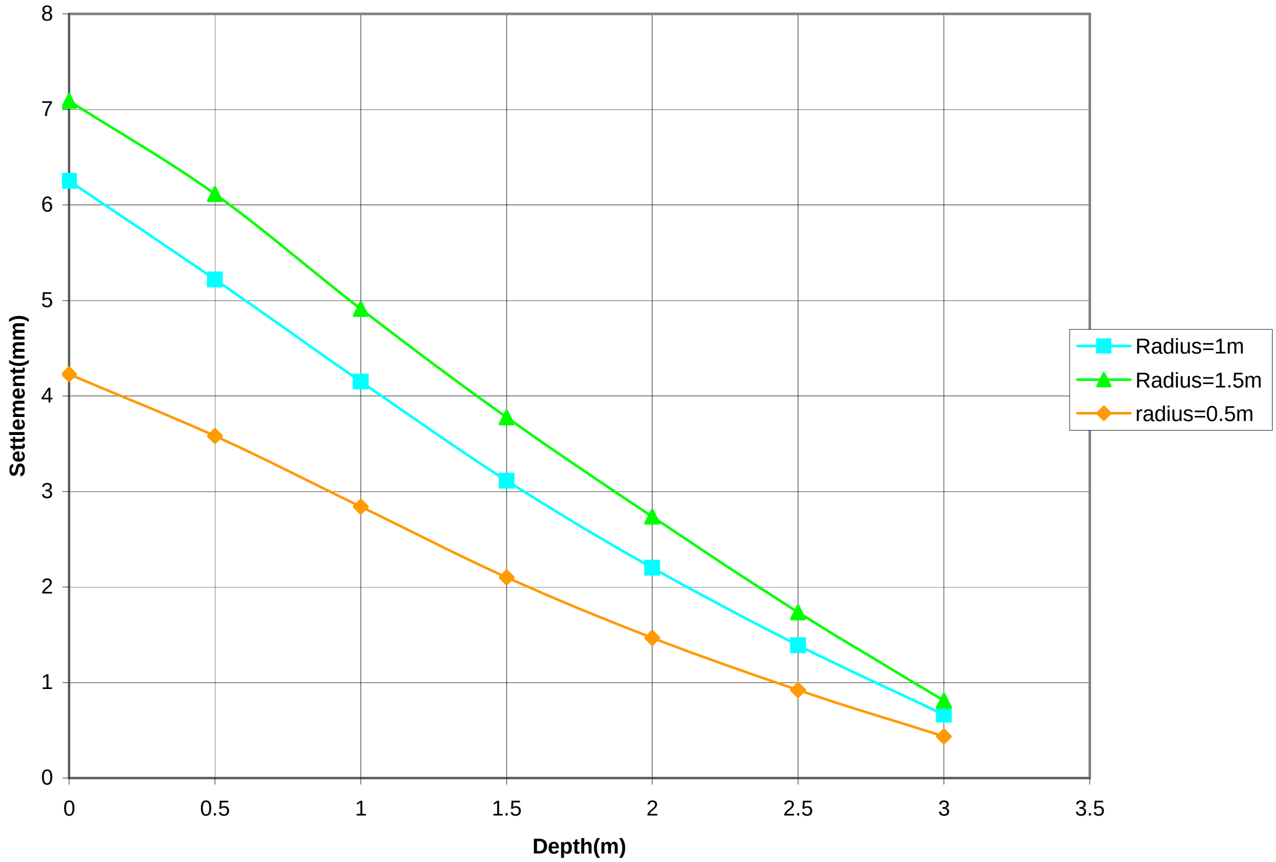


Fig.7.27 Vertical Settlement vs. Depth for soil type B

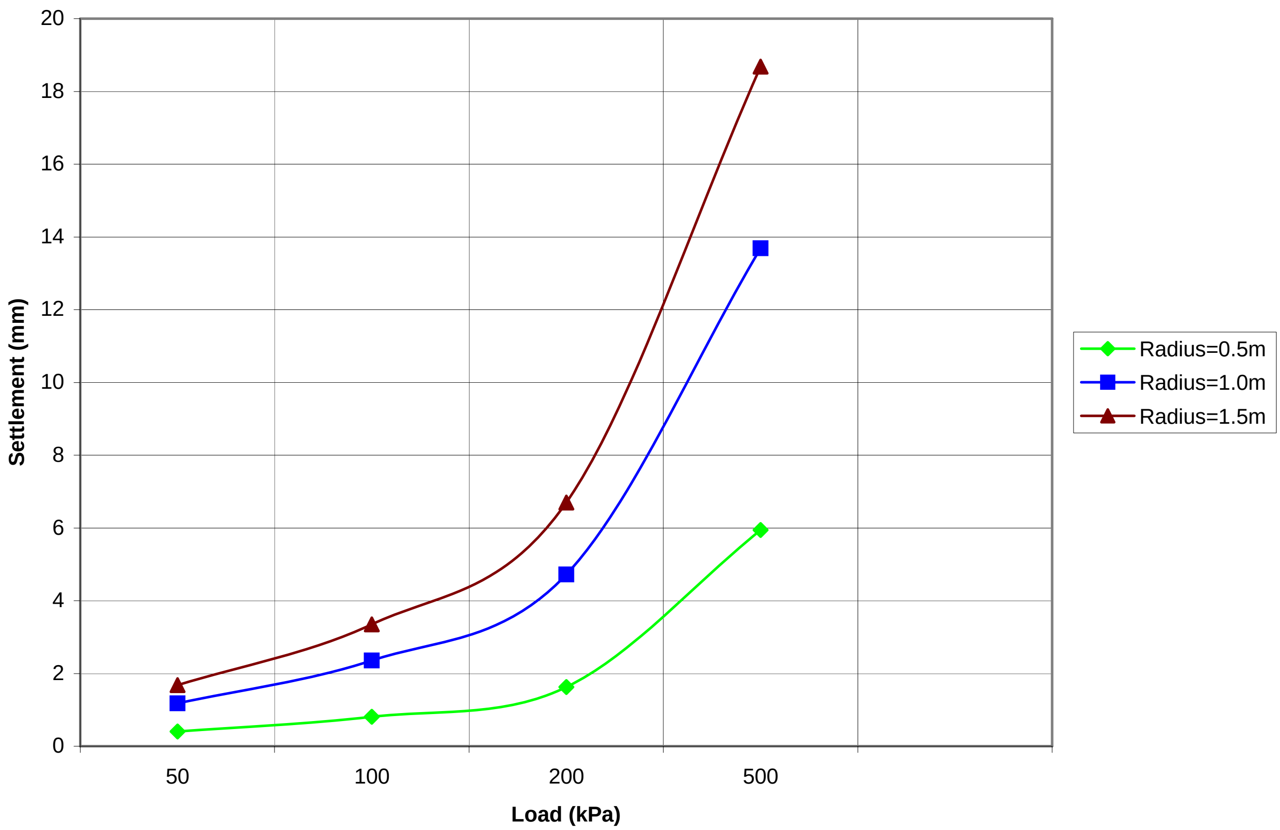


Fig.7.28 Load vs. Settlement for different loading for soil type A

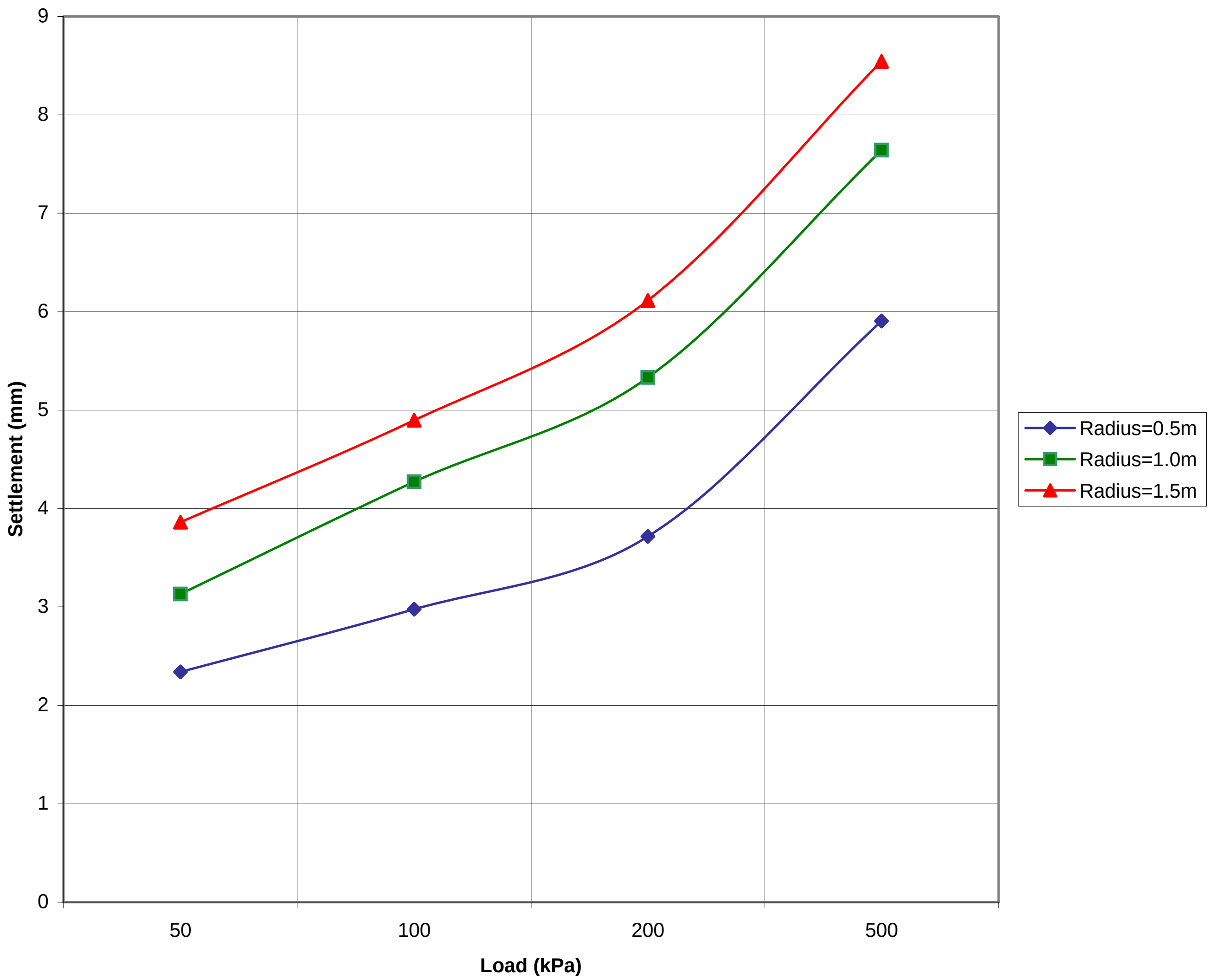


Fig. 7.29 Load vs. Settlement for different loading for soil type B

Table 7.30 Soil data sets parameters

Linear Elastic		Sand 01
Type		Drained
γ_{unsat}	[kN/m³]	18.00
γ_{sat}	[kN/m³]	20.00
k_x	[m/day]	0.010
k_y	[m/day]	0.010
e_{init}	[-]	0.500
c_k	[-]	1E15
E_{ref}	[kN/m²]	50000.00
ν	[-]	0.300
G_{ref}	[kN/m²]	19230.769
E_{oed}	[kN/m²]	67307.692
E_{incr}	[kN/m²/m]	0.00
y_{ref}	[m]	0.000
R_{inter}	[-]	1.000
Interface permeability		Neutral

Table 7.31 Table of total stresses for soil type A

Soil element	x-coord.	y-coord.	σ_{xx}	σ_{yy}	σ_{xy}	σ_{zz}
	[m]	[m]	[kN/m²]	[kN/m²]	[kN/m²]	[kN/m²]
1	0.032	0.437	-48.207	-60.573	0.106	-48.208
	0.063	0.468	-106.373	-125.921	0.206	-106.373
	0.500	0.500	-164.313	-190.628	1.519	-164.293
2	0.984	0.322	-51.035	-58.240	2.264	-50.353
	1.548	0.345	-109.038	-122.646	3.910	-108.331
	2.517	0.368	-166.390	-185.327	6.292	-165.625
3	1.047	0.322	-51.034	-57.853	2.318	-50.294
	2.079	0.345	-108.270	-120.227	4.769	-107.466
	3.517	0.368	-164.719	-180.569	7.396	-163.825
4	2.002	0.323	-50.512	-52.448	2.013	-49.351
	3.567	0.347	-106.716	-112.169	4.608	-105.459
	5.538	0.370	-162.207	-170.112	6.914	-160.850
5	2.065	0.323	-50.455	-52.177	1.948	-49.299
	4.099	0.347	-105.848	-109.876	4.190	-104.592
	6.538	0.370	-160.694	-166.250	5.959	-159.345
6	2.957	0.214	-51.408	-51.669	1.010	-50.791
	5.519	0.229	-106.239	-107.868	2.633	-105.564
	8.487	0.245	-160.722	-163.251	3.748	-159.997
7	3.020	0.214	-51.363	-51.568	0.941	-50.764
	6.051	0.229	-105.804	-106.854	1.980	-105.157
	9.487	0.245	-160.049	-161.689	2.552	-159.363
8	3.968	0.437	-47.328	-46.686	0.023	-46.651
	7.532	0.468	-101.285	-101.130	0.455	-100.535
	11.500	0.500	-155.182	-155.450	0.485	-154.372
9	3.650	0.995	-37.831	-36.356	0.193	-36.551
	7.625	1.560	-83.006	-80.644	0.210	-80.915
	11.599	2.531	-121.154	-117.391	0.224	-117.987
10	3.650	1.058	-36.707	-35.187	0.184	-35.406
	7.625	2.092	-73.732	-70.765	0.197	-71.337
	11.599	3.531	-103.363	-98.879	0.205	-99.892
11	3.679	1.997	-19.055	-17.984	0.015	-18.252
	7.656	3.562	-46.320	-43.788	0.021	-44.496
	11.633	5.533	-65.812	-62.234	0.022	-63.284
12	3.679	2.060	-17.814	-16.845	0.005	-17.092
	7.656	4.094	-36.073	-34.153	0.005	-34.716
	11.633	6.533	-46.311	-44.192	0.002	-44.837
13	3.563	2.968	0.457	-0.567	-0.010	-0.367
	7.532	5.532	-7.303	-8.389	-0.014	-8.188
	11.500	8.500	-6.987	-8.957	-0.015	-8.497
14	3.047	2.704	-4.782	-5.315	-0.110	-5.290
	6.485	5.683	-4.045	-5.700	-0.119	-5.488
	9.517	8.661	-2.948	-6.084	-0.134	-5.763
15	2.984	2.704	-4.768	-5.318	-0.114	-5.300
	5.953	5.683	-3.598	-5.702	-0.130	-5.593
	8.517	8.661	-1.826	-6.086	-0.151	-6.053
16	2.059	2.672	-5.811	-5.994	0.131	-6.266

Soil element	x-coord.	y-coord.	σ_{xx}	σ_{yy}	σ_{xy}	σ_{zz}
	[m]	[m]	[kN/m²]	[kN/m²]	[kN/m²]	[kN/m²]
	4.498	5.649	-3.858	-6.419	0.107	-6.851
	6.532	8.625	-0.923	-6.845	0.081	-7.905
17	1.996	2.672	-5.922	-6.018	0.187	-6.323
	3.967	5.649	-2.799	-6.443	0.163	-7.489
	5.532	8.625	1.966	-6.863	0.125	-9.838
18	1.030	2.668	-18.296	-8.708	7.792	-8.607
	2.467	5.644	-12.849	-9.075	7.798	-11.595
	3.498	8.620	-4.250	-9.379	8.035	-18.729
19	0.967	2.668	-22.463	-11.873	9.365	-9.835
	1.935	5.644	-13.751	-12.295	9.565	-18.373
	2.498	8.620	-28.931	-24.841	29.730	-56.500
20	0.032	2.563	-10.873	-147.341	1.048	-9.180
	0.468	5.532	-105.009	-331.401	26.442	-125.907
	0.500	8.500	-247.772	-537.327	21.595	-270.148
21	0.302	2.038	-18.392	-70.338	12.123	-15.874
	0.324	4.475	-33.382	-193.814	13.345	-30.903
	0.345	6.507	-49.177	-271.668	14.316	-46.680
22	0.302	1.975	-18.976	-67.258	10.761	-16.850
	0.324	3.944	-35.809	-140.922	11.600	-33.671
	0.345	5.507	-60.245	-199.771	12.014	-58.098
23	0.322	1.046	-35.461	-54.669	2.303	-35.092
	0.345	2.484	-62.475	-111.689	2.634	-62.101
	0.368	3.517	-97.818	-167.507	2.806	-97.442
24	0.322	0.983	-36.727	-54.995	2.096	-36.397
	0.345	1.953	-73.384	-111.053	2.252	-73.053
	0.368	2.517	-118.720	-170.252	2.343	-118.387
25	0.600	0.526	-46.570	-57.759	1.937	-46.086
	1.137	0.586	-103.703	-121.798	3.480	-103.164
	2.094	0.944	-153.996	-179.671	5.747	-152.738
26	3.427	2.516	-8.730	-8.684	-0.074	-8.710
	6.893	5.461	-8.690	-9.670	-0.094	-9.519
	9.968	8.132	-14.189	-15.585	-0.204	-15.415
27	1.564	0.585	-46.222	-50.187	2.547	-44.438
	3.100	0.649	-101.751	-109.372	5.085	-99.716
	5.072	1.013	-151.608	-161.223	7.118	-148.293
28	3.361	0.600	-44.824	-43.904	0.478	-43.672
	7.291	1.139	-90.460	-88.695	0.527	-88.506
	10.896	2.107	-128.787	-125.567	0.751	-125.533
29	0.499	2.416	-25.448	-71.571	31.184	-12.471
	0.555	4.881	-42.528	-201.874	35.555	-29.545
	0.892	6.947	-61.119	-271.785	49.400	-44.971
30	0.970	0.926	-39.536	-49.669	3.800	-37.676
	1.601	1.515	-84.841	-106.470	5.938	-82.393
	2.589	1.936	-133.889	-163.224	8.309	-130.577
31	3.122	2.208	-15.147	-14.270	0.022	-14.337
	6.524	4.671	-24.969	-23.910	-0.048	-24.024
	9.574	7.288	-31.598	-30.784	-0.157	-30.895
32	2.651	0.580	-45.692	-45.448	1.195	-44.128
	5.190	0.637	-99.922	-100.945	2.806	-98.148

Soil element	x-coord.	y-coord.	σ xx	σ yy	σ xy	σ zz
	[m]	[m]	[kN/m²]	[kN/m²]	[kN/m²]	[kN/m²]
	8.125	0.891	-150.699	-151.889	3.820	-148.200
33	2.022	0.877	-41.200	-42.184	2.161	-38.682
	3.621	1.521	-86.410	-91.118	4.749	-81.940
	5.628	1.943	-135.256	-141.750	6.740	-129.342
34	3.108	1.008	-37.962	-36.553	0.588	-36.241
	6.410	1.643	-82.249	-79.879	1.108	-79.298
	9.956	2.645	-120.006	-116.152	1.363	-115.699
35	3.290	0.519	-46.225	-45.492	0.560	-45.144
	6.329	0.770	-96.997	-96.348	1.463	-95.235
	9.784	0.821	-150.672	-150.481	2.004	-148.784
36	0.831	2.090	-25.679	-38.769	15.764	-15.612
	1.386	4.479	-51.770	-101.317	45.175	-27.876
	1.779	6.517	-71.587	-166.722	59.678	-43.744
37	1.499	0.585	-46.212	-50.616	2.605	-44.468
	2.580	0.948	-96.442	-107.643	4.978	-93.844
	4.052	1.012	-152.082	-167.154	7.531	-149.241
38	0.421	0.987	-36.874	-54.294	2.647	-36.326
	1.011	1.617	-81.282	-110.856	4.781	-80.172
	1.940	2.584	-119.996	-160.401	8.748	-117.018
39	3.588	2.068	-17.691	-16.712	0.008	-16.935
	7.029	4.491	-28.314	-27.061	-0.053	-27.350
	10.190	6.660	-44.240	-42.043	-0.021	-42.413
40	1.467	0.648	-45.106	-49.768	2.730	-43.226
	2.498	1.578	-84.711	-98.762	6.558	-80.827
	3.547	2.004	-133.701	-155.052	8.981	-128.876
41	2.611	2.350	-12.692	-11.839	0.137	-11.774
	5.676	4.550	-28.050	-26.274	0.179	-26.257
	8.669	7.160	-34.856	-33.297	0.070	-33.283
42	2.591	0.588	-45.611	-45.456	1.264	-43.996
	4.696	0.952	-95.354	-96.725	3.157	-92.470
	7.176	1.016	-149.527	-152.233	4.833	-146.399
43	3.236	0.560	-45.578	-44.787	0.597	-44.408
	5.937	1.177	-90.619	-89.431	1.724	-87.851
	8.922	1.468	-140.778	-139.608	2.666	-137.215
44	1.622	1.146	-36.869	-39.676	3.498	-33.541
	3.192	1.842	-81.174	-87.882	6.197	-75.774
	5.187	2.771	-121.514	-129.223	8.451	-113.470
45	2.563	0.647	-44.655	-44.434	1.286	-42.911
	4.655	1.524	-85.809	-86.274	3.315	-81.585
	6.732	1.947	-134.598	-136.593	5.228	-128.938
46	2.706	0.667	-44.197	-43.694	1.109	-42.516
	5.946	1.277	-88.932	-87.540	1.688	-86.015
	8.994	2.261	-127.373	-124.628	2.340	-122.693
47	1.620	2.237	-19.183	-16.099	3.528	-13.828
	3.670	4.462	-36.109	-30.943	4.841	-27.882
	5.675	7.034	-44.772	-38.877	5.263	-35.918
48	3.250	1.502	-28.857	-27.201	0.306	-27.251
	6.355	2.573	-65.713	-62.585	0.878	-62.348
	9.898	3.639	-102.356	-97.697	1.125	-97.611

Soil element	x-coord.	y-coord.	σ_{xx}	σ_{yy}	σ_{xy}	σ_{zz}
	[m]	[m]	[kN/m²]	[kN/m²]	[kN/m²]	[kN/m²]
49	0.963	2.574	-23.872	-15.488	12.273	-9.760
	1.559	5.002	-51.710	-72.743	42.608	-21.807
	2.432	7.130	-77.616	-108.712	58.175	-36.889
50	0.636	1.597	-27.410	-49.081	8.480	-24.232
	1.473	3.629	-53.110	-88.375	23.222	-40.779
	1.871	5.610	-73.494	-151.378	36.366	-57.614
51	0.060	0.537	-45.944	-59.466	0.224	-45.937
	0.587	1.136	-90.909	-116.761	2.119	-90.464
	0.945	2.092	-128.253	-171.695	4.331	-127.423
52	3.942	2.469	-9.647	-9.509	-0.008	-9.571
	7.439	4.913	-19.820	-19.465	-0.068	-19.588
	11.084	7.003	-37.060	-35.779	-0.066	-36.131
53	2.759	1.913	-21.544	-19.951	0.454	-19.694
	5.807	4.063	-37.945	-35.312	0.529	-35.094
	8.401	6.363	-51.756	-48.099	0.734	-47.776
54	2.536	2.937	0.831	-1.132	-0.056	-1.228
	5.108	5.340	-10.743	-11.997	0.051	-12.049
	8.063	8.003	-16.403	-18.063	-0.063	-18.111
55	1.067	0.983	-38.793	-47.858	4.046	-36.564
	2.571	1.683	-83.006	-96.467	6.831	-78.738
	4.126	2.833	-119.797	-136.633	10.541	-112.185
56	1.985	1.320	-33.814	-33.802	2.555	-30.372
	3.636	2.511	-69.965	-72.380	6.054	-63.067
	5.661	3.485	-109.532	-112.710	8.279	-99.913
57	2.566	1.300	-33.363	-32.019	1.162	-30.848
	5.223	2.017	-76.762	-74.896	2.318	-72.450
	8.222	3.051	-114.369	-111.121	3.003	-108.215
58	1.949	1.954	-22.882	-20.850	2.371	-18.832
	3.995	4.134	-40.796	-36.623	3.818	-33.671
	5.611	6.326	-60.927	-53.821	7.579	-48.255
59	2.603	1.358	-32.266	-30.809	1.075	-29.786
	5.640	2.451	-68.807	-65.885	1.704	-64.490
	8.823	3.974	-97.329	-92.745	2.047	-91.343
60	1.537	2.927	2.607	-1.338	-0.008	-3.228
	3.121	5.214	-15.793	-16.436	3.573	-16.207
	5.089	7.837	-23.176	-23.401	3.913	-23.400
61	3.927	1.475	-28.975	-27.458	0.022	-27.886
	7.236	3.006	-57.233	-54.079	0.282	-54.614
	10.838	4.100	-93.322	-88.621	0.488	-89.359
62	0.534	2.941	-42.779	-59.071	37.208	-48.271
	1.106	5.427	-72.532	-120.702	71.148	-60.499
	2.044	8.060	-94.913	-133.089	83.709	-69.247
63	1.177	1.634	-29.610	-35.916	7.391	-24.101
	2.070	3.641	-55.617	-73.101	20.985	-41.162
	2.763	5.213	-83.768	-120.803	29.338	-65.931
64	0.062	1.534	-25.142	-58.251	1.062	-25.083
	0.642	3.095	-52.619	-108.729	8.844	-49.960
	0.984	5.040	-72.527	-173.023	19.979	-67.344
65	2.330	1.935	-21.987	-20.124	1.125	-19.236

Soil element	x-coord.	y-coord.	σ_{xx}	σ_{yy}	σ_{xy}	σ_{zz}
	[m]	[m]	[kN/m²]	[kN/m²]	[kN/m²]	[kN/m²]
	4.833	4.239	-35.866	-32.889	1.408	-31.854
	6.965	6.425	-53.312	-48.360	2.575	-46.584
66	3.200	1.625	-26.577	-24.955	0.286	-24.994
	6.291	3.727	-43.927	-41.192	0.374	-41.284
	9.094	5.591	-66.366	-62.020	0.824	-61.869
67	2.372	1.914	-22.294	-20.458	1.060	-19.621
	5.082	3.810	-44.255	-40.782	1.584	-39.619
	7.627	6.093	-58.515	-53.916	1.853	-52.610
68	1.754	1.608	-29.482	-29.396	3.640	-24.998
	3.386	2.844	-64.911	-67.255	7.299	-56.824
	5.351	4.210	-97.986	-100.251	9.939	-86.344
69	2.293	1.909	-22.612	-20.734	1.254	-19.713
	4.388	4.068	-40.768	-36.822	2.594	-34.916
	6.386	6.001	-63.850	-57.914	4.787	-54.127
70	2.123	0.940	-40.075	-40.506	2.003	-37.490
	4.716	1.650	-83.644	-83.676	3.239	-79.227
	7.219	2.943	-117.214	-115.989	4.515	-110.192
71	1.749	1.707	-27.899	-27.316	3.686	-23.188
	3.677	3.626	-51.505	-48.991	6.220	-42.623
	5.272	5.784	-72.455	-67.161	10.299	-57.780
72	2.766	1.814	-23.472	-21.809	0.533	-21.488
	5.350	3.225	-54.825	-51.659	1.616	-50.316
	8.513	4.800	-82.383	-77.566	1.948	-76.215
73	1.495	2.165	-21.614	-18.863	5.023	-15.013
	2.446	4.215	-47.613	-52.822	18.242	-31.420
	3.681	5.892	-76.785	-86.876	25.466	-54.792
74	0.988	1.065	-37.210	-47.580	4.462	-34.909
	2.172	2.631	-67.653	-84.608	11.405	-60.280
	2.871	4.135	-96.788	-132.446	18.997	-86.384
75	0.471	2.933	-66.044	-125.063	43.994	-73.237
	0.537	5.461	-90.576	-276.024	52.176	-92.808
	1.046	7.940	-117.780	-348.477	86.951	-105.166
76	2.086	1.620	-28.510	-27.248	2.206	-24.889
	3.872	3.255	-57.472	-55.823	5.686	-49.396
	5.871	4.649	-90.038	-88.087	8.226	-78.414
77	2.527	1.431	-31.075	-29.595	1.174	-28.453
	5.236	3.265	-54.249	-51.091	1.758	-49.569
	7.606	5.118	-77.749	-72.768	2.902	-70.302
78	2.120	1.670	-27.539	-26.086	2.053	-23.989
	4.415	3.544	-50.834	-47.523	3.352	-44.340
	6.415	5.441	-74.574	-69.354	5.597	-64.183
79	2.083	1.656	-27.891	-26.534	2.193	-24.244
	4.046	3.539	-52.035	-48.828	4.622	-44.339
	5.830	5.209	-80.429	-76.671	8.119	-68.202
80	3.934	2.530	-8.428	-8.419	-0.009	-8.441
	7.462	5.465	-8.627	-9.587	-0.030	-9.428
	10.951	7.970	-17.560	-18.451	-0.097	-18.323
81	0.068	0.473	-47.383	-60.147	0.238	-47.377
	0.541	0.541	-104.372	-124.295	1.615	-104.316

Soil element	x-coord.	y-coord.	σ_{xx}	σ_{yy}	σ_{xy}	σ_{zz}
	[m]	[m]	[kN/m²]	[kN/m²]	[kN/m²]	[kN/m²]
	1.077	1.075	-150.698	-182.304	3.407	-150.240
82	3.519	0.068	-53.403	-53.784	0.469	-53.243
	7.443	0.541	-100.133	-99.791	0.524	-99.239
	10.797	1.077	-146.051	-144.918	1.024	-144.088
83	2.487	1.416	-31.396	-29.974	1.257	-28.709
	4.817	3.255	-55.272	-52.024	2.513	-49.695
	6.972	4.890	-83.351	-78.684	4.476	-74.326
84	1.289	1.651	-29.488	-33.661	6.688	-23.896
	2.992	3.338	-57.819	-61.746	10.639	-47.412
	4.541	5.477	-79.476	-80.766	15.212	-62.887
85	2.511	2.401	-11.714	-10.933	0.153	-10.871
	4.984	5.336	-10.842	-12.107	0.095	-12.180
	7.078	7.967	-17.716	-18.874	0.271	-19.139
86	2.485	2.350	-12.886	-11.896	0.239	-11.783
	4.552	4.930	-21.160	-19.639	0.550	-19.638
	6.664	7.163	-37.660	-34.201	1.656	-33.545
87	3.312	1.595	-27.063	-25.452	0.234	-25.574
	7.241	3.133	-54.852	-51.753	0.254	-52.303
	10.871	5.103	-74.459	-70.235	0.279	-71.048
88	3.255	1.636	-26.324	-24.721	0.248	-24.808
	6.830	3.648	-45.142	-42.462	0.270	-42.785
	9.977	5.759	-62.228	-58.486	0.334	-58.884
89	0.582	1.498	-28.423	-50.414	7.074	-26.101
	0.645	2.970	-54.838	-107.797	8.041	-52.482
	1.007	4.049	-89.729	-162.030	10.722	-86.890
90	0.644	1.466	-29.327	-49.159	7.023	-26.787
	1.069	2.514	-65.025	-103.043	9.939	-61.868
	2.003	3.541	-102.707	-151.757	14.204	-97.504
91	1.518	2.287	-19.086	-15.577	4.196	-13.014
	2.990	5.214	-16.305	-16.939	4.233	-16.535
	4.055	7.832	-34.552	-27.581	12.233	-25.328
92	1.480	2.229	-20.534	-17.325	4.896	-13.920
	2.506	4.788	-42.302	-31.664	15.286	-23.606
	3.442	6.902	-68.171	-64.907	29.467	-38.972
93	2.044	1.331	-33.559	-33.286	2.377	-30.190
	4.126	2.315	-72.907	-73.123	4.483	-66.837
	6.589	3.654	-105.737	-104.678	5.819	-96.973
94	2.047	1.375	-32.800	-32.379	2.376	-29.367
	4.514	2.758	-64.828	-63.057	3.687	-58.686
	6.648	4.360	-93.549	-90.474	5.738	-83.920
95	1.583	1.253	-35.182	-37.972	3.867	-31.509
	3.289	2.876	-64.514	-67.406	7.774	-56.192
	4.580	4.463	-94.810	-102.295	14.163	-81.271
96	1.536	1.211	-35.824	-39.194	3.927	-32.283
	2.780	2.756	-66.621	-75.618	10.402	-58.102
	3.829	3.800	-104.343	-122.793	14.740	-93.448

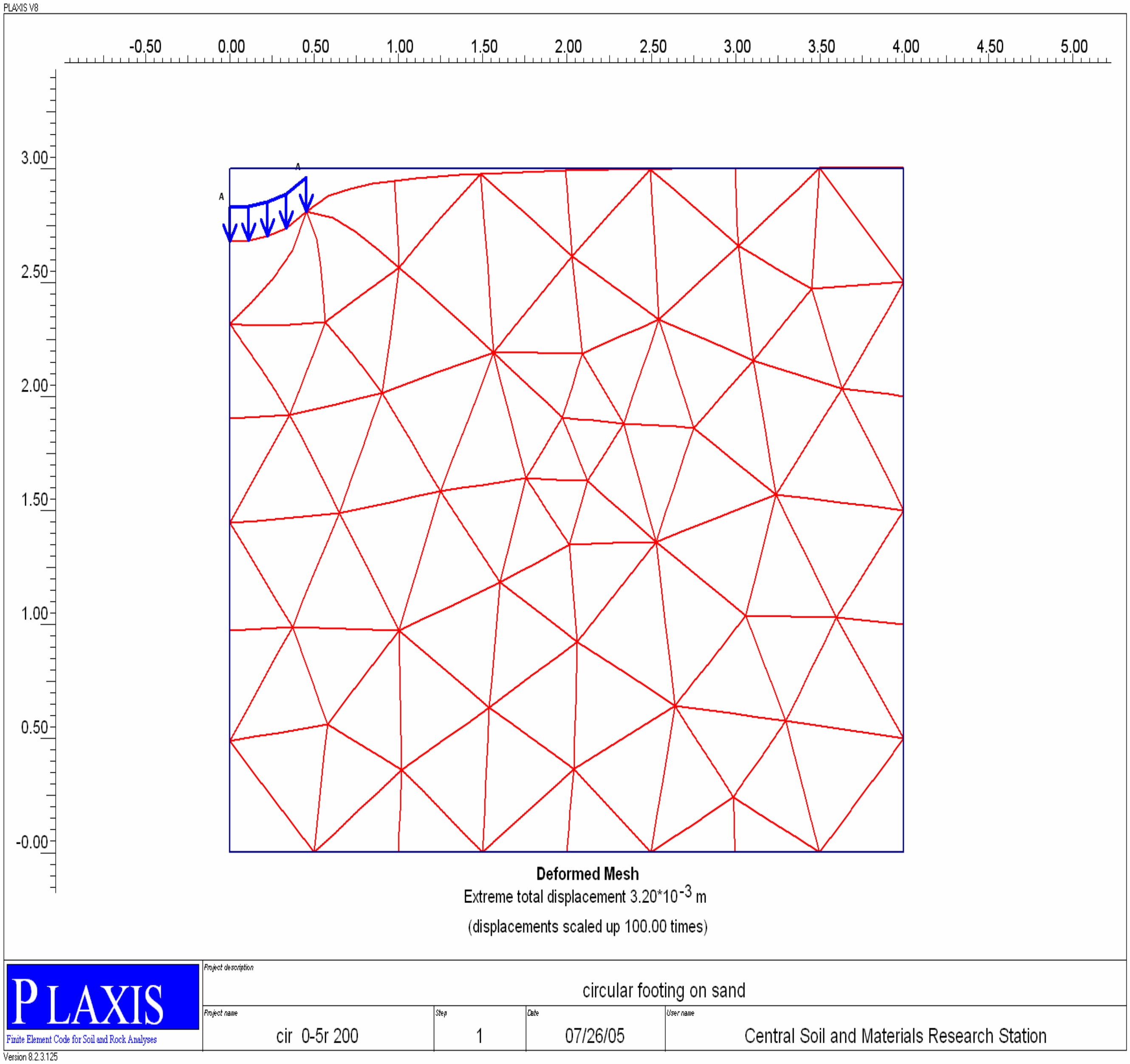


Fig. 7.32 Problem geometry with mesh generation

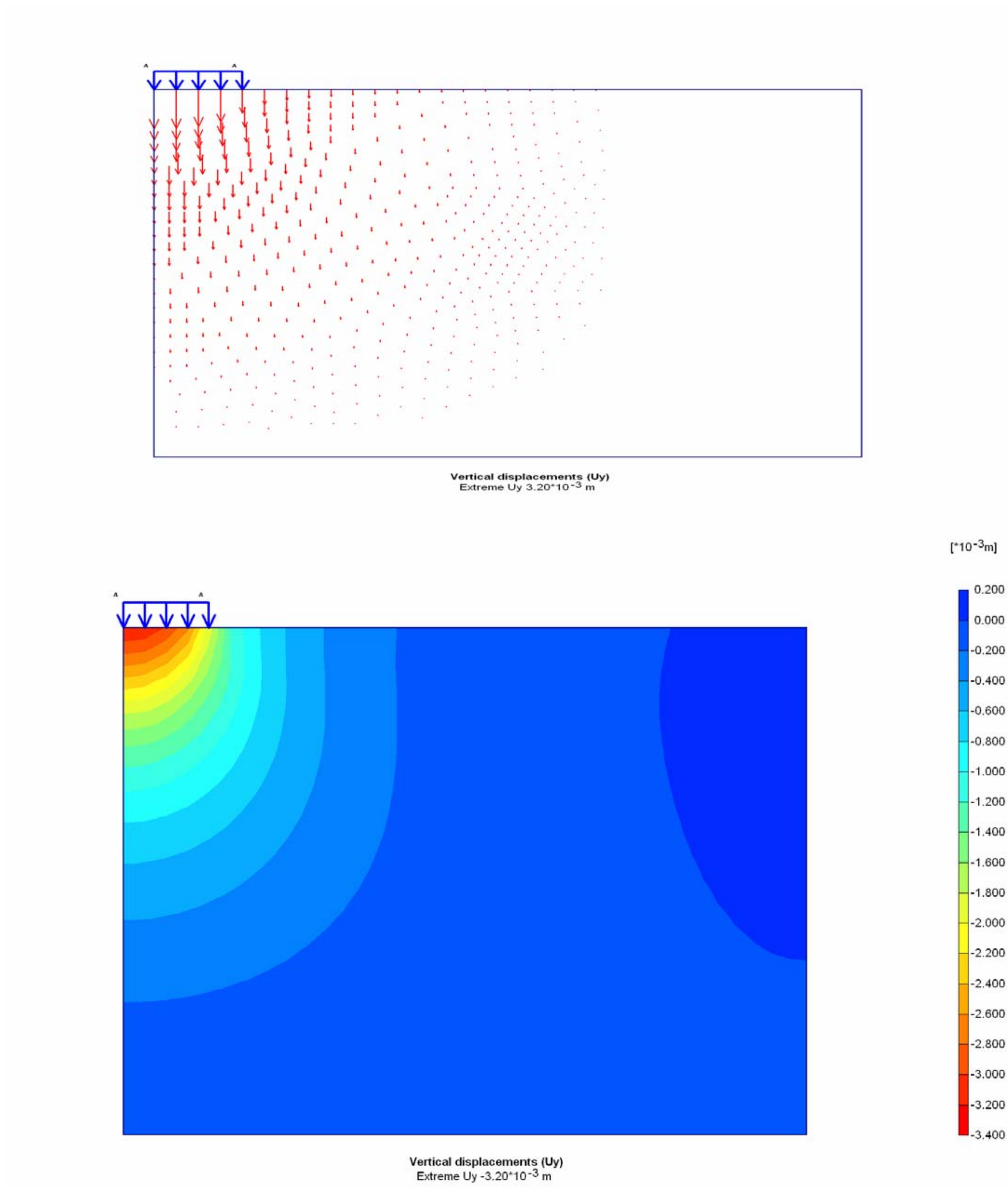


Fig.7.33 Vertical settlement in soil type A for circular footing (r=0.5m).

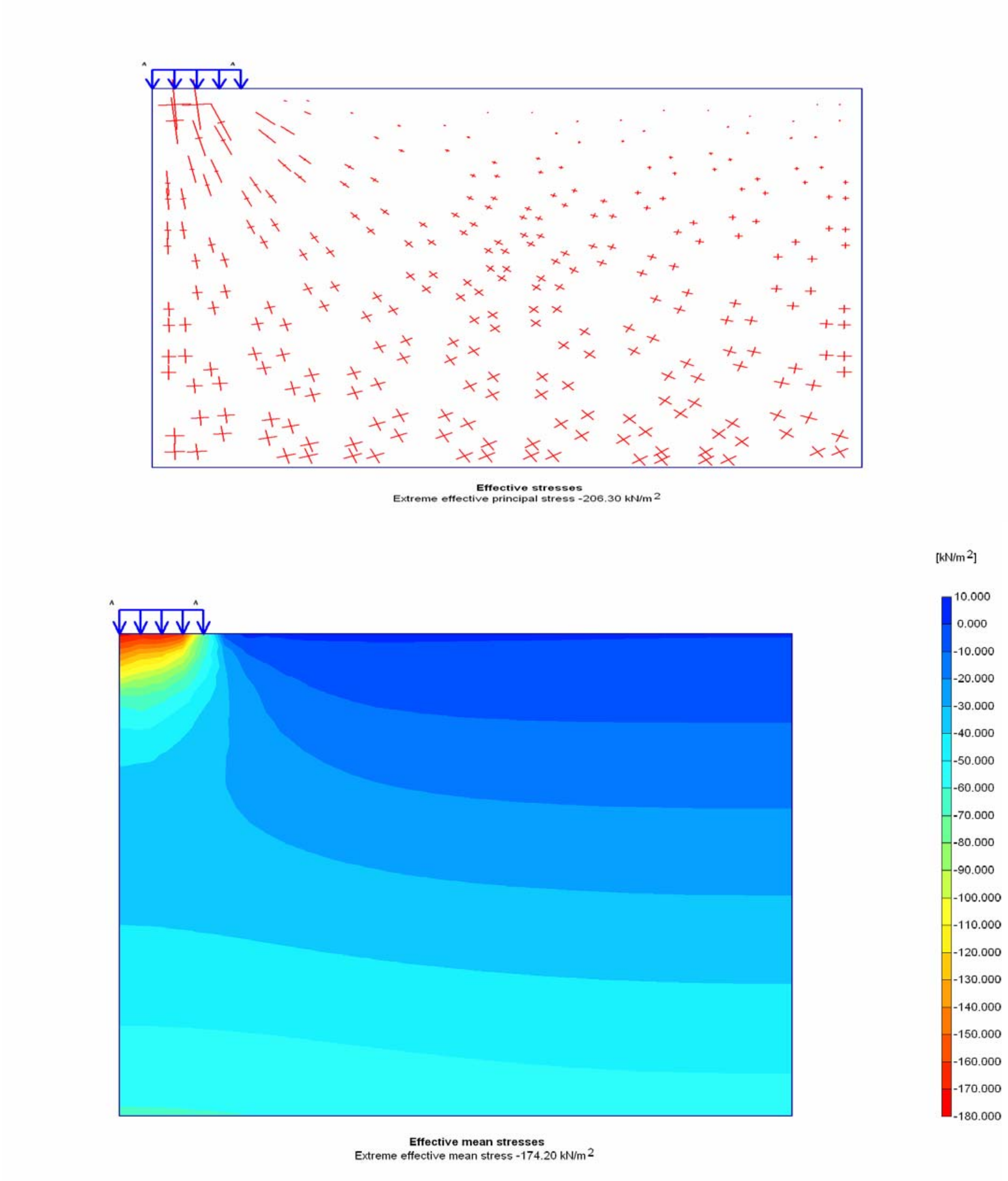


Fig. 4.34 Effective mean stress in soil type A for circular footing (r=0.5m).

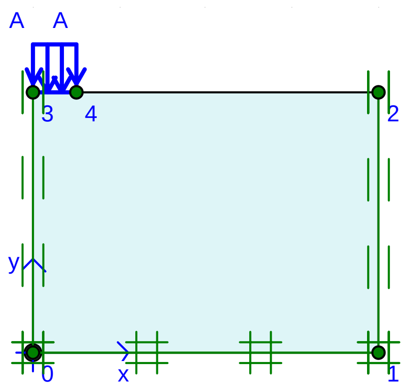


Fig. 7.35 Soil geometry with boundary

Table 7.36 Table of Settlement for soil type A

Node no.	x-coord.	y-coord.	uy [10^-3 m]
1	0	0	0
2	0.125	0	0
3	0.25	0	0
4	0.375	0	0
23	0.625	0	0
24	0.75	0	0
25	0.875	0	0
32	0.5	0	0
33	1	0	0
37	1.125	0	0
38	1.25	0	0
39	1.375	0	0
123	1.5	0	0
124	1.625	0	0
125	1.75	0	0
126	1.875	0	0
139	2	0	0
143	2.125	0	0
144	2.25	0	0
145	2.375	0	0
169	0	2.375	-1.547
170	0	2.25	-1.315
171	0	2.125	-1.122
187	0	2.5	-1.823
188	0.375	3	-2.638
189	0.25	3	-2.982
190	0.125	3	-3.161
191	0	3	-3.202
192	0	2.875	-2.886
193	0	2.75	-2.534
194	0	2.625	-2.174
303	2.625	0	0
304	2.75	0	0
305	2.875	0	0
309	2.5	0	0
313	3	0	0

317	3.125	0	0
318	3.25	0	0
319	3.375	0	0
355	0.5	3	-1.893
361	0.625	3	-1.201
371	3.5	0	0
372	3.625	0	0
373	3.75	0	0
374	3.875	0	0
375	4	0	0
376	4	0.125	-1.436
377	4	0.25	-2.374
378	4	0.375	-2.865
511	4	0.5	-2.959
515	4	0.625	-2.703
516	4	0.75	-2.144
517	4	0.875	-1.326
600	4	1.25	2.24
601	4	1.375	3.654
709	4	1.5	5.108
713	4	1.625	6.556
714	4	1.75	7.954
715	4	1.875	9.257
776	3	3	-7.438
781	4	2.5	13.007
795	3.5	3	7.219
807	3.125	3	-2.515
808	3.25	3	1.499
809	3.375	3	4.714

Table7.37 Soil data sets parameters for soil type A

LINEAR ELASTIC		SAND 01
Type		Drained
γ_{unsat}	[kN/m³]	18.00
γ_{sat}	[kN/m³]	20.00
k_x	[m/day]	0.010
k_y	[m/day]	0.010
e_{init}	[-]	0.500
c_k	[-]	1E15
E_{ref}	[kN/m²]	50000.00
ν	[-]	0.300
G_{ref}	[kN/m²]	19230.769
E_{oed}	[kN/m²]	67307.692
E_{incr}	[kN/m²/m]	0.00
y_{ref}	[m]	0.000
R_{inter}	[-]	1.000
Interface permeability		Neutral

Table 7.38 Table of deformations for soil type A

Node no.	x-coord.	y-coord.	uy [10^-3 m]
1	0	0	0
2	0.125	0	0
3	0.25	0	0
4	0.375	0	0
23	0.625	0	0
24	0.75	0	0
25	0.875	0	0
32	0.5	0	0
33	1	0	0
37	1.125	0	0
38	1.25	0	0
39	1.375	0	0
63	0	1.375	-1.497
64	0	1.25	-1.3
65	0	1.125	-1.119
123	1.5	0	0
124	1.625	0	0
125	1.75	0	0
126	1.875	0	0
139	2	0	0
143	2.125	0	0
144	2.25	0	0
145	2.375	0	0
155	0	1.5	-1.712
156	0	1.875	-2.481
157	0	1.75	-2.202
158	0	1.625	-1.946
165	0	2	-2.785
169	0	2.375	-3.841
170	0	2.25	-3.468
171	0	2.125	-3.115
187	0	2.5	-4.225
188	0.375	3	-5.366
189	0.25	3	-5.508
190	0.125	3	-5.591
191	0	3	-5.618
192	0	2.875	-5.32
193	0	2.75	-4.981
194	0	2.625	-4.611
303	2.625	0	0

304	2.75	0	0
305	2.875	0	0
309	2.5	0	0
313	3	0	0
317	3.125	0	0
318	3.25	0	0
319	3.375	0	0
355	0.5	3	-5.155
359	0.875	3	-3.958
360	0.75	3	-4.495
361	0.625	3	-4.874
371	3.5	0	0
372	3.625	0	0
373	3.75	0	0
374	3.875	0	0
375	4	0	0
376	4	0.125	-6.63
377	4	0.25	-11.289
378	4	0.375	-14.179
443	1	3	-3.041
447	1.375	3	-1.341
448	1.25	3	-1.676
449	1.125	3	-2.165
511	4	0.5	-15.493
515	4	0.625	-15.409
516	4	0.75	-14.1
517	4	0.875	-11.728
585	1.5	3	-1.085
595	4	1	-8.453
599	4	1.125	-4.436

Table 7.39 Table of total stresses for soil type A

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
1	0.032	0.437	-54.662	-96.796	0.320	-54.663
	0.063	0.468	-125.210	-192.959	0.642	-125.210
	0.500	0.500	-195.054	-287.176	4.898	-195.000
2	0.984	0.322	-58.723	-85.476	7.494	-56.780
	1.548	0.345	-128.454	-178.445	12.863	-126.446
	2.517	0.368	-196.100	-265.956	20.854	-193.927
3	1.047	0.322	-58.695	-84.238	7.727	-56.565
	2.079	0.345	-125.957	-170.751	15.995	-123.645
	3.517	0.368	-190.575	-250.453	25.175	-187.987
4	2.002	0.323	-56.990	-65.590	7.421	-53.077
	3.567	0.347	-120.759	-143.136	16.598	-116.547
	5.538	0.370	-181.980	-214.290	25.039	-177.430
5	2.065	0.323	-56.808	-64.573	7.221	-52.875
	4.099	0.347	-117.665	-134.824	15.468	-113.393
	6.538	0.370	-176.467	-200.046	22.162	-171.865
6	2.957	0.214	-55.261	-56.805	3.928	-52.936
	5.519	0.229	-113.608	-120.933	10.115	-111.070
	8.487	0.245	-170.585	-181.822	14.450	-167.860
7	3.020	0.214	-55.095	-56.392	3.669	-52.826
	6.051	0.229	-111.906	-116.893	7.716	-109.454
	9.487	0.245	-167.928	-175.563	9.973	-165.324
8	3.968	0.437	-51.060	-48.817	0.094	-48.390
	7.532	0.468	-106.795	-106.785	1.800	-103.834
	11.500	0.500	-162.288	-164.241	1.917	-159.090
9	3.650	0.995	-43.190	-37.622	0.839	-38.124
	7.625	1.560	-92.548	-83.763	0.910	-84.279
	11.599	2.531	-135.683	-121.580	0.968	-123.143
10	3.650	1.058	-42.106	-36.340	0.811	-36.950
	7.625	2.092	-84.160	-72.880	0.866	-74.651
	11.599	3.531	-118.561	-101.351	0.906	-104.718
11	3.679	1.997	-22.276	-17.941	0.161	-18.956
	7.656	3.562	-54.045	-43.953	0.192	-46.590
	11.633	5.533	-76.613	-62.296	0.204	-66.254
12	3.679	2.060	-20.711	-16.772	0.115	-17.714
	7.656	4.094	-41.750	-33.953	0.123	-36.119
	11.633	6.533	-52.482	-43.854	0.115	-46.344
13	3.563	2.968	3.774	-0.566	-0.037	0.281
	7.532	5.532	-3.687	-8.284	-0.051	-7.420
	11.500	8.500	-0.478	-8.850	-0.053	-6.882
14	3.047	2.704	-3.264	-5.303	-0.343	-5.229
	6.485	5.683	1.085	-5.687	-0.375	-4.831
	9.517	8.661	6.832	-6.071	-0.430	-4.792
15	2.984	2.704	-3.240	-5.314	-0.346	-5.279
	5.953	5.683	2.791	-5.698	-0.404	-5.316
	8.517	8.661	11.135	-6.081	-0.482	-6.093
16	2.059	2.672	-7.462	-6.526	1.405	-7.707

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	4.498	5.649	1.685	-6.946	1.321	-8.883
	6.532	8.625	14.499	-7.371	1.245	-12.035
17	1.996	2.672	-8.247	-6.776	1.782	-8.015
	3.967	5.649	5.226	-7.202	1.714	-11.647
	5.532	8.625	24.579	-7.689	1.661	-20.155
18	1.030	2.668	-47.296	-80.384	55.007	-30.107
	2.467	5.644	-27.385	-80.231	55.998	-41.600
	3.498	8.620	-40.872	-132.239	85.213	-95.014
19	0.967	2.668	-49.493	-111.497	56.217	-38.944
	1.935	5.644	-140.672	-258.654	88.020	-146.540
	2.498	8.620	-281.221	-460.215	90.146	-289.728
20	0.032	2.563	-63.458	-195.405	0.983	-63.306
	0.468	5.532	-203.856	-395.662	1.587	-204.699
	0.500	8.500	-346.480	-596.338	1.502	-347.370
21	0.302	2.038	-28.125	-147.087	15.358	-26.755
	0.324	4.475	-77.374	-334.581	16.318	-76.027
	0.345	6.507	-104.875	-487.402	17.430	-103.521
22	0.302	1.975	-27.345	-142.040	14.758	-25.916
	0.324	3.944	-53.877	-289.627	15.835	-52.441
	0.345	5.507	-81.498	-410.230	16.602	-80.064
23	0.322	1.046	-38.256	-100.197	5.929	-37.475
	0.345	2.484	-67.747	-214.910	6.622	-66.960
	0.368	3.517	-105.580	-317.401	7.058	-104.789
24	0.322	0.983	-39.711	-99.174	5.510	-38.987
	0.345	1.953	-79.075	-200.512	5.914	-78.347
	0.368	2.517	-129.625	-297.753	6.178	-128.894
25	0.600	0.526	-52.561	-91.831	5.943	-51.285
	1.137	0.586	-120.907	-185.075	10.968	-119.482
	2.094	0.944	-178.653	-270.848	18.424	-175.191
26	3.427	2.516	-8.871	-8.614	-0.204	-8.716
	6.893	5.461	-5.530	-9.597	-0.278	-8.958
	9.968	8.132	-9.877	-15.499	-0.609	-14.808
27	1.564	0.585	-53.349	-70.016	8.970	-47.772
	3.100	0.649	-116.502	-147.549	17.921	-110.146
	5.072	1.013	-172.989	-212.932	25.407	-162.334
28	3.361	0.600	-49.781	-46.584	1.939	-45.308
	7.291	1.139	-99.543	-93.293	2.138	-91.903
	10.896	2.107	-143.264	-131.547	3.107	-130.492
29	0.499	2.416	-41.476	-169.514	26.706	-40.554
	0.555	4.881	-93.205	-358.742	29.152	-92.348
	0.892	6.947	-121.918	-506.531	46.469	-119.374
30	0.970	0.926	-44.940	-82.906	11.602	-40.082
	1.601	1.515	-95.851	-174.198	18.102	-89.452
	2.589	1.936	-151.982	-258.954	25.869	-143.134
31	3.122	2.208	-18.159	-14.493	0.383	-14.700
	6.524	4.671	-28.556	-24.067	0.211	-24.463
	9.574	7.288	-34.609	-30.940	-0.084	-31.348
32	2.651	0.580	-51.843	-51.851	4.710	-46.097
	5.190	0.637	-109.931	-115.386	10.842	-103.424

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	8.125	0.891	-164.842	-171.502	14.789	-155.605
33	2.022	0.877	-49.045	-54.970	8.264	-40.496
	3.621	1.521	-101.441	-123.320	17.426	-86.760
	5.628	1.943	-156.997	-187.077	24.798	-137.447
34	3.108	1.008	-44.276	-39.270	2.525	-37.623
	6.410	1.643	-93.721	-85.391	4.641	-82.307
	9.956	2.645	-136.990	-123.086	5.753	-120.227
35	3.290	0.519	-50.997	-48.565	2.249	-46.822
	6.329	0.770	-105.679	-104.081	5.778	-98.919
	9.784	0.821	-161.376	-161.909	7.909	-154.120
36	0.831	2.090	-34.587	-107.603	36.373	-22.647
	1.386	4.479	-73.627	-269.413	66.942	-59.620
	1.779	6.517	-102.224	-412.197	86.662	-85.840
37	1.499	0.585	-53.285	-71.508	9.081	-47.903
	2.580	0.948	-110.937	-154.760	16.993	-103.090
	4.052	1.012	-174.474	-233.420	25.946	-165.877
38	0.421	0.987	-40.079	-97.575	7.054	-38.858
	1.011	1.617	-89.761	-189.631	13.449	-87.089
	1.940	2.584	-133.661	-273.749	25.360	-126.213
39	3.588	2.068	-20.663	-16.678	0.159	-17.519
	7.029	4.491	-32.133	-26.953	0.022	-28.055
	10.190	6.660	-51.252	-42.161	0.437	-43.518
40	1.467	0.648	-52.143	-71.440	9.456	-46.395
	2.498	1.578	-97.408	-152.481	21.320	-86.339
	3.547	2.004	-153.490	-235.937	29.315	-139.726
41	2.611	2.350	-16.365	-12.524	1.155	-12.170
	5.676	4.550	-34.956	-27.255	1.648	-27.025
	8.669	7.160	-41.312	-34.294	1.371	-34.102
42	2.591	0.588	-51.899	-52.320	4.969	-46.001
	4.696	0.952	-108.036	-115.527	12.019	-97.794
	7.176	1.016	-166.297	-179.521	18.377	-155.179
43	3.236	0.560	-50.589	-47.965	2.406	-46.080
	5.937	1.177	-101.814	-98.563	6.879	-91.397
	8.922	1.468	-156.209	-153.579	10.561	-142.790
44	1.622	1.146	-45.207	-59.636	12.658	-34.903
	3.192	1.842	-96.708	-127.876	22.170	-79.949
	5.187	2.771	-145.049	-182.358	30.799	-119.350
45	2.563	0.647	-51.192	-51.428	5.077	-44.838
	4.655	1.524	-100.164	-105.063	12.916	-85.285
	6.732	1.947	-155.560	-167.651	20.046	-135.786
46	2.706	0.667	-50.537	-49.516	4.428	-44.316
	5.946	1.277	-100.449	-96.417	6.776	-89.460
	8.994	2.261	-145.299	-136.549	9.556	-127.532
47	1.620	2.237	-35.456	-29.315	18.141	-15.654
	3.670	4.462	-62.366	-48.473	25.391	-30.521
	5.675	7.034	-76.231	-57.691	28.537	-39.987
48	3.250	1.502	-34.690	-28.349	1.495	-28.287
	6.355	2.573	-77.933	-66.323	3.982	-64.717
	9.898	3.639	-120.136	-102.747	5.069	-101.466

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
49	0.963	2.574	-46.388	-104.722	53.465	-31.675
	1.559	5.002	-86.947	-266.366	86.283	-70.258
	2.432	7.130	-122.543	-370.421	124.505	-92.557
50	0.636	1.597	-31.732	-104.390	19.443	-26.655
	1.473	3.629	-65.929	-209.074	54.062	-49.093
	1.871	5.610	-93.952	-347.089	73.304	-74.590
51	0.060	0.537	-51.411	-97.085	0.661	-51.392
	0.587	1.136	-101.743	-190.147	6.337	-100.598
	0.945	2.092	-142.241	-288.383	12.230	-140.232
52	3.942	2.469	-9.957	-9.378	-0.022	-9.620
	7.439	4.913	-20.771	-19.247	-0.171	-19.726
	11.084	7.003	-40.790	-35.490	-0.061	-36.842
53	2.759	1.913	-27.793	-21.620	2.450	-20.266
	5.807	4.063	-47.843	-37.369	3.107	-36.080
	8.401	6.363	-65.971	-51.030	4.599	-49.180
54	2.536	2.937	6.702	-1.130	-0.195	-1.616
	5.108	5.340	-8.117	-12.604	0.826	-12.848
	8.063	8.003	-12.741	-18.677	0.507	-18.940
55	1.067	0.983	-44.607	-79.686	12.590	-38.689
	2.571	1.683	-95.924	-149.521	22.291	-83.740
	4.126	2.833	-140.889	-211.171	35.484	-118.565
56	1.985	1.320	-43.132	-46.167	10.271	-31.362
	3.636	2.511	-87.870	-104.065	23.078	-65.318
	5.661	3.485	-135.563	-157.017	31.675	-103.754
57	2.566	1.300	-41.320	-37.230	5.072	-31.883
	5.223	2.017	-91.303	-86.195	9.704	-75.240
	8.222	3.051	-135.490	-125.580	12.645	-112.352
58	1.949	1.954	-34.598	-29.571	11.255	-19.527
	3.995	4.134	-62.841	-50.168	19.033	-35.141
	5.611	6.326	-99.019	-81.524	37.781	-51.391
59	2.603	1.358	-40.154	-35.496	4.768	-30.766
	5.640	2.451	-83.250	-73.411	7.506	-66.773
	8.823	3.974	-117.739	-101.561	9.180	-94.619
60	1.537	2.927	10.525	-2.464	0.607	-10.029
	3.121	5.214	-24.468	-30.862	19.687	-24.970
	5.089	7.837	-36.106	-38.848	22.397	-33.802
61	3.927	1.475	-33.722	-27.777	0.112	-29.327
	7.236	3.006	-67.609	-55.349	1.399	-57.091
	10.838	4.100	-109.178	-91.050	2.313	-93.335
62	0.534	2.941	-133.939	-202.152	0.078	-134.890
	1.106	5.427	-178.572	-372.324	30.142	-179.381
	2.044	8.060	-224.034	-485.733	85.796	-215.471
63	1.177	1.634	-38.545	-72.106	23.481	-25.406
	2.070	3.641	-73.559	-169.978	57.838	-47.415
	2.763	5.213	-106.278	-270.639	77.780	-74.370
64	0.062	1.534	-28.082	-118.975	2.082	-28.024
	0.642	3.095	-59.507	-224.957	19.633	-55.289
	0.984	5.040	-86.831	-363.009	35.910	-80.761
65	2.330	1.935	-30.708	-24.164	5.663	-19.801

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	4.833	4.239	-49.427	-38.023	7.583	-32.885
	6.965	6.425	-76.177	-57.352	13.969	-48.311
66	3.200	1.625	-32.258	-25.988	1.455	-25.904
	6.291	3.727	-53.452	-42.624	2.151	-42.652
	9.094	5.591	-82.154	-65.116	4.548	-63.848
67	2.372	1.914	-30.789	-24.290	5.328	-20.188
	5.082	3.810	-59.315	-46.541	8.106	-40.764
	7.627	6.093	-78.363	-60.731	9.929	-54.200
68	1.754	1.608	-40.415	-45.346	14.678	-25.751
	3.386	2.844	-84.583	-102.985	28.061	-58.757
	5.351	4.210	-127.174	-148.517	38.719	-89.210
69	2.293	1.909	-31.667	-25.277	6.207	-20.289
	4.388	4.068	-59.704	-45.843	13.388	-36.202
	6.386	6.001	-94.076	-75.005	23.810	-56.071
70	2.123	0.940	-48.053	-51.771	7.822	-39.110
	4.716	1.650	-98.301	-101.554	12.749	-82.650
	7.219	2.943	-140.003	-139.658	18.272	-114.650
71	1.749	1.707	-39.479	-42.817	15.246	-23.922
	3.677	3.626	-74.891	-73.973	27.057	-44.050
	5.272	5.784	-111.816	-107.759	46.799	-60.825
72	2.766	1.814	-30.098	-23.789	2.751	-22.128
	5.350	3.225	-69.429	-58.284	7.601	-51.886
	8.513	4.800	-102.894	-85.423	9.254	-78.724
73	1.495	2.165	-38.309	-38.762	23.530	-16.938
	2.446	4.215	-74.663	-131.477	59.451	-38.429
	3.681	5.892	-113.617	-199.670	83.223	-63.072
74	0.988	1.065	-42.651	-82.139	13.446	-36.822
	2.172	2.631	-81.574	-154.253	35.484	-63.403
	2.871	4.135	-115.093	-252.825	54.112	-91.454
75	0.471	2.933	-130.512	-198.893	0.966	-131.638
	0.537	5.461	-189.847	-392.719	3.625	-190.866
	1.046	7.940	-235.543	-567.601	29.632	-236.755
76	2.086	1.620	-38.627	-36.403	9.641	-25.588
	3.872	3.255	-78.653	-79.959	23.905	-50.828
	5.871	4.649	-120.801	-124.062	34.280	-80.745
77	2.527	1.431	-39.257	-34.627	5.246	-29.352
	5.236	3.265	-69.252	-58.285	8.254	-51.087
	7.606	5.118	-101.416	-84.184	13.874	-72.407
78	2.120	1.670	-37.630	-34.390	9.152	-24.659
	4.415	3.544	-70.042	-60.594	15.502	-45.595
	6.415	5.441	-105.011	-90.804	26.033	-66.087
79	2.083	1.656	-38.112	-35.496	9.669	-24.923
	4.046	3.539	-73.717	-66.959	20.918	-45.679
	5.830	5.209	-113.394	-109.630	35.374	-70.266
80	3.934	2.530	-8.346	-8.304	-0.029	-8.382
	7.462	5.465	-5.419	-9.469	-0.105	-8.785
	10.951	7.970	-14.540	-18.250	-0.291	-17.691
81	0.068	0.473	-53.460	-96.832	0.715	-53.445
	0.541	0.541	-121.666	-190.727	5.185	-121.517

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	1.077	1.075	-173.838	-283.581	10.636	-172.638
82	3.519	0.068	-55.476	-57.297	1.849	-54.849
	7.443	0.541	-106.104	-105.364	2.073	-102.580
	10.797	1.077	-156.759	-153.328	4.084	-149.088
83	2.487	1.416	-39.704	-35.415	5.569	-29.616
	4.817	3.255	-72.528	-62.131	11.671	-51.196
	6.972	4.890	-110.470	-96.820	20.394	-76.518
84	1.289	1.651	-39.403	-65.260	22.617	-25.025
	2.992	3.338	-79.251	-110.220	38.637	-49.298
	4.541	5.477	-117.091	-147.114	60.118	-66.476
85	2.511	2.401	-15.344	-11.665	1.288	-11.328
	4.984	5.336	-8.398	-12.846	1.088	-13.160
	7.078	7.967	-17.790	-20.287	2.746	-21.388
86	2.485	2.350	-17.312	-12.857	1.726	-12.256
	4.552	4.930	-29.749	-21.637	4.182	-21.382
	6.664	7.163	-55.414	-39.807	10.383	-36.039
87	3.312	1.595	-32.536	-26.285	1.193	-26.554
	7.241	3.133	-64.935	-52.831	1.293	-54.666
	10.871	5.103	-87.960	-71.310	1.511	-74.131
88	3.255	1.636	-31.825	-25.602	1.279	-25.729
	6.830	3.648	-53.936	-43.348	1.502	-44.355
	9.977	5.759	-74.635	-59.680	2.067	-60.908
89	0.582	1.498	-32.162	-103.893	16.416	-28.239
	0.645	2.970	-61.170	-219.934	18.404	-57.195
	1.007	4.049	-98.866	-320.043	25.274	-93.874
90	0.644	1.466	-33.286	-100.210	16.998	-28.727
	1.069	2.514	-72.033	-198.574	24.623	-66.138
	2.003	3.541	-114.888	-282.578	37.330	-103.843
91	1.518	2.287	-37.400	-32.247	21.702	-15.675
	2.990	5.214	-25.446	-32.813	21.733	-25.899
	4.055	7.832	-73.378	-108.414	73.423	-53.268
92	1.480	2.229	-38.349	-37.115	24.001	-16.337
	2.506	4.788	-84.911	-124.282	76.028	-43.721
	3.442	6.902	-121.565	-219.944	114.029	-65.245
93	2.044	1.331	-42.845	-44.624	9.681	-31.164
	4.126	2.315	-90.326	-96.198	17.905	-69.354
	6.589	3.654	-131.461	-133.741	23.711	-100.474
94	2.047	1.375	-42.236	-43.470	9.765	-30.286
	4.514	2.758	-82.607	-79.918	15.520	-60.541
	6.648	4.360	-121.205	-115.865	24.521	-86.488
95	1.583	1.253	-43.872	-58.925	13.975	-32.677
	3.289	2.876	-84.271	-105.508	29.569	-58.125
	4.580	4.463	-124.007	-171.462	51.022	-84.291
96	1.536	1.211	-44.187	-61.246	13.954	-33.544
	2.780	2.756	-83.773	-130.212	35.115	-60.484
	3.829	3.800	-127.238	-210.302	48.455	-97.766

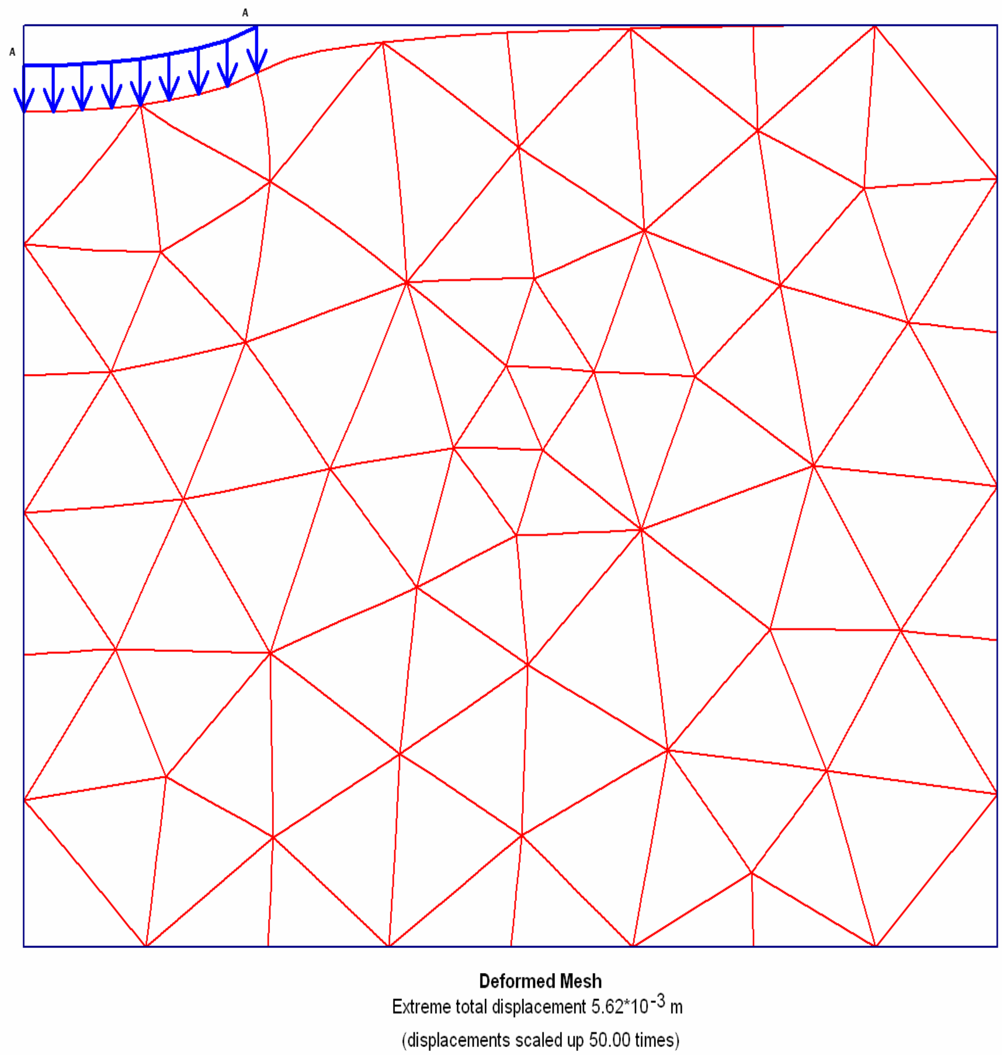


Fig.7.40 Problem geometry with mesh generation

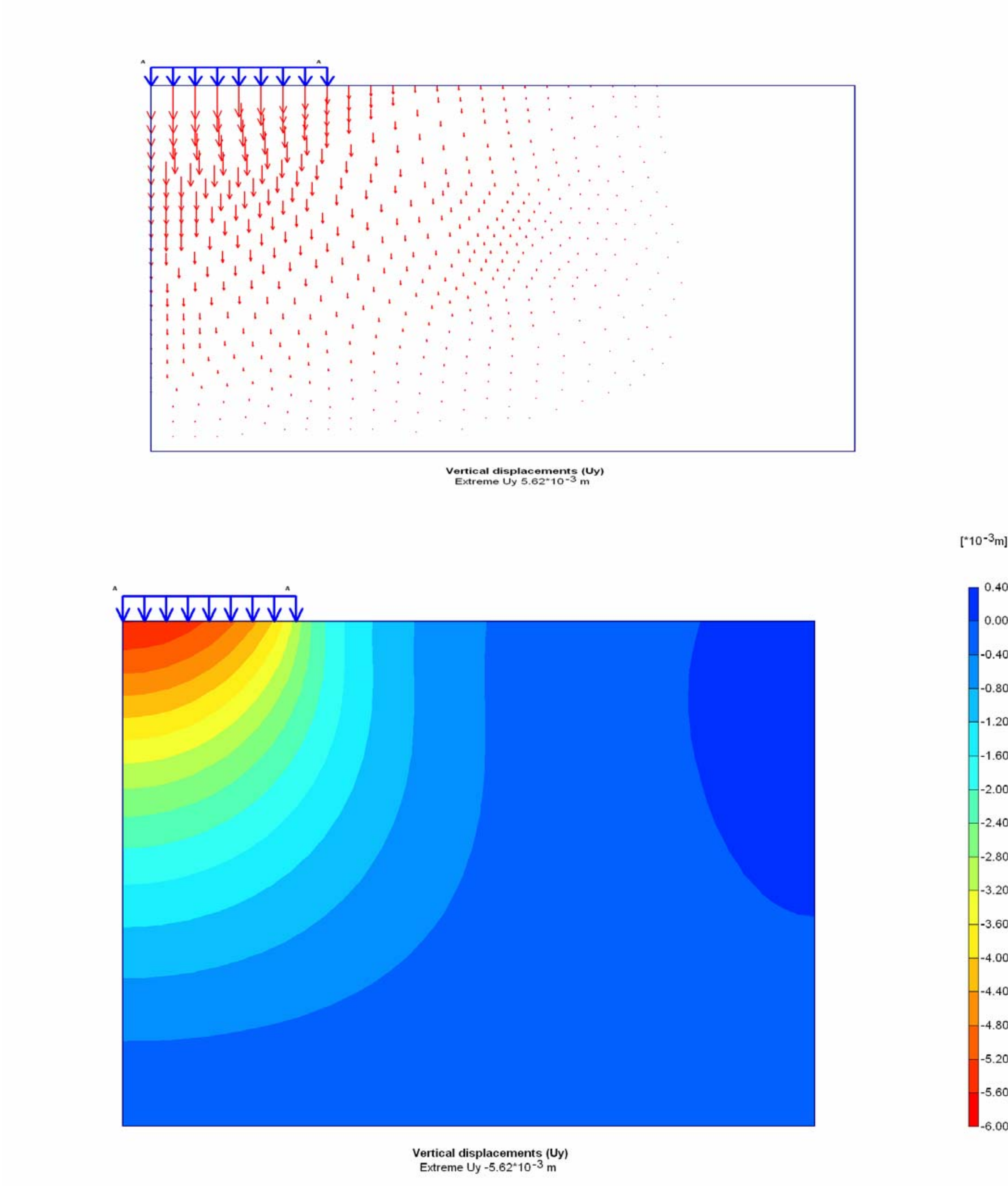
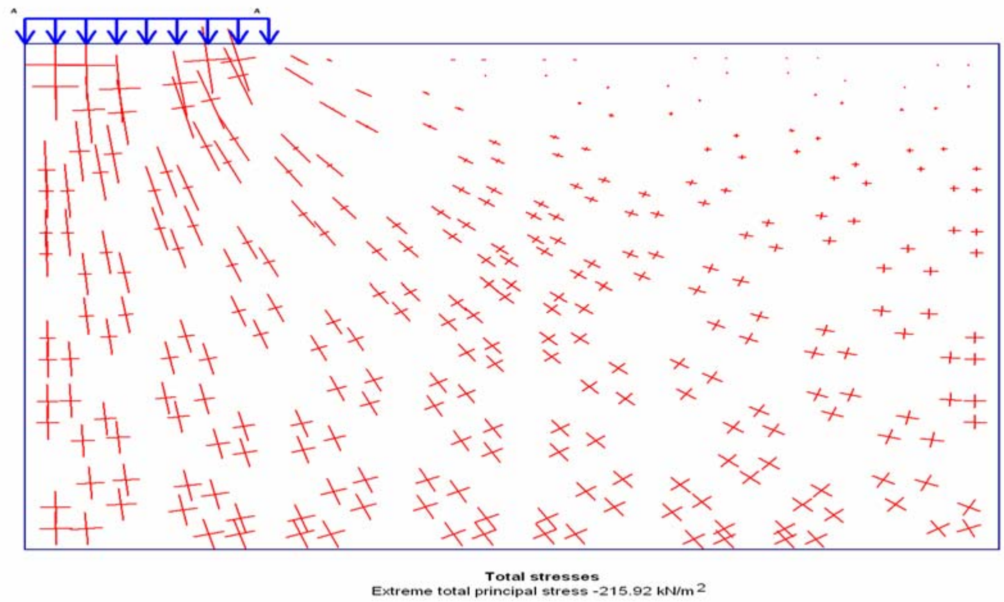


Fig.4.41 Vertical settlement in soil type A for circular footing (r=1.0m).



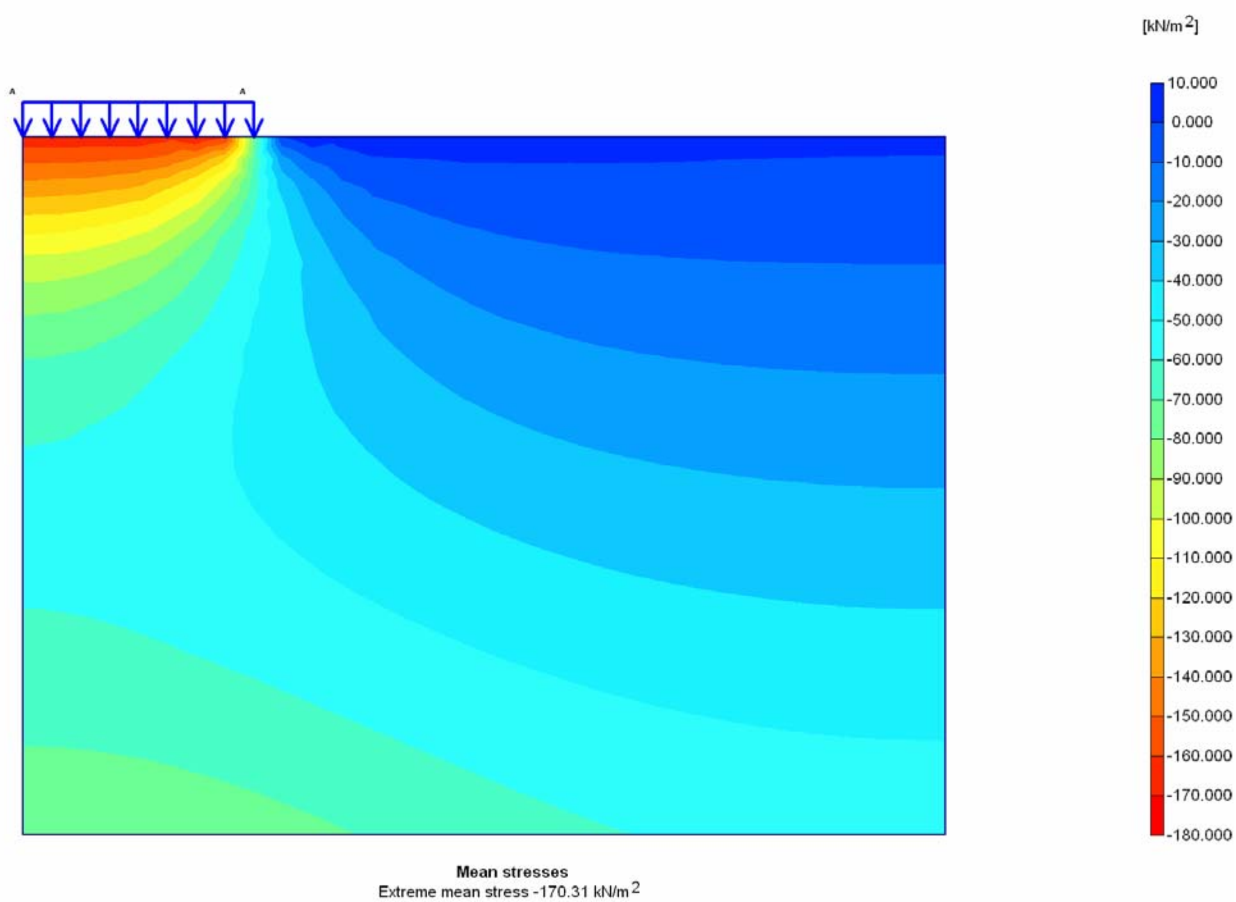


Fig.4.42 Mean stresses in soil type A for circular footing (r=1.0m).

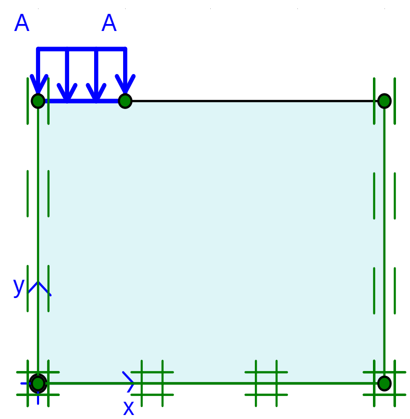


Fig.7.43 Soil geometry with boundary

Table 7.44 Soil data sets parameters for soil type A

LINEAR ELASTIC		1 SAND 01
Type		Drained
γ_{unsat}	[kN/m³]	18.00
γ_{sat}	[kN/m³]	20.00
k_x	[m/day]	0.010
k_y	[m/day]	0.010
e_{init}	[-]	0.500
c_k	[-]	1E15
E_{ref}	[kN/m²]	50000.00
ν	[-]	0.300
G_{ref}	[kN/m²]	19230.769
E_{oed}	[kN/m²]	67307.692
E_{incr}	[kN/m²/m]	0.00
y_{ref}	[m]	0.000
R_{inter}	[-]	1.000
Interface permeability		Neutral

Table 7.45 Table of deformations for soil type A

Node no.	x-coord.	y-coord.	uy [10^-3 m]
1	0	0	0
2	0.125	0	0
3	0.25	0	0
4	0.375	0	0
23	0.625	0	0
24	0.75	0	0
25	0.875	0	0
32	0.5	0	0
33	1	0	0
37	1.125	0	0
38	1.25	0	0
39	1.375	0	0
49	0	0.875	-1.434
50	0	0.75	-1.186
59	0	1	-1.696
63	0	1.375	-2.572
64	0	1.25	-2.265
65	0	1.125	-1.973
123	1.5	0	0
124	1.625	0	0
125	1.75	0	0
126	1.875	0	0
139	2	0	0
143	2.125	0	0
144	2.25	0	0
145	2.375	0	0
155	0	1.5	-2.896
156	0	1.875	-3.959
157	0	1.75	-3.59
158	0	1.625	-3.235
165	0	2	-4.34
169	0	2.375	-5.523
170	0	2.25	-5.127
171	0	2.125	-4.731
187	0	2.5	-5.913
188	0.375	3	-7.135
189	0.25	3	-7.22
190	0.125	3	-7.27
191	0	3	-7.287

192	0	2.875	-6.981
193	0	2.75	-6.648
194	0	2.625	-6.29
303	2.625	0	0
304	2.75	0	0
305	2.875	0	0
309	2.5	0	0
313	3	0	0
317	3.125	0	0
318	3.25	0	0
319	3.375	0	0
355	0.5	3	-7.014
359	0.875	3	-6.399
360	0.75	3	-6.651
361	0.625	3	-6.853
371	3.5	0	0
372	3.625	0	0
373	3.75	0	0
374	3.875	0	0
375	4	0	0
443	1	3	-6.09
447	1.375	3	-4.627
448	1.25	3	-5.246
449	1.125	3	-5.712
585	1.5	3	-3.625
589	1.875	3	-1.68
590	1.75	3	-2.088
591	1.625	3	-2.659
679	2.125	3	-1.098
709	4	1.5	-17.679
713	4	1.625	-7.645
714	4	1.75	2.567

Table 7.46 Table of total stresses for soil type A

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
1	0.032	0.437	-65.296	-140.039	0.477	-65.298
	0.063	0.468	-151.757	-275.003	0.991	-151.757
	0.500	0.500	-237.141	-407.099	7.878	-237.078
2	0.984	0.322	-69.597	-122.375	12.423	-67.081
	1.548	0.345	-154.752	-252.503	21.175	-152.159
	2.517	0.368	-236.608	-374.164	34.688	-233.802
3	1.047	0.322	-69.476	-120.441	12.920	-66.679
	2.079	0.345	-150.706	-240.501	26.999	-147.673
	3.517	0.368	-227.470	-349.164	43.420	-224.042
4	2.002	0.323	-66.321	-87.888	14.319	-59.829
	3.567	0.347	-141.587	-192.758	31.007	-134.653
	5.538	0.370	-212.156	-285.798	47.165	-204.669
5	2.065	0.323	-66.028	-85.890	14.053	-59.406
	4.099	0.347	-135.900	-177.187	29.963	-128.713
	6.538	0.370	-201.689	-258.368	43.468	-193.899
6	2.957	0.214	-61.865	-67.186	8.305	-57.213
	5.519	0.229	-126.588	-145.907	20.935	-121.537
	8.487	0.245	-188.380	-217.623	30.077	-182.954
7	3.020	0.214	-61.548	-66.254	7.788	-56.965
	6.051	0.229	-122.976	-137.107	16.358	-118.024
	9.487	0.245	-182.648	-203.826	21.231	-177.374
8	3.968	0.437	-57.725	-54.060	0.210	-51.931
	7.532	0.468	-117.093	-119.552	3.909	-110.676
	11.500	0.500	-175.919	-183.857	4.165	-168.983
9	3.650	0.995	-52.398	-41.370	2.123	-41.349
	7.625	1.560	-109.126	-92.288	2.284	-91.118
	11.599	2.531	-160.944	-133.456	2.430	-133.550
10	3.650	1.058	-51.401	-39.862	2.090	-40.119
	7.625	2.092	-102.220	-79.534	2.233	-81.347
	11.599	3.531	-145.116	-109.785	2.353	-114.451
11	3.679	1.997	-28.395	-18.473	0.784	-20.498
	7.656	3.562	-68.272	-45.879	0.883	-50.943
	11.633	5.533	-96.660	-64.629	0.937	-72.445
12	3.679	2.060	-26.285	-17.179	0.664	-19.094
	7.656	4.094	-52.640	-34.657	0.710	-39.155
	11.633	6.533	-64.556	-44.482	0.707	-49.706
13	3.563	2.968	10.048	-0.565	-0.072	1.499
	7.532	5.532	2.917	-8.195	-0.088	-6.079
	11.500	8.500	11.698	-8.760	-0.093	-3.909
14	3.047	2.704	-1.381	-5.359	-0.324	-5.318
	6.485	5.683	9.744	-5.742	-0.388	-3.819
	9.517	8.661	23.848	-6.126	-0.494	-3.301
15	2.984	2.704	-1.484	-5.397	-0.274	-5.468

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m^2]	σ_{yy} [kN/m^2]	σ_{xy} [kN/m^2]	σ_{zz} [kN/m^2]
	5.953	5.683	13.215	-5.781	-0.386	-5.153
	8.517	8.661	32.737	-6.157	-0.544	-6.742
16	2.059	2.672	-20.572	-9.974	9.992	-12.400
	4.498	5.649	0.578	-10.339	9.855	-14.922
	6.532	8.625	28.240	-10.514	9.939	-22.356
17	1.996	2.672	-25.540	-12.971	12.357	-14.389
	3.967	5.649	2.629	-13.447	12.356	-23.265
	5.532	8.625	-2.135	-24.440	32.908	-58.264
18	1.030	2.668	-67.307	-193.421	19.369	-78.462
	2.467	5.644	-154.454	-381.975	41.226	-194.593
	3.498	8.620	-289.436	-582.784	40.990	-330.569
19	0.967	2.668	-73.303	-197.617	16.168	-82.381
	1.935	5.644	-208.919	-398.265	16.474	-218.500
	2.498	8.620	-345.333	-598.795	16.436	-354.759
20	0.032	2.563	-84.454	-204.659	0.329	-84.429
	0.468	5.532	-219.563	-405.247	0.253	-219.491
	0.500	8.500	-354.559	-605.817	0.255	-354.490
21	0.302	2.038	-46.169	-188.276	9.326	-46.024
	0.324	4.475	-118.379	-391.677	9.723	-118.236
	0.345	6.507	-165.121	-582.289	10.384	-164.977
22	0.302	1.975	-44.156	-185.401	9.557	-43.905
	0.324	3.944	-88.722	-373.256	10.239	-88.470
	0.345	5.507	-127.753	-542.796	10.911	-127.502
23	0.322	1.046	-46.087	-148.560	7.093	-45.379
	0.345	2.484	-85.632	-312.906	7.758	-84.922
	0.368	3.517	-131.545	-463.857	8.272	-130.831
24	0.322	0.983	-47.557	-147.023	6.769	-46.872
	0.345	1.953	-94.985	-296.378	7.259	-94.297
	0.368	2.517	-155.129	-438.114	7.636	-154.440
25	0.600	0.526	-62.132	-134.078	8.921	-60.632
	1.137	0.586	-145.382	-265.049	17.103	-143.705
	2.094	0.944	-213.724	-388.343	29.343	-209.428
26	3.427	2.516	-9.637	-8.644	-0.058	-8.886
	6.893	5.461	-0.149	-9.627	-0.202	-8.089
	9.968	8.132	-3.276	-15.601	-0.452	-14.057
27	1.564	0.585	-62.539	-100.811	15.997	-54.343
	3.100	0.649	-137.081	-206.323	32.216	-127.738
	5.072	1.013	-202.960	-294.578	46.617	-186.536
28	3.361	0.600	-58.135	-52.929	4.402	-48.709
	7.291	1.139	-115.173	-104.528	4.853	-98.870
	10.896	2.107	-168.132	-146.759	7.282	-140.675
29	0.499	2.416	-65.365	-199.387	10.498	-66.778
	0.555	4.881	-139.999	-403.193	11.390	-141.460
	0.892	6.947	-187.003	-592.089	21.683	-188.330
30	0.970	0.926	-52.337	-126.173	16.795	-46.800
	1.601	1.515	-112.352	-260.134	26.411	-105.019
	2.589	1.936	-178.542	-382.662	39.073	-168.056
31	3.122	2.208	-24.379	-15.573	2.157	-15.715

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	6.524	4.671	-36.384	-25.233	2.255	-25.798
	9.574	7.288	-42.399	-32.252	2.229	-32.946
32	2.651	0.580	-61.144	-64.678	10.109	-50.161
	5.190	0.637	-125.990	-143.018	22.596	-113.590
	8.125	0.891	-187.867	-209.612	30.948	-170.023
33	2.022	0.877	-58.963	-78.205	16.537	-44.809
	3.621	1.521	-120.456	-177.103	32.957	-97.244
	5.628	1.943	-185.398	-263.317	47.243	-154.098
34	3.108	1.008	-54.506	-45.908	6.214	-40.627
	6.410	1.643	-112.565	-98.603	11.037	-88.701
	9.956	2.645	-165.236	-140.359	13.847	-129.768
35	3.290	0.519	-59.029	-55.587	5.014	-50.303
	6.329	0.770	-120.406	-120.714	12.532	-106.447
	9.784	0.821	-180.118	-186.370	17.134	-165.104
36	0.831	2.090	-43.912	-170.484	28.152	-41.563
	1.386	4.479	-106.019	-367.901	40.899	-105.198
	1.779	6.517	-151.725	-554.349	53.190	-150.615
37	1.499	0.585	-62.423	-103.411	15.969	-54.654
	2.580	0.948	-130.482	-222.840	29.183	-119.463
	4.052	1.012	-205.702	-330.326	45.272	-193.595
38	0.421	0.987	-47.786	-145.234	8.775	-46.617
	1.011	1.617	-106.337	-280.643	18.060	-103.507
	1.940	2.584	-157.497	-408.997	34.913	-149.382
39	3.588	2.068	-26.398	-17.174	0.889	-18.840
	7.029	4.491	-39.944	-27.548	1.064	-29.783
	10.190	6.660	-65.516	-43.866	3.248	-46.324
40	1.467	0.648	-61.066	-104.343	16.426	-52.898
	2.498	1.578	-113.748	-227.960	33.899	-99.398
	3.547	2.004	-179.804	-348.537	47.050	-161.941
41	2.611	2.350	-25.100	-15.177	5.538	-13.606
	5.676	4.550	-50.360	-31.203	8.099	-29.523
	8.669	7.160	-57.055	-38.444	8.213	-36.958
42	2.591	0.588	-61.292	-65.913	10.602	-50.141
	4.696	0.952	-126.640	-149.893	24.423	-108.106
	7.176	1.016	-191.959	-229.368	37.298	-171.812
43	3.236	0.560	-58.934	-55.200	5.390	-49.557
	5.937	1.177	-119.559	-117.922	15.096	-98.780
	8.922	1.468	-181.071	-182.887	22.936	-154.260
44	1.622	1.146	-53.928	-93.488	22.625	-39.528
	3.192	1.842	-114.385	-193.130	39.515	-90.572
	5.187	2.771	-172.667	-271.550	56.743	-134.196
45	2.563	0.647	-60.839	-65.312	10.894	-48.875
	4.655	1.524	-119.872	-140.762	26.851	-93.504
	6.732	1.947	-184.594	-224.674	40.812	-149.959
46	2.706	0.667	-60.142	-61.476	9.696	-48.109
	5.946	1.277	-118.649	-115.418	15.005	-96.683
	8.994	2.261	-173.775	-162.806	21.766	-137.796
47	1.620	2.237	-48.765	-80.209	45.387	-25.557

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	3.670	4.462	-92.952	-116.695	69.812	-44.472
	5.675	7.034	-124.255	-134.973	87.123	-59.202
48	3.250	1.502	-44.740	-31.887	4.395	-30.608
	6.355	2.573	-98.353	-76.284	10.621	-69.958
	9.898	3.639	-150.054	-116.556	13.418	-109.789
49	0.963	2.574	-63.774	-191.675	22.283	-70.597
	1.559	5.002	-127.934	-389.513	35.323	-136.769
	2.432	7.130	-172.397	-559.426	65.004	-178.902
50	0.636	1.597	-40.162	-157.458	19.201	-37.177
	1.473	3.629	-83.057	-324.448	47.659	-76.970
	1.871	5.610	-126.985	-508.050	60.387	-120.439
51	0.060	0.537	-61.193	-141.301	0.949	-61.173
	0.587	1.136	-120.640	-277.777	9.203	-119.332
	0.945	2.092	-168.963	-423.554	16.557	-166.821
52	3.942	2.469	-10.796	-9.296	-0.012	-9.836
	7.439	4.913	-23.256	-19.212	0.056	-20.274
	11.084	7.003	-48.661	-35.841	0.746	-38.676
53	2.759	1.913	-38.964	-26.729	8.078	-21.999
	5.807	4.063	-66.352	-44.055	11.118	-38.976
	8.401	6.363	-94.172	-61.002	17.672	-53.608
54	2.536	2.937	16.094	-1.131	-0.323	-2.627
	5.108	5.340	-7.061	-15.052	4.944	-15.348
	8.063	8.003	-10.977	-21.259	4.872	-21.752
55	1.067	0.983	-51.892	-122.431	18.544	-45.057
	2.571	1.683	-112.047	-224.913	35.478	-96.261
	4.126	2.833	-165.411	-322.103	58.429	-135.896
56	1.985	1.320	-53.988	-71.217	21.568	-34.904
	3.636	2.511	-107.612	-162.620	44.769	-73.361
	5.661	3.485	-165.406	-238.962	62.153	-115.848
57	2.566	1.300	-53.042	-49.220	12.509	-34.619
	5.223	2.017	-112.869	-110.660	22.682	-81.722
	8.222	3.051	-167.539	-157.566	29.895	-121.811
58	1.949	1.954	-49.093	-55.585	29.065	-23.622
	3.995	4.134	-94.259	-94.622	54.125	-43.224
	5.611	6.326	-142.452	-176.345	98.176	-68.696
59	2.603	1.358	-51.977	-46.576	12.052	-33.383
	5.640	2.451	-105.616	-91.422	18.899	-72.281
	8.823	3.974	-150.340	-123.430	23.825	-102.391
60	1.537	2.927	-32.444	-62.083	42.868	-50.056
	3.121	5.214	-83.721	-148.372	90.465	-77.823
	5.089	7.837	-113.968	-164.794	107.789	-92.749
61	3.927	1.475	-42.194	-29.459	0.341	-32.242
	7.236	3.006	-85.872	-60.122	4.197	-62.304
	10.838	4.100	-136.857	-99.360	6.578	-101.642
62	0.534	2.941	-130.567	-200.919	0.012	-130.855
	1.106	5.427	-200.382	-401.125	10.687	-202.897
	2.044	8.060	-271.486	-597.547	27.970	-281.675
63	1.177	1.634	-44.534	-125.295	31.739	-33.322

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	2.070	3.641	-86.947	-287.018	62.249	-71.588
	2.763	5.213	-127.514	-440.987	82.744	-108.548
64	0.062	1.534	-39.075	-168.164	1.872	-39.048
	0.642	3.095	-79.142	-326.478	19.167	-76.624
	0.984	5.040	-122.349	-509.687	30.134	-119.443
65	2.330	1.935	-44.746	-36.354	16.901	-22.230
	4.833	4.239	-74.312	-54.383	25.032	-37.083
	6.965	6.425	-117.408	-88.331	46.395	-55.901
66	3.200	1.625	-42.178	-29.271	4.491	-28.003
	6.291	3.727	-70.932	-47.528	7.546	-45.986
	9.094	5.591	-110.704	-75.042	15.311	-68.950
67	2.372	1.914	-44.551	-35.696	15.904	-22.515
	5.082	3.810	-84.642	-63.748	24.868	-44.891
	7.627	6.093	-114.228	-81.886	32.532	-59.996
68	1.754	1.608	-50.849	-79.369	29.420	-30.111
	3.386	2.844	-103.819	-171.452	53.507	-67.645
	5.351	4.210	-157.335	-242.682	75.918	-101.647
69	2.293	1.909	-45.791	-38.726	18.033	-22.822
	4.388	4.068	-90.306	-75.900	41.191	-42.331
	6.386	6.001	-139.212	-128.898	68.482	-65.969
70	2.123	0.940	-58.314	-73.034	16.161	-43.074
	4.716	1.650	-118.423	-136.163	26.884	-90.454
	7.219	2.943	-171.869	-187.389	40.296	-125.252
71	1.749	1.707	-50.340	-77.918	31.292	-28.547
	3.677	3.626	-99.800	-136.301	60.832	-52.818
	5.272	5.784	-148.233	-221.386	104.551	-78.804
72	2.766	1.814	-41.549	-29.577	8.600	-23.993
	5.350	3.225	-92.856	-75.202	21.003	-56.306
	8.513	4.800	-136.513	-106.080	25.963	-85.316
73	1.495	2.165	-48.437	-99.597	45.850	-28.576
	2.446	4.215	-91.345	-259.142	78.676	-66.862
	3.681	5.892	-136.263	-380.627	112.008	-99.359
74	0.988	1.065	-49.649	-126.978	18.913	-43.336
	2.172	2.631	-94.732	-250.389	49.269	-77.263
	2.871	4.135	-135.792	-401.507	69.174	-114.587
75	0.471	2.933	-129.715	-201.294	-0.017	-129.972
	0.537	5.461	-210.477	-405.778	0.830	-210.768
	1.046	7.940	-281.077	-606.770	10.063	-283.119
76	2.086	1.620	-51.238	-58.333	22.788	-28.739
	3.872	3.255	-102.100	-134.832	52.118	-58.237
	5.871	4.649	-155.430	-203.724	74.341	-91.607
77	2.527	1.431	-51.368	-46.645	13.331	-31.927
	5.236	3.265	-93.106	-76.622	22.710	-55.543
	7.606	5.118	-138.891	-114.613	38.935	-79.223
78	2.120	1.670	-50.624	-55.024	22.335	-27.710
	4.415	3.544	-97.024	-95.014	40.442	-51.183
	6.415	5.441	-146.216	-149.201	67.494	-75.367
79	2.083	1.656	-50.915	-57.398	23.119	-28.081

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
	4.046	3.539	-100.450	-114.580	51.247	-52.703
	5.830	5.209	-151.114	-190.550	81.230	-81.632
80	3.934	2.530	-8.436	-8.217	-0.027	-8.381
	7.462	5.465	0.332	-9.376	-0.174	-7.702
	10.951	7.970	-9.591	-18.167	-0.237	-16.777
81	0.068	0.473	-63.761	-140.414	1.051	-63.744
	0.541	0.541	-146.860	-272.450	8.299	-146.685
	1.077	1.075	-208.592	-408.027	16.398	-207.195
82	3.519	0.068	-59.563	-64.821	4.002	-58.224
	7.443	0.541	-117.113	-118.018	4.504	-109.487
	10.797	1.077	-175.788	-172.579	9.015	-159.438
83	2.487	1.416	-51.815	-48.243	13.959	-32.245
	4.817	3.255	-98.376	-88.176	31.239	-56.286
	6.972	4.890	-149.199	-142.553	52.534	-84.569
84	1.289	1.651	-45.824	-116.566	33.201	-32.333
	2.992	3.338	-95.926	-198.427	64.960	-61.414
	4.541	5.477	-144.286	-290.211	109.358	-88.473
85	2.511	2.401	-24.653	-14.637	6.319	-13.019
	4.984	5.336	-8.187	-15.864	6.030	-16.147
	7.078	7.967	-30.483	-27.664	16.687	-28.593
86	2.485	2.350	-27.763	-16.655	7.706	-14.037
	4.552	4.930	-55.606	-32.204	21.648	-27.735
	6.664	7.163	-98.043	-64.817	43.173	-45.992
87	3.312	1.595	-42.142	-29.102	3.684	-28.745
	7.241	3.133	-82.815	-57.137	3.999	-59.659
	10.871	5.103	-112.299	-76.266	4.985	-80.706
88	3.255	1.636	-41.496	-28.526	4.001	-27.825
	6.830	3.648	-69.882	-46.910	5.076	-47.906
	9.977	5.759	-97.710	-64.612	7.685	-65.655
89	0.582	1.498	-40.442	-155.830	16.775	-37.895
	0.645	2.970	-79.767	-321.392	18.636	-77.190
	1.007	4.049	-125.250	-470.070	26.782	-121.761
90	0.644	1.466	-41.029	-152.156	18.098	-37.869
	1.069	2.514	-87.442	-298.716	27.357	-83.051
	2.003	3.541	-137.386	-427.681	45.068	-127.438
91	1.518	2.287	-46.038	-93.773	49.117	-27.292
	2.990	5.214	-107.372	-221.264	94.856	-106.554
	4.055	7.832	-168.896	-409.575	121.185	-177.432
92	1.480	2.229	-48.349	-101.850	48.305	-29.361
	2.506	4.788	-108.410	-288.686	75.134	-95.909
	3.442	6.902	-152.260	-453.065	107.528	-136.306
93	2.044	1.331	-53.985	-68.071	20.811	-34.568
	4.126	2.315	-111.740	-141.790	37.728	-76.681
	6.589	3.654	-164.765	-192.992	51.873	-110.566
94	2.047	1.375	-53.556	-66.837	21.223	-33.641
	4.514	2.758	-105.951	-116.667	35.436	-66.591
	6.648	4.360	-157.207	-173.064	56.984	-95.561
95	1.583	1.253	-52.400	-94.831	24.629	-37.346

Soil element	x-coord. [m]	y-coord. [m]	σ_{xx} [kN/m²]	σ_{yy} [kN/m²]	σ_{xy} [kN/m²]	σ_{zz} [kN/m²]
96	3.289 4.580 1.536 2.780 3.829	2.876 4.463 1.211 2.756 3.800	-102.850 -149.088 -52.473 -98.439 -149.004	-177.504 -293.000 -97.908 -216.458 -340.356	55.111 86.845 24.068 54.538 73.796	-67.419 -100.413 -38.371 -72.159 -115.802

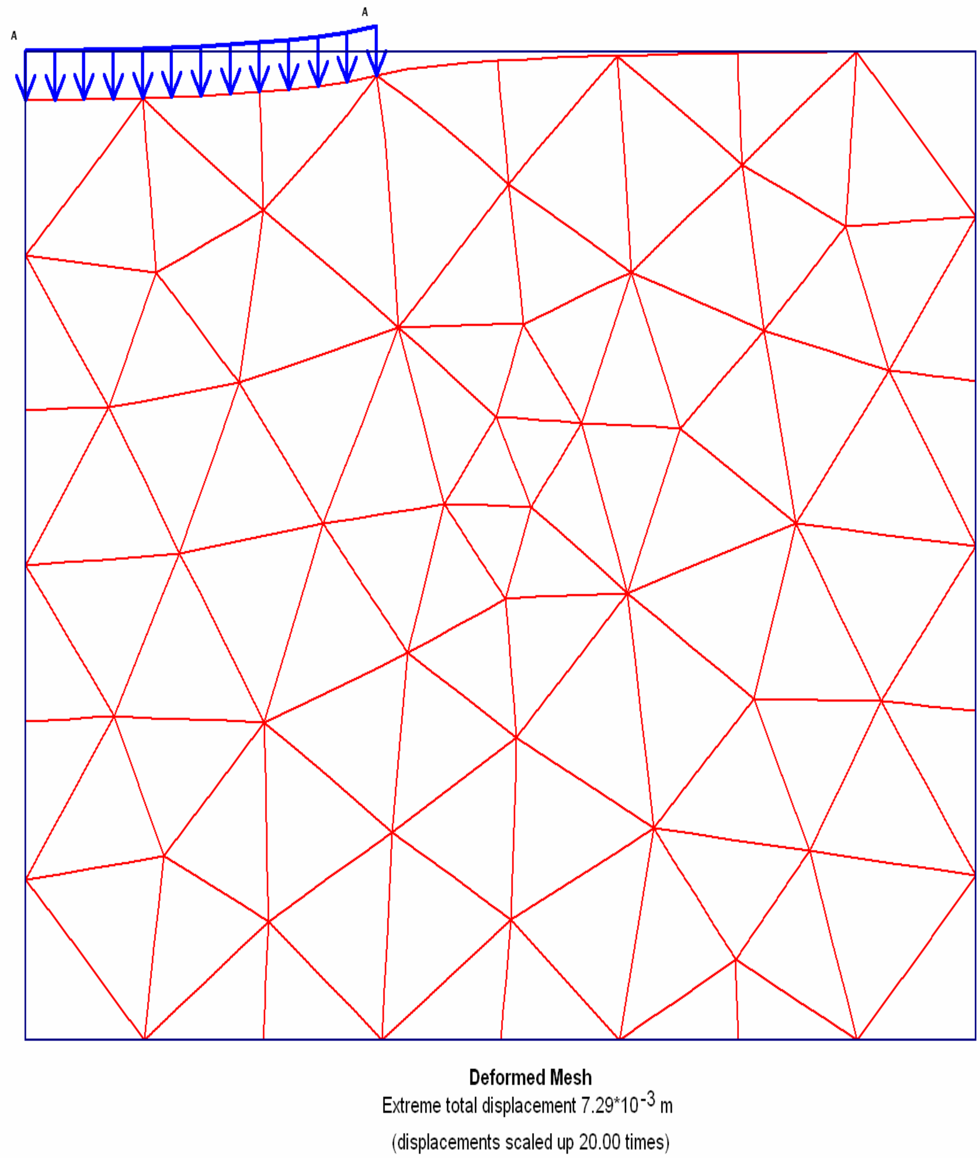
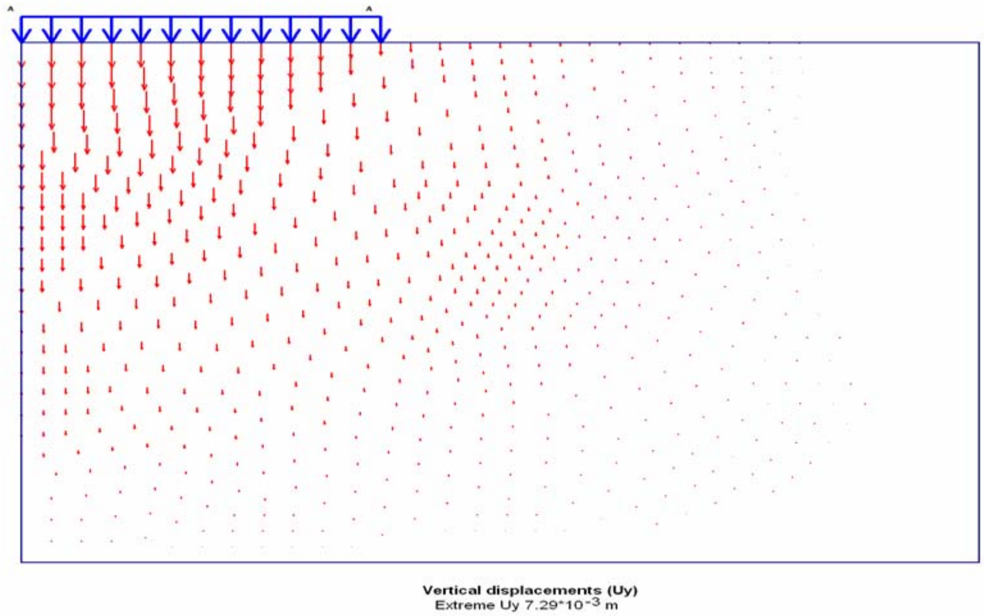


Figure 7.47 Problem geometry with mesh generation



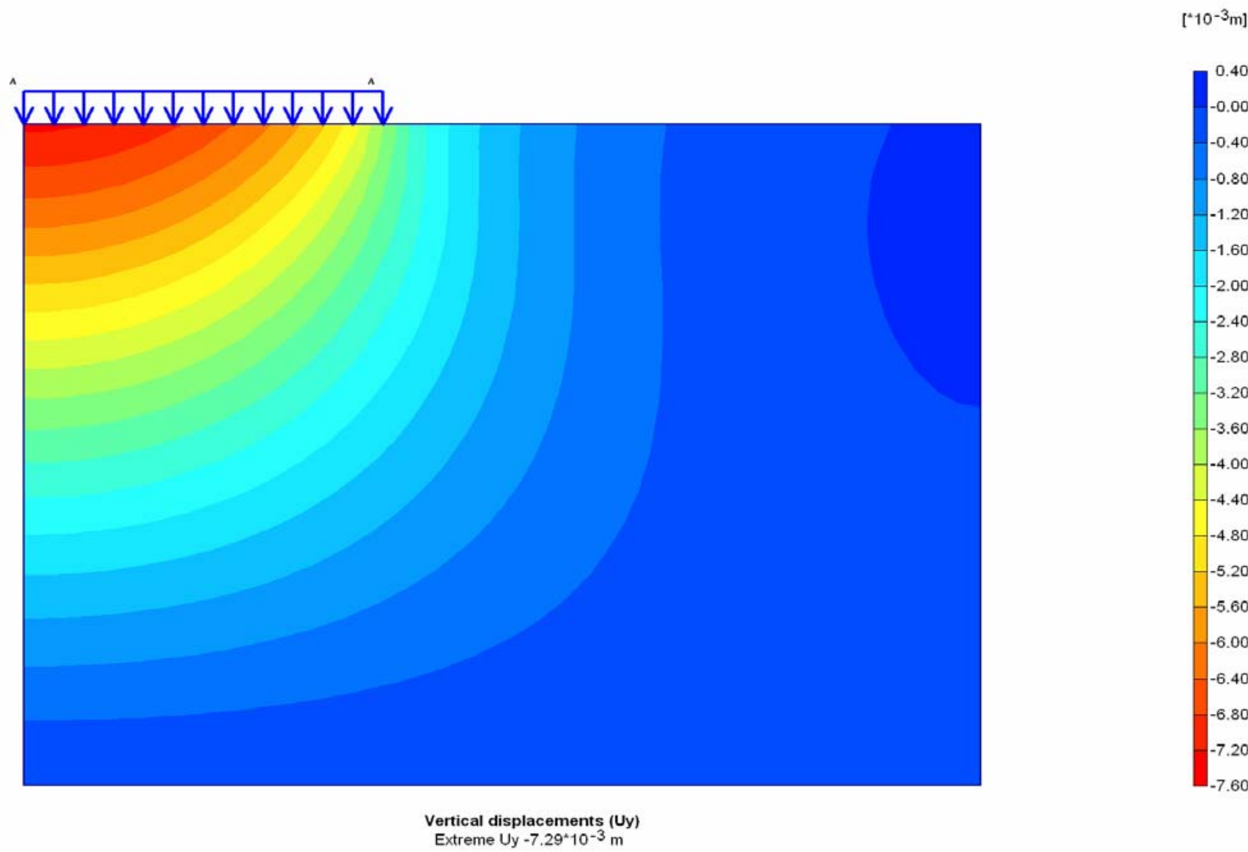
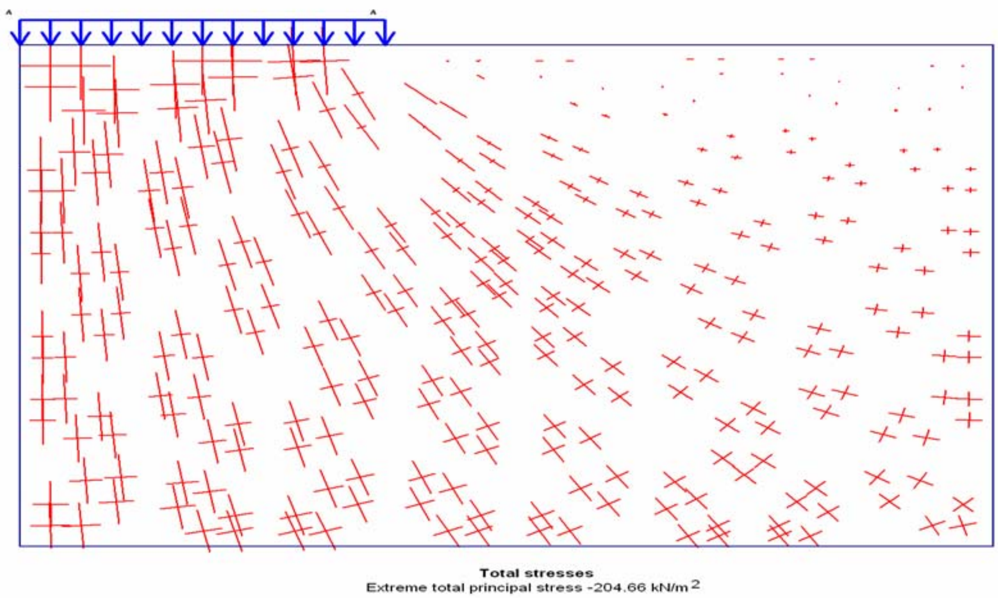


Fig. 4.48 Vertical settlement in soil type A for circular footing (r=1.5m).



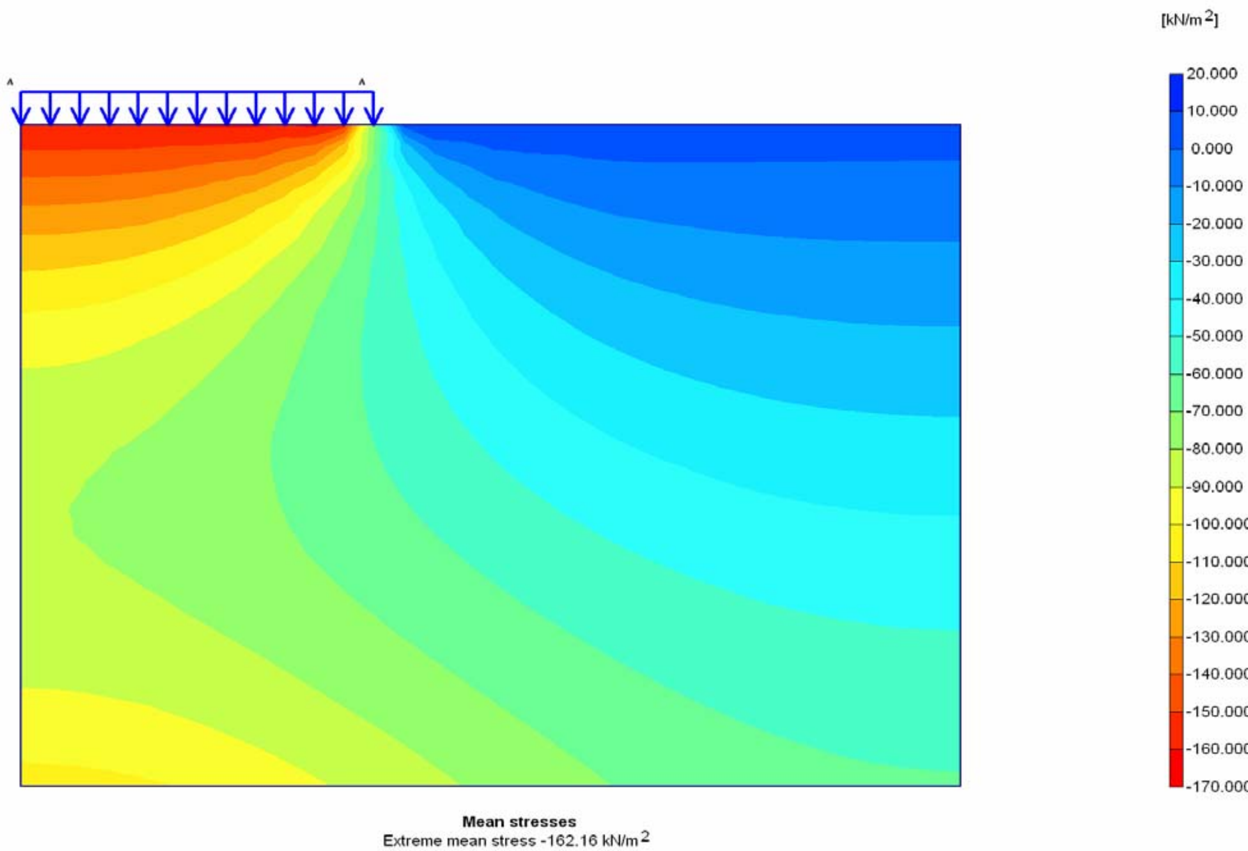


Fig. 4.49 Mean stresses in soil type A for circular footing (r=1.5m)

Table 7.50 Soil data sets parameters for soil type B

Linear Elastic		Sand 101	Sand 102	Sand 103	Sand 104
Type		Drained	Drained	Drained	Drained
γ_{unsat}	[kN/m³]	18.00	18.00	18.00	18.00
γ_{sat}	[kN/m³]	20.00	20.00	20.00	20.00
k_x	[m/day]	0.000	0.000	0.000	0.000
k_y	[m/day]	0.000	0.000	0.000	0.000
e_{init}	[-]	0.500	0.500	0.500	0.500
c_k	[-]	1E15	1E15	1E15	1E15
E_{ref}	[kN/m²]	46109.95	44796.46	41293.22	37325.40
ν	[-]	0.300	0.300	0.300	0.300
G_{ref}	[kN/m²]	17734.596	17229.408	15882.008	14355.923
E_{oed}	[kN/m²]	62071.087	60302.927	55587.027	50245.731
E_{incr}	[kN/m²/m]	0.00	0.00	0.00	0.00
y_{ref}	[m]	0.000	0.000	0.000	0.000
R_{inter}	[-]	1.000	1.000	1.000	1.000
Interface permeability		Neutral	Neutral	Neutral	Neutral

Linear Elastic		Sand 105	Sand 106
Type		Drained	Drained
γ_{unsat}	[kN/m³]	18.00	18.00
γ_{sat}	[kN/m³]	20.00	20.00
k_x	[m/day]	0.000	0.000
k_y	[m/day]	0.000	0.000
e_{init}	[-]	0.500	0.500
c_k	[-]	1E15	1E15
E_{ref}	[kN/m²]	32576.71	27854.83
ν	[-]	0.300	0.300
G_{ref}	[kN/m²]	12529.504	10713.396
E_{oed}	[kN/m²]	43853.263	37496.887
E_{incr}	[kN/m²/m]	0.00	0.00
y_{ref}	[m]	0.000	0.000
R_{inter}	[-]	1.000	1.000
Interface permeability		Neutral	Neutral

Table7.51 Table of deformations for soil type B

Node no.	x-coord.	y-coord.	uy [10^-3 m]
38	0.125	0	0
39	0.25	0	0
40	0.375	0	0
44	0	0	0
59	0.5	0	0
63	0.625	0	0
64	0.75	0	0
65	0.875	0	0
147	1	0	0
148	1.125	0	0
149	1.25	0	0
150	1.375	0	0
181	1.5	0	0
185	1.625	0	0
186	1.75	0	0
187	1.875	0	0
217	2	0	0
218	2.125	0	0
219	2.25	0	0
220	2.375	0	0
227	2.5	0	0
231	2.625	0	0
232	2.75	0	0
233	2.875	0	0
249	3	0	0
250	3.125	0	0
251	3.25	0	0
252	3.375	0	0
259	3.5	0	0
263	3.625	0	0
264	3.75	0	0
265	3.875	0	0
506	3.188	0.5	-12.358
507	3.125	0.5	-13.47
508	3.063	0.5	-14.69
519	3.25	0.5	-11.35
520	3.438	0.5	-8.901
521	3.375	0.5	-9.625
522	3.313	0.5	-10.44
533	3.5	0.5	-8.264
534	3.688	0.5	-6.845

535	3.625	0.5	-7.239
536	3.563	0.5	-7.711
547	4	0	0
551	4	0.125	-2.501
552	4	0.25	-4.235
553	4	0.375	-5.293
558	4	0.5	-5.758
562	3.75	0.5	-6.526
563	4	0.5	-5.973
564	3.938	0.5	-6.007
565	3.875	0.5	-6.109
566	3.813	0.5	-6.282
1114	4	0.5	-6.156
1115	4	0.563	-5.944
1116	4	0.625	-5.635
1117	4	0.688	-5.238
1277	4	0.75	-4.76
1281	4	0.813	-4.209
1282	4	0.875	-3.591
1283	4	0.938	-2.912
1287	4	1	-2.181
1291	4	1	-2.272
1292	3.938	1	-2.337
1293	3.875	1	-2.534
1294	3.813	1	-2.866
1397	0	1.688	-1.023
1511	3.188	1	-14.657
1512	3.125	1	-16.836
1513	3.063	1	-19.233
1514	3	1	-21.862
1525	3.438	1	-7.923
1526	3.375	1	-9.328
1527	3.313	1	-10.913
1528	3.25	1	-12.686
1544	3.5	1	-6.689
1545	3.75	1	-3.337
1546	3.688	1	-3.949
1547	3.625	1	-4.708
1548	3.563	1	-5.62
1635	0	1.75	-1.086
1636	0	1.938	-1.298
1637	0	1.875	-1.223
1638	0	1.813	-1.152
1648	0	2	-1.375
1649	0.188	2	-1.353

1650	0.125	2	-1.376
1651	0.063	2	-1.391
1652	0	2	-1.395
1773	0.438	2	-1.183
1774	0.375	2	-1.235
1775	0.313	2	-1.281
1776	0.25	2	-1.321
1937	0.5	2	-1.127
1939	0.625	2	-1.008
1940	0.563	2	-1.069
2054	4	1	-2.351
2055	4	1.063	-1.443
2056	4	1.125	-507.337
2057	4	1.188	449.628
2219	1	2	-661.459
2220	1.188	2	-520.377
2221	1.125	2	-564.504
2222	1.063	2	-611.53
2223	0	2	-1.414
2227	0	2.188	-1.706
2228	0	2.125	-1.602
2229	0	2.063	-1.505
2295	4	1.25	1.422
2296	4	1.313	2.404
2297	4	1.375	3.391
2298	4	1.438	4.376
2305	4	1.5	5.355
2309	4	1.5	5.336
2310	3.938	1.5	5.248
2311	3.875	1.5	4.982
2312	3.813	1.5	4.535
2377	0	2.25	-1.82
2378	0	2.438	-2.217
2379	0	2.375	-2.076
2380	0	2.313	-1.943
2390	0	2.5	-2.36
2391	0.188	2.5	-2.292
2392	0.125	2.5	-2.354
2393	0.063	2.5	-2.393
2394	0	2.5	-2.403
2535	3.75	1.5	3.901
2536	3.688	1.5	3.075
2537	3.625	1.5	2.05
2554	3.438	1.5	-2.299
2555	3.375	1.5	-4.205

2556	3.313	1.5	-6.357
2615	0.438	2.5	-1.841
2616	0.375	2.5	-1.976
2617	0.313	2.5	-2.099
2618	0.25	2.5	-2.205
2715	0.5	2.5	-1.699
2716	0.688	2.5	-1.282
2717	0.625	2.5	-1.415
2718	0.563	2.5	-1.555
2815	0	2.5	-2.445
2819	0	2.875	-3.609
2820	0	2.75	-3.214
2821	0	2.625	-2.814
2849	0.75	2.5	-1.159
2852	0.813	2.5	-1.046
3011	0	3	-3.919
3015	0.375	3	-3.241
3016	0.25	3	-3.643
3017	0.125	3	-3.858
3328	0.75	3	-1.206
3329	0.625	3	-1.589
3365	2.75	2	-38.348
3366	2.938	2	-22.153
3367	2.875	2	-27.085
3368	2.813	2	-32.473
3458	3.188	2	-6.436
3459	3.125	2	-9.813
3950	3.188	2.5	-3.653
3951	3.125	2.5	-7.098
4075	3.25	3	-3.971

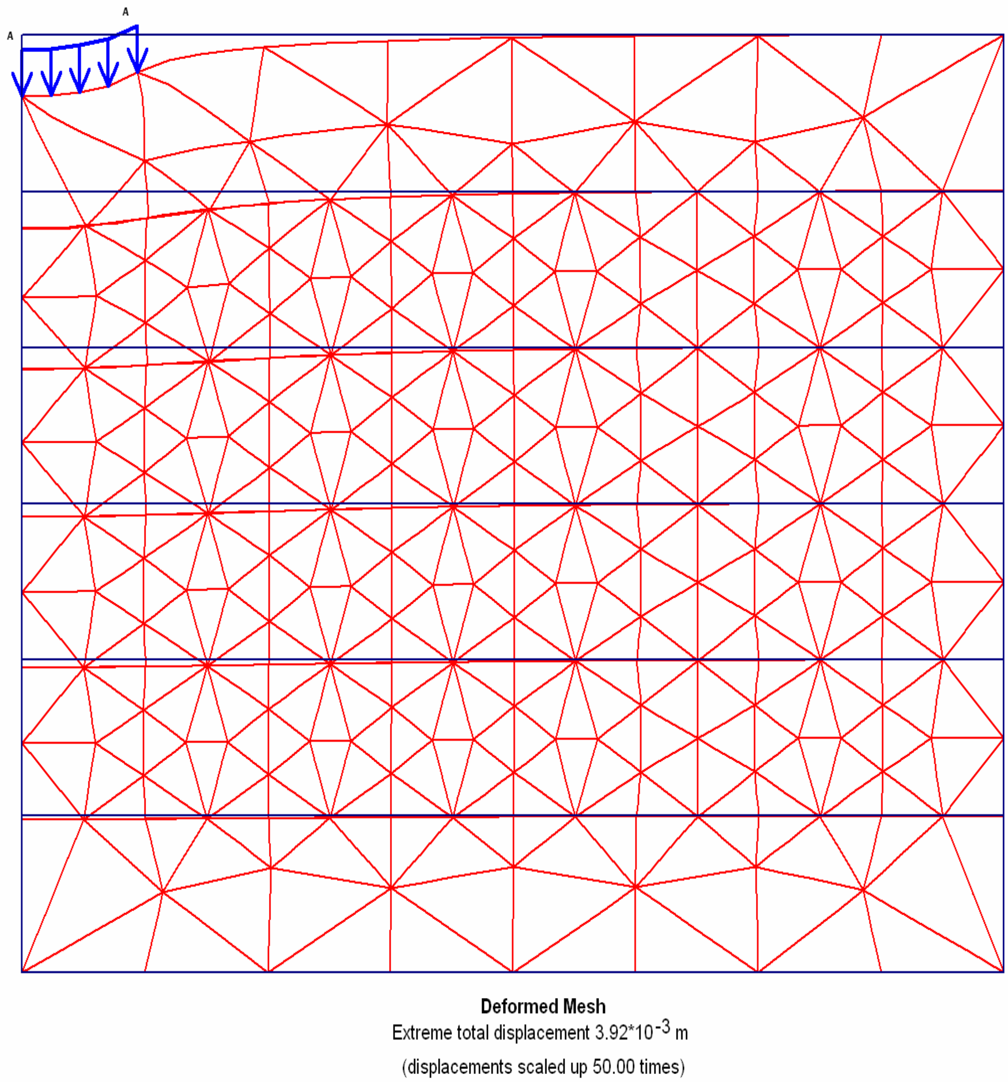


Fig.7.52 Problem geometry with mesh generation

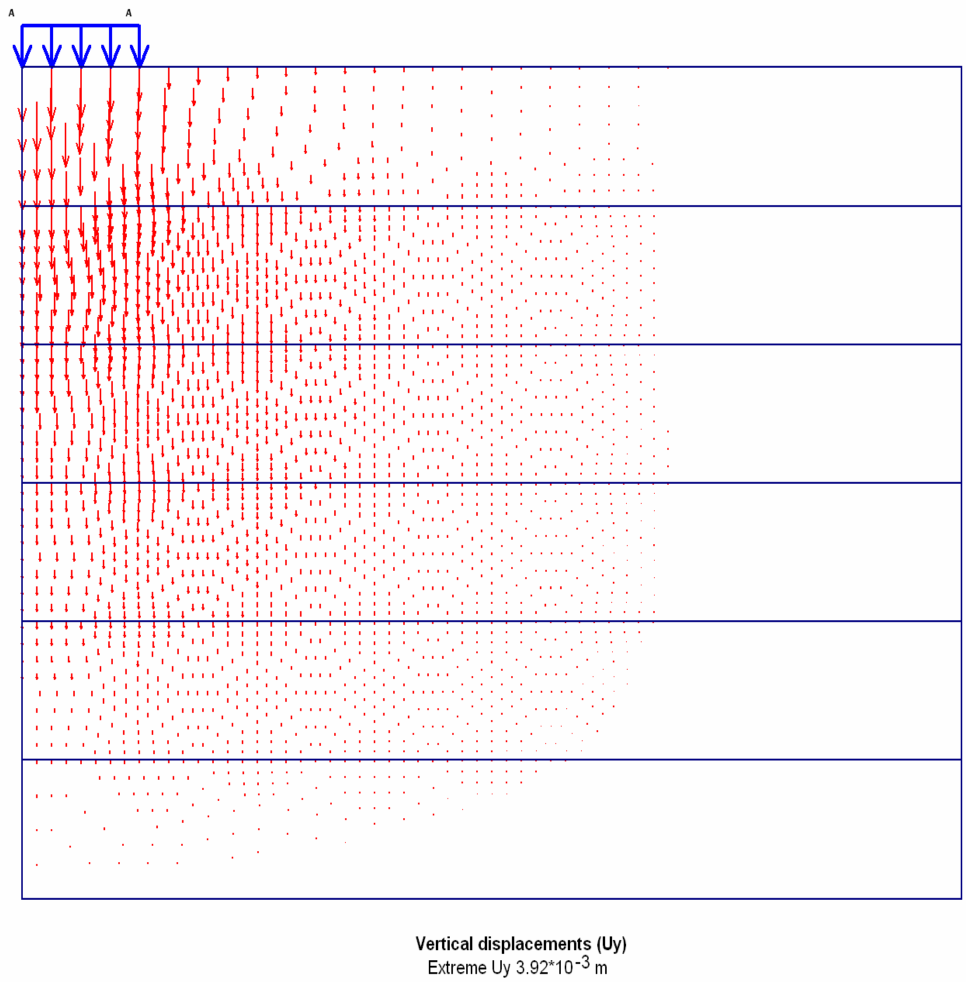


Fig.7.53 Vertical settlement for circular footing (r=0.5m) in soil type B.

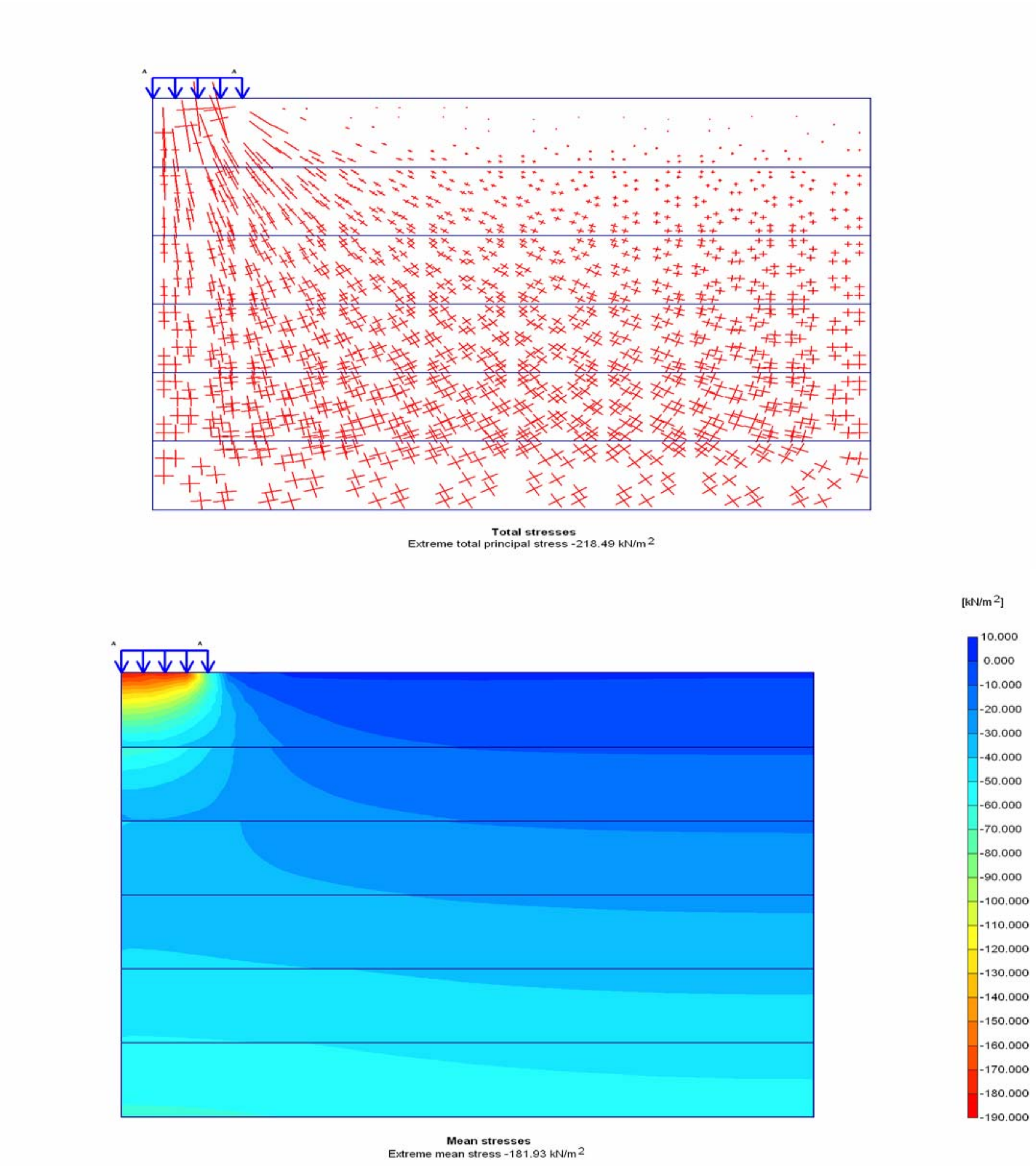


Fig.7.54 Mean stresses for circular footing (r=0.5m) in soil type B.

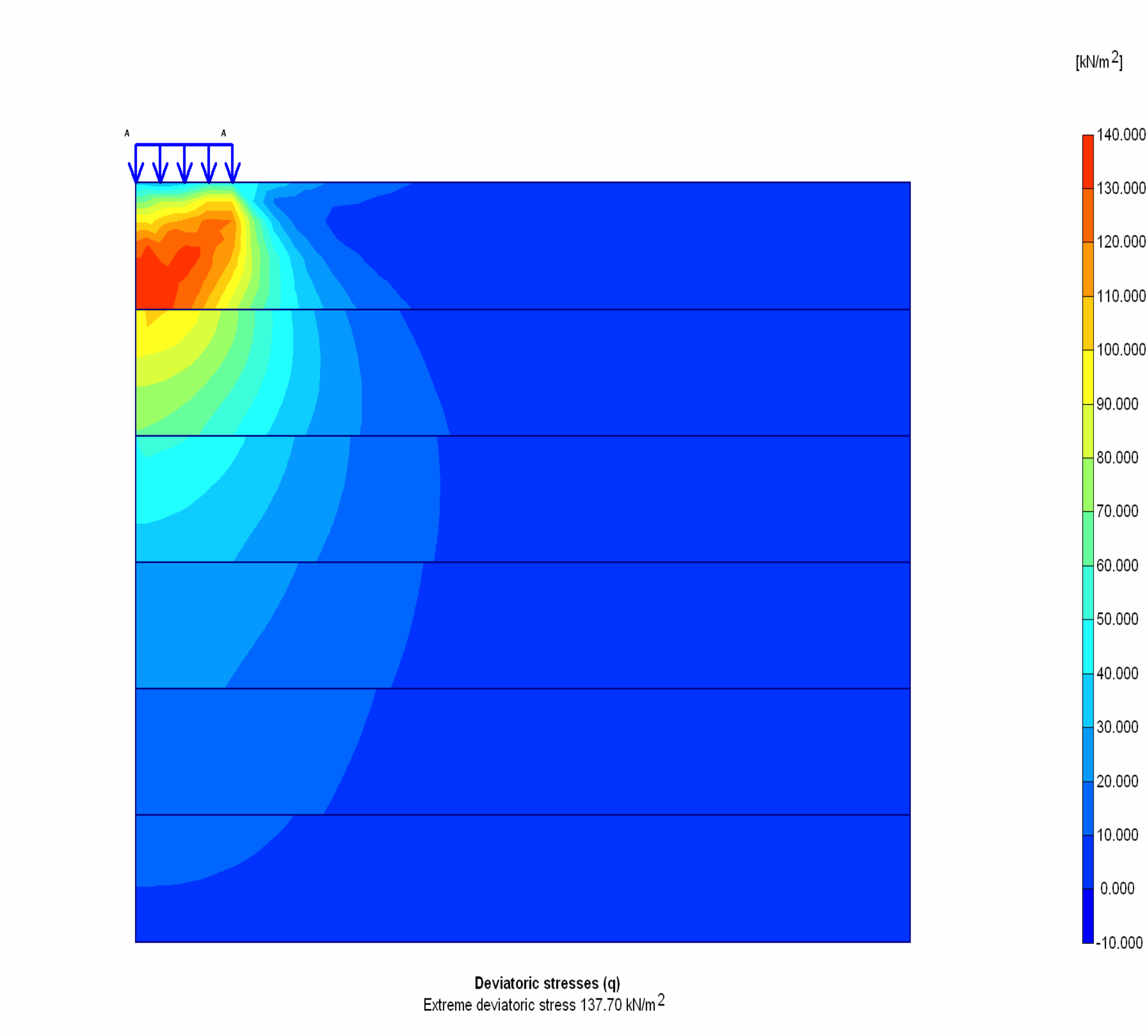


Fig.7.55 Deviatoric stresses for circular footing (r=0.5m) in soil type B.

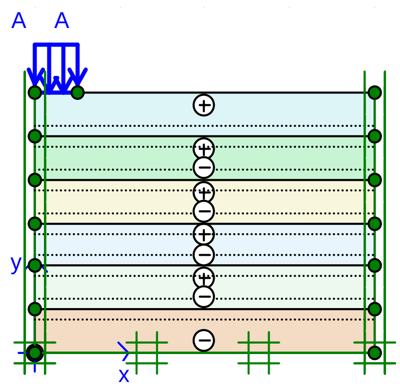


Fig.7.57 Soil geometry with boundary

Table 7.56 Soil data sets parameters for soil type B

Linear Elastic		Sand 101	Sand 102	Sand 103	Sand 104
Type		Drained	Drained	Drained	Drained
γ_{unsat}	[kN/m³]	18.00	18.00	18.00	18.00
γ_{sat}	[kN/m³]	20.00	20.00	20.00	20.00
k_x	[m/day]	0.000	0.000	0.000	0.000
k_y	[m/day]	0.000	0.000	0.000	0.000
e_{init}	[-]	0.500	0.500	0.500	0.500
c_k	[-]	1E15	1E15	1E15	1E15
E_{ref}	[kN/m²]	34735.95	31573.69	27449.85	23487.73
ν	[-]	0.300	0.300	0.300	0.300
G_{ref}	[kN/m²]	13359.981	12143.727	10557.635	9033.742
E_{oed}	[kN/m²]	46759.933	42503.044	36951.721	31618.098
E_{incr}	[kN/m²/m]	0.00	0.00	0.00	0.00
y_{ref}	[m]	0.000	0.000	0.000	0.000
R_{inter}	[-]	1.000	1.000	1.000	1.000
Interface permeability		Neutral	Neutral	Neutral	Neutral

Linear Elastic		Sand 105	Sand 106
Type		Drained	Drained
□$_{unsat}$	[kN/m³]	18.00	18.00
□$_{sat}$	[kN/m³]	20.00	20.00
k_x	[m/day]	0.000	0.000
k_y	[m/day]	0.000	0.000
e_{init}	[-]	0.500	0.500
c_k	[-]	1E15	1E15
E_{ref}	[kN/m²]	19151.57	17402.69
□	[-]	0.300	0.300
G_{ref}	[kN/m²]	7365.988	6693.342
E_{oed}	[kN/m²]	25780.960	23426.698
E_{incr}	[kN/m²/m]	0.00	0.00
y_{ref}	[m]	0.000	0.000
R_{inter}	[-]	1.000	1.000
Interface permeability		Neutral	Neutral

Table 7.57 Table of deformations for soil type B

Node no.	x-coord.	y-coord.	uy [10^-3 m]
10	0	0.5	-1.129
11	0.188	0.5	-1.152
12	0.125	0.5	-1.159
13	0.063	0.5	-1.163
14	0	0.5	-1.164
31	0.438	0.5	-1.1
32	0.375	0.5	-1.116
33	0.313	0.5	-1.131
34	0.25	0.5	-1.143
38	0.125	0	0
39	0.25	0	0
40	0.375	0	0
44	0	0	0
55	0.5	0.5	-1.08
56	0.688	0.5	-1.011
57	0.625	0.5	-1.036
58	0.563	0.5	-1.059
59	0.5	0	0
63	0.625	0	0
64	0.75	0	0
65	0.875	0	0
99	0	0.5	-1.197
103	0	0.688	-1.686
104	0	0.625	-1.518
105	0	0.563	-1.355
147	1	0	0
148	1.125	0	0
149	1.25	0	0
150	1.375	0	0
181	1.5	0	0
185	1.625	0	0
186	1.75	0	0
187	1.875	0	0
217	2	0	0
218	2.125	0	0
219	2.25	0	0
220	2.375	0	0
227	2.5	0	0

231	2.625	0	0
232	2.75	0	0
233	2.875	0	0
249	3	0	0
250	3.125	0	0
251	3.25	0	0
252	3.375	0	0
259	3.5	0	0
263	3.625	0	0
264	3.75	0	0
265	3.875	0	0
291	0	0.75	-1.858
292	0	0.938	-2.404
293	0	0.875	-2.217
294	0	0.813	-2.035
304	0	1	-2.593
305	0.188	1	-2.603
306	0.125	1	-2.62
307	0.063	1	-2.631
308	0	1	-2.634
367	0.438	1	-2.47
368	0.375	1	-2.513
369	0.313	1	-2.549
370	0.25	1	-2.58
613	0.5	1	-2.422
614	0.688	1	-2.249
615	0.625	1	-2.311
616	0.563	1	-2.369
665	0	1	-2.667
669	0	1.188	-3.21
670	0	1.125	-3.025
671	0	1.063	-2.844
685	0.75	1	-2.183
686	0.938	1	-1.966
687	0.875	1	-2.041
688	0.813	1	-2.113
735	1	1	-1.89
736	1.188	1	-1.654
737	1.125	1	-1.733
738	1.063	1	-1.812
793	0	1.25	-3.401
794	0	1.438	-4.009

795	0	1.375	-3.801
796	0	1.313	-3.598
806	0	1.5	-4.222
807	0.188	1.5	-4.211
808	0.125	1.5	-4.242
809	0.063	1.5	-4.262
810	0	1.5	-4.267
901	1.25	1	-1.575
902	1.438	1	-1.343
903	1.375	1	-1.419
904	1.313	1	-1.497
983	1.5	1	-1.268
984	1.688	1	-1.056
985	1.625	1	-1.125
986	1.563	1	-1.196
1173	0.438	1.5	-3.969
1174	0.375	1.5	-4.046
1175	0.313	1.5	-4.113
1176	0.25	1.5	-4.168
1341	0.5	1.5	-3.881
1342	0.688	1.5	-3.565
1343	0.625	1.5	-3.678
1344	0.563	1.5	-3.784
1393	0	1.5	-4.305
1397	0	1.688	-4.958
1398	0	1.625	-4.734
1399	0	1.563	-4.517
1559	0.75	1.5	-3.445
1560	0.938	1.5	-3.055
1561	0.875	1.5	-3.189
1562	0.813	1.5	-3.319
1609	1	1.5	-2.918
1610	1.188	1.5	-2.504
1611	1.125	1.5	-2.642
1612	1.063	1.5	-2.78
1635	0	1.75	-5.189
1636	0	1.938	-5.928
1637	0	1.875	-5.674
1638	0	1.813	-5.427
1648	0	2	-6.185
1649	0.188	2	-6.152
1650	0.125	2	-6.202

1651	0.063	2	-6.233
1652	0	2	-6.242
1759	1.25	1.5	-2.367
1760	1.438	1.5	-1.973
1761	1.375	1.5	-2.101
1762	1.313	1.5	-2.232
1773	0.438	2	-5.752
1774	0.375	2	-5.88
1775	0.313	2	-5.99
1776	0.25	2	-6.08
1823	1.5	1.5	-1.848
1824	1.688	1.5	-1.502
1825	1.625	1.5	-1.613
1826	1.563	1.5	-1.728
1937	0.5	2	-5.606
1938	0.688	2	-5.076
1939	0.625	2	-5.267
1940	0.563	2	-5.444
1983	1.75	1.5	-1.396
1984	1.938	1.5	-1.109
1985	1.875	1.5	-1.2
1986	1.813	1.5	-1.296
1997	0.75	2	-4.873
1998	0.938	2	-4.216
1999	0.875	2	-4.441
2000	0.813	2	-4.661
2047	2	1.5	-1.023
2219	1	2	-3.989
2220	1.188	2	-3.312
2221	1.125	2	-3.534
2222	1.063	2	-3.761
2223	0	2	-6.291
2227	0	2.188	-7.095
2228	0	2.125	-6.822
2229	0	2.063	-6.554
2305	4	1.5	-5.229
2309	4	1.5	-5.595
2310	3.938	1.5	-6.238
2311	3.875	1.5	-8.185
2312	3.813	1.5	-11.461
2377	0	2.25	-7.374
2378	0	2.438	-8.236

2379	0	2.375	-7.946
2380	0	2.313	-7.658
2390	0	2.5	-8.523
2391	0.188	2.5	-8.469
2392	0.125	2.5	-8.537
2393	0.063	2.5	-8.579
2394	0	2.5	-8.591
2601	1.25	2	-3.096
2602	1.438	2	-2.492
2603	1.375	2	-2.684
2604	1.313	2	-2.886
2615	0.438	2.5	-7.912
2616	0.375	2.5	-8.094
2617	0.313	2.5	-8.247
2618	0.25	2.5	-8.372
2665	1.5	2	-2.309
2666	1.688	2	-1.818
2667	1.625	2	-1.972
2668	1.563	2	-2.136
2715	0.5	2.5	-7.701
2716	0.688	2.5	-6.89
2717	0.625	2.5	-7.189
2718	0.563	2.5	-7.46
2815	0	2.5	-8.653
2819	0	2.875	-10.212
2820	0	2.75	-9.745
2821	0	2.625	-9.218
2835	1.75	2	-1.674
2836	1.938	2	-1.293
2837	1.875	2	-1.411
2838	1.813	2	-1.538
2849	0.75	2.5	-6.563
2850	0.938	2.5	-5.462
2851	0.875	2.5	-5.844
2852	0.813	2.5	-6.213
2899	2	2	-1.183
2902	2.063	2	-1.08
2949	1	2.5	-5.075
2950	1.188	2.5	-3.965
2951	1.125	2.5	-4.32
2952	1.063	2.5	-4.692
3011	0	3	-10.597

3015	0.375	3	-10.1
3016	0.25	3	-10.378
3017	0.125	3	-10.542
3119	1.25	2.5	-3.632
3120	1.438	2.5	-2.778
3121	1.375	2.5	-3.039
3122	1.313	2.5	-3.323
3126	4	1.5	-5.908
3128	4	1.625	5.815
3129	4	1.688	11.431
3285	1.5	2.5	-2.539
3286	1.688	2.5	-1.935
3287	1.625	2.5	-2.119
3288	1.563	2.5	-2.32
3323	0.5	3	-9.694
3327	0.875	3	-7.522
3328	0.75	3	-8.451
3329	0.625	3	-9.152
3379	1.75	2.5	-1.766
3380	1.938	2.5	-1.338
3381	1.875	2.5	-1.469
3382	1.813	2.5	-1.611
3571	2.125	2.5	-1.003
3572	2.063	2.5	-1.106
3573	1	3	-6.006
3577	1.375	3	-3.043
3578	1.25	3	-3.681
3579	1.125	3	-4.551
3596	3.563	2	-1.776
3680	1.5	3	-2.526
3684	1.875	3	-1.467
3685	1.75	3	-1.757
3686	1.625	3	-2.104
3843	2	3	-1.221

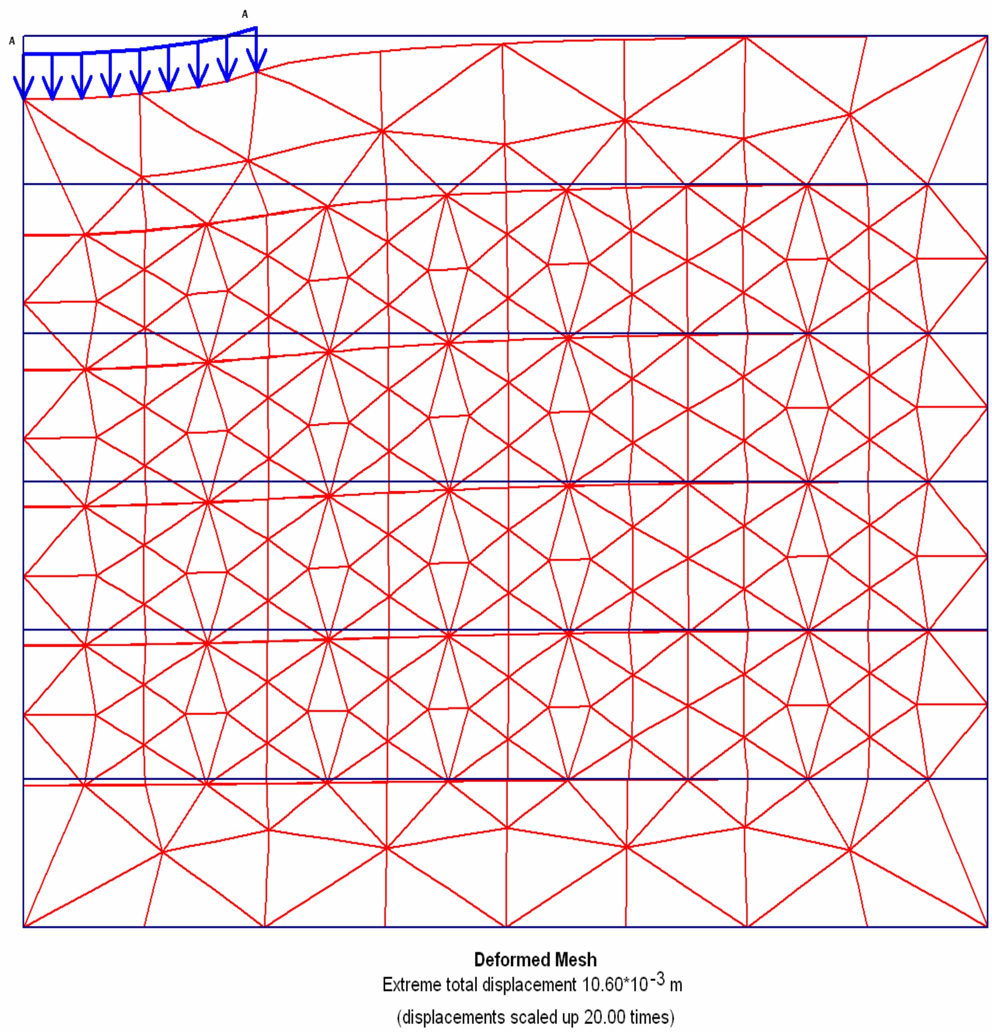


Fig.7.58 Problem geometry with mesh generation

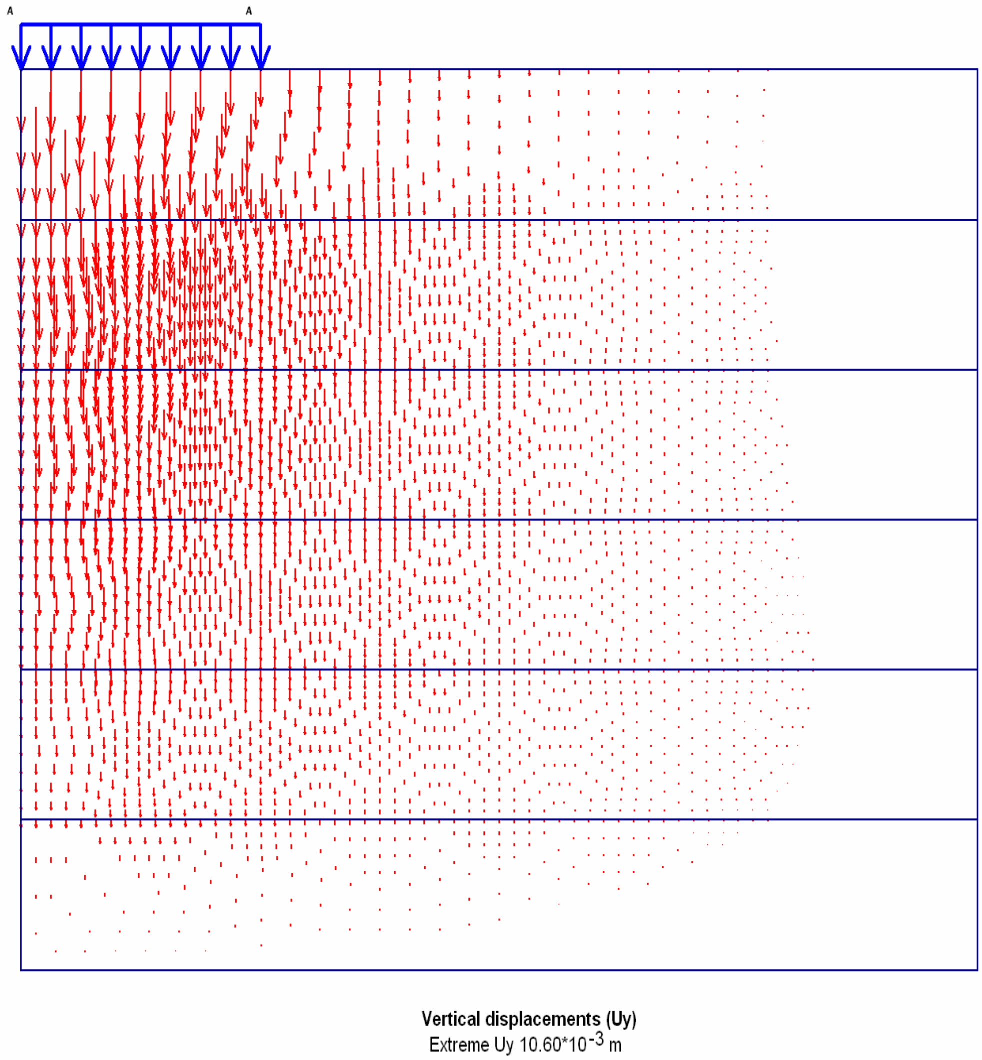
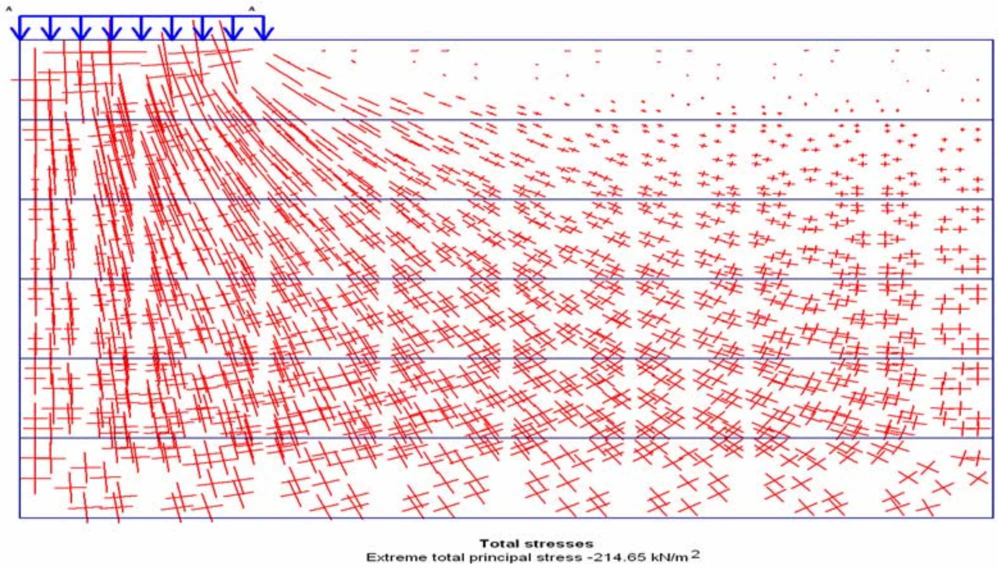


Fig.7.59 Vertical settlement in soil type B for circular footing (r=1.0m).



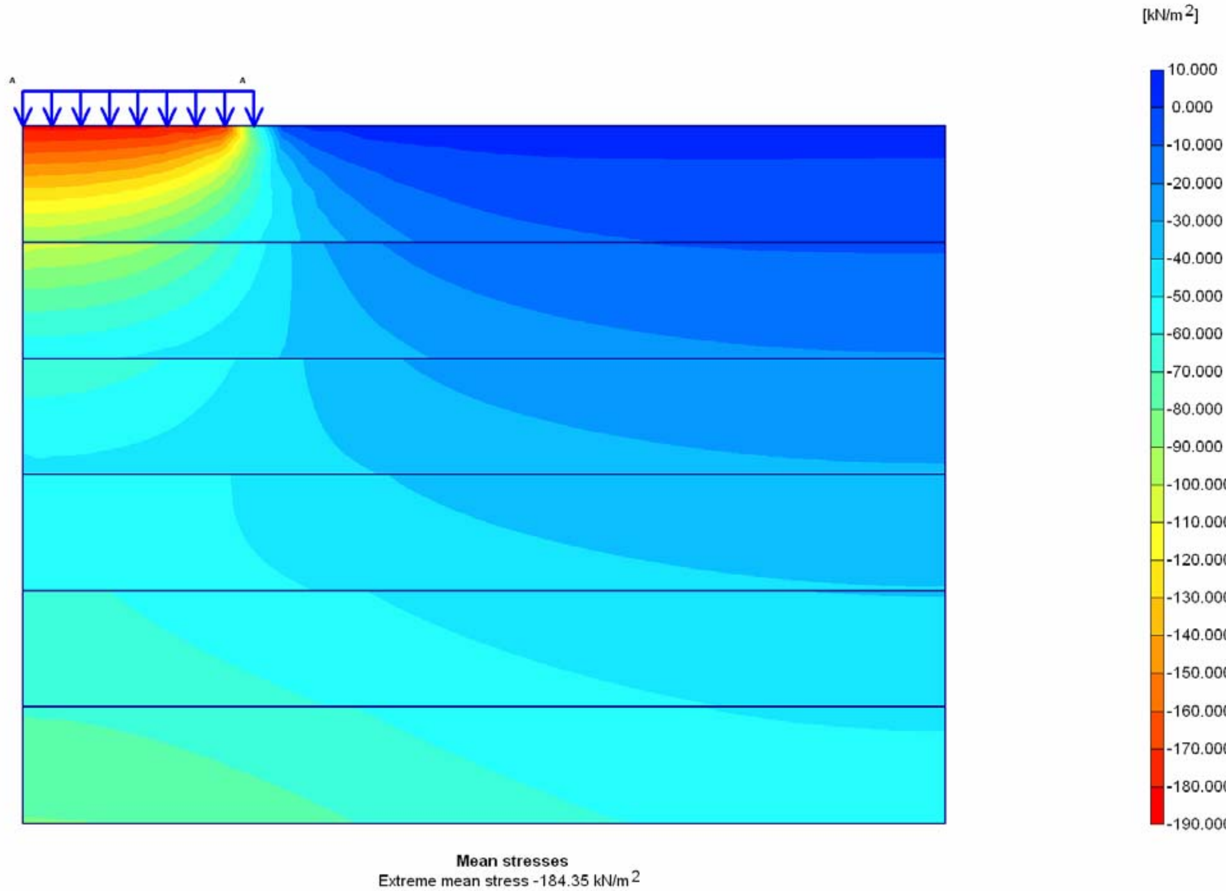


Fig.7.60 Mean stresses in soil type B for circular footing (r=1.0m).

Table 7.61 Soil data sets parameters for soil type B

Linear Elastic		Sand 101	Sand 102	Sand 103	Sand 104
Type		Drained	Drained	Drained	Drained
γ_{unsat}	[kN/m³]	18.00	18.00	18.00	18.00
γ_{sat}	[kN/m³]	20.00	20.00	20.00	20.00
k_x	[m/day]	0.000	0.000	0.000	0.000
k_y	[m/day]	0.000	0.000	0.000	0.000
e_{init}	[-]	0.500	0.500	0.500	0.500
c_k	[-]	1E15	1E15	1E15	1E15
E_{ref}	[kN/m²]	46109.95	44796.46	41293.22	37325.40
ν	[-]	0.300	0.300	0.300	0.300
G_{ref}	[kN/m²]	17734.596	17229.408	15882.008	14355.923
E_{oed}	[kN/m²]	62071.087	60302.927	55587.027	50245.731
E_{incr}	[kN/m²/m]	0.00	0.00	0.00	0.00
y_{ref}	[m]	0.000	0.000	0.000	0.000
R_{inter}	[-]	1.000	1.000	1.000	1.000
Interface permeability		Neutral	Neutral	Neutral	Neutral

Linear Elastic		Sand 105	Sand 106
Type		Drained	Drained
γ_{unsat}	[kN/m³]	18.00	18.00
γ_{sat}	[kN/m³]	20.00	20.00
k_x	[m/day]	0.000	0.000
k_y	[m/day]	0.000	0.000
e_{init}	[-]	0.500	0.500
c_k	[-]	1E15	1E15
E_{ref}	[kN/m²]	32576.71	27854.83
ν	[-]	0.300	0.300
G_{ref}	[kN/m²]	12529.504	10713.396
E_{oed}	[kN/m²]	43853.263	37496.887
E_{incr}	[kN/m²/m]	0.00	0.00
y_{ref}	[m]	0.000	0.000
R_{inter}	[-]	1.000	1.000
Interface permeability		Neutral	Neutral

Table 7.62 Table of deformations for soil type B

Node no.	x-coord.	y-coord.	uy [10^-3 m]
10	0	0.5	-1.325
11	0.188	0.5	-1.356
12	0.125	0.5	-1.362
13	0.063	0.5	-1.365
14	0	0.5	-1.366
31	0.438	0.5	-1.311
32	0.375	0.5	-1.326
33	0.313	0.5	-1.338
34	0.25	0.5	-1.348
38	0.125	0	0
39	0.25	0	0
40	0.375	0	0
44	0	0	0
55	0.5	0.5	-1.295
56	0.688	0.5	-1.233
57	0.625	0.5	-1.255
58	0.563	0.5	-1.276
59	0.5	0	0
63	0.625	0	0
64	0.75	0	0
65	0.875	0	0
95	0.75	0.5	-1.209
96	0.938	0.5	-1.127
97	0.875	0.5	-1.156
98	0.813	0.5	-1.183
99	0	0.5	-1.401
103	0	0.688	-1.927
104	0	0.625	-1.748
105	0	0.563	-1.573
147	1	0	0
148	1.125	0	0
149	1.25	0	0
150	1.375	0	0
167	1	0.5	-1.097
168	1.188	0.5	-1
169	1.125	0.5	-1.033

170	1.063	0.5	-1.066
181	1.5	0	0
185	1.625	0	0
186	1.75	0	0
187	1.875	0	0
217	2	0	0
218	2.125	0	0
219	2.25	0	0
220	2.375	0	0
227	2.5	0	0
231	2.625	0	0
232	2.75	0	0
233	2.875	0	0
249	3	0	0
250	3.125	0	0
251	3.25	0	0
252	3.375	0	0
259	3.5	0	0
263	3.625	0	0
264	3.75	0	0
265	3.875	0	0
291	0	0.75	-2.11
292	0	0.938	-2.68
293	0	0.875	-2.486
294	0	0.813	-2.296
304	0	1	-2.876
305	0.188	1	-2.895
306	0.125	1	-2.907
307	0.063	1	-2.915
308	0	1	-2.917
367	0.438	1	-2.793
368	0.375	1	-2.826
369	0.313	1	-2.854
370	0.25	1	-2.877
613	0.5	1	-2.755
614	0.688	1	-2.616
615	0.625	1	-2.666
616	0.563	1	-2.713
665	0	1	-2.953
669	0	1.188	-3.525
670	0	1.125	-3.331

671	0	1.063	-3.141
685	0.75	1	-2.561
686	0.938	1	-2.374
687	0.875	1	-2.439
688	0.813	1	-2.502
735	1	1	-2.305
736	1.188	1	-2.084
737	1.125	1	-2.16
738	1.063	1	-2.234
793	0	1.25	-3.721
794	0	1.438	-4.333
795	0	1.375	-4.126
796	0	1.313	-3.922
806	0	1.5	-4.542
807	0.188	1.5	-4.549
808	0.125	1.5	-4.569
809	0.063	1.5	-4.582
810	0	1.5	-4.585
901	1.25	1	-2.007
902	1.438	1	-1.768
903	1.375	1	-1.848
904	1.313	1	-1.928
983	1.5	1	-1.688
984	1.688	1	-1.449
985	1.625	1	-1.528
986	1.563	1	-1.608
1173	0.438	1.5	-4.384
1174	0.375	1.5	-4.438
1175	0.313	1.5	-4.483
1176	0.25	1.5	-4.52
1187	1.75	1	-1.372
1188	1.938	1	-1.148
1189	1.875	1	-1.221
1190	1.813	1	-1.295
1231	2	1	-1.078
1234	2.063	1	-1.01
1341	0.5	1.5	-4.324
1342	0.688	1.5	-4.095
1343	0.625	1.5	-4.179
1344	0.563	1.5	-4.255
1393	0	1.5	-4.624

1397	0	1.688	-5.254
1398	0	1.625	-5.041
1399	0	1.563	-4.832
1559	0.75	1.5	-4.005
1560	0.938	1.5	-3.695
1561	0.875	1.5	-3.804
1562	0.813	1.5	-3.907
1609	1	1.5	-3.58
1610	1.188	1.5	-3.208
1611	1.125	1.5	-3.336
1612	1.063	1.5	-3.46
1635	0	1.75	-5.469
1636	0	1.938	-6.13
1637	0	1.875	-5.908
1638	0	1.813	-5.687
1648	0	2	-6.353
1649	0.188	2	-6.352
1650	0.125	2	-6.379
1651	0.063	2	-6.396
1652	0	2	-6.401
1759	1.25	1.5	-3.077
1760	1.438	1.5	-2.673
1761	1.375	1.5	-2.809
1762	1.313	1.5	-2.944
1773	0.438	2	-6.129
1774	0.375	2	-6.202
1775	0.313	2	-6.263
1776	0.25	2	-6.313
1823	1.5	1.5	-2.538
1824	1.688	1.5	-2.138
1825	1.625	1.5	-2.269
1826	1.563	1.5	-2.403
1937	0.5	2	-6.046
1938	0.688	2	-5.728
1939	0.625	2	-5.845
1940	0.563	2	-5.951
1983	1.75	1.5	-2.009
1984	1.938	1.5	-1.643
1985	1.875	1.5	-1.761
1986	1.813	1.5	-1.883
1997	0.75	2	-5.6

1998	0.938	2	-5.151
1999	0.875	2	-5.311
2000	0.813	2	-5.46
2047	2	1.5	-1.529
2048	2.188	1.5	-1.216
2049	2.125	1.5	-1.316
2050	2.063	1.5	-1.42
2219	1	2	-4.981
2220	1.188	2	-4.42
2221	1.125	2	-4.615
2222	1.063	2	-4.802
2223	0	2	-6.445
2227	0	2.188	-7.11
2228	0	2.125	-6.888
2229	0	2.063	-6.667
2275	2.25	1.5	-1.122
2278	2.313	1.5	-1.033
2601	1.25	2	-4.219
2602	1.438	2	-3.594
2603	1.375	2	-3.805
2604	1.313	2	-4.014
2615	0.438	2.5	-7.942
2616	0.375	2.5	-8.024
2617	0.313	2.5	-8.092
2618	0.25	2.5	-8.147
2665	1.5	2	-3.384
2666	1.688	2	-2.77
2667	1.625	2	-2.97
2668	1.563	2	-3.175
2715	0.5	2.5	-7.848
2716	0.688	2.5	-7.48
2717	0.625	2.5	-7.617
2718	0.563	2.5	-7.739
2815	0	2.5	-8.293
2819	0	2.875	-9.432
2820	0	2.75	-9.084
2821	0	2.625	-8.702
2835	1.75	2	-2.575
2836	1.938	2	-2.038
2837	1.875	2	-2.209
2838	1.813	2	-2.388

2849	0.75	2.5	-7.327
2850	0.938	2.5	-6.773
2851	0.875	2.5	-6.975
2852	0.813	2.5	-7.159
2899	2	2	-1.876
2900	2.188	2	-1.444
2901	2.125	2	-1.579
2902	2.063	2	-1.723
2949	1	2.5	-6.554
2950	1.188	2.5	-5.786
2951	1.125	2.5	-6.061
2952	1.063	2.5	-6.317
3011	0	3	-9.741
3015	0.375	3	-9.527
3016	0.25	3	-9.647
3017	0.125	3	-9.718
3105	2.25	2	-1.317
3107	2.375	2	-1.089
3108	2.313	2	-1.199
3119	1.25	2.5	-5.494
3120	1.438	2.5	-4.538
3121	1.375	2.5	-4.866
3122	1.313	2.5	-5.186
3285	1.5	2.5	-4.208
3286	1.688	2.5	-3.265
3287	1.625	2.5	-3.566
3288	1.563	2.5	-3.882
3323	0.5	3	-9.357
3327	0.875	3	-8.503
3328	0.75	3	-8.851
3329	0.625	3	-9.133
3379	1.75	2.5	-2.981
3380	1.938	2.5	-2.248
3381	1.875	2.5	-2.473
3382	1.813	2.5	-2.717
3409	4	2	-8.751
3413	4	2	-8.945
3414	3.938	2	-10.066
3569	2	2.5	-2.042
3570	2.188	2.5	-1.521
3571	2.125	2.5	-1.68

3572	2.063	2.5	-1.853
3573	1	3	-8.079
3577	1.375	3	-6.163
3578	1.25	3	-6.951
3579	1.125	3	-7.567
3627	2.25	2.5	-1.376
3628	2.438	2.5	-1.007
3629	2.375	2.5	-1.119
3630	2.313	2.5	-1.242
3680	1.5	3	-4.946
3684	1.875	3	-2.495
3685	1.75	3	-3.039
3686	1.625	3	-3.772
3843	2	3	-2.049
3847	2.375	3	-1.134
3848	2.25	3	-1.385
3849	2.125	3	-1.686
3870	4	2	-9.123
3871	4	2.063	-3.541
3872	4	2.125	1.509
3873	4	2.188	5.957

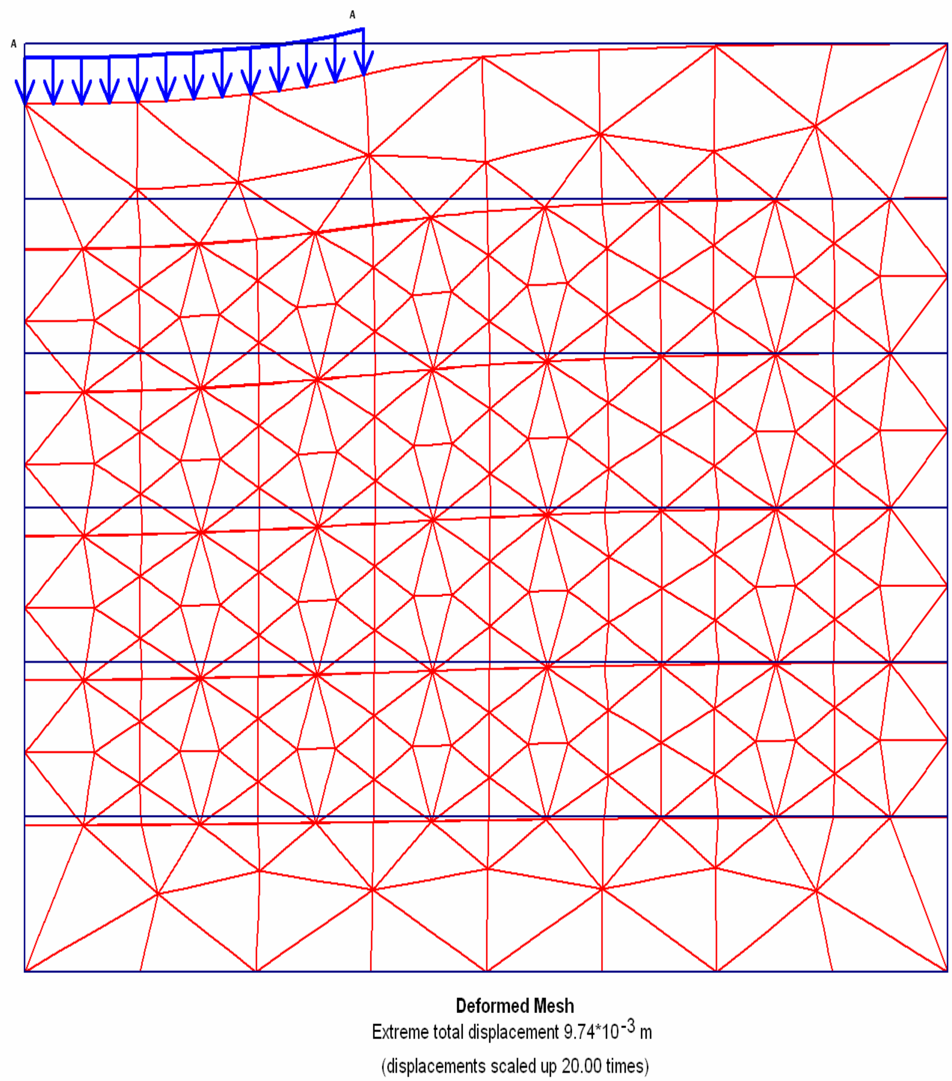


Fig.7.63 Problem geometry with mesh generation

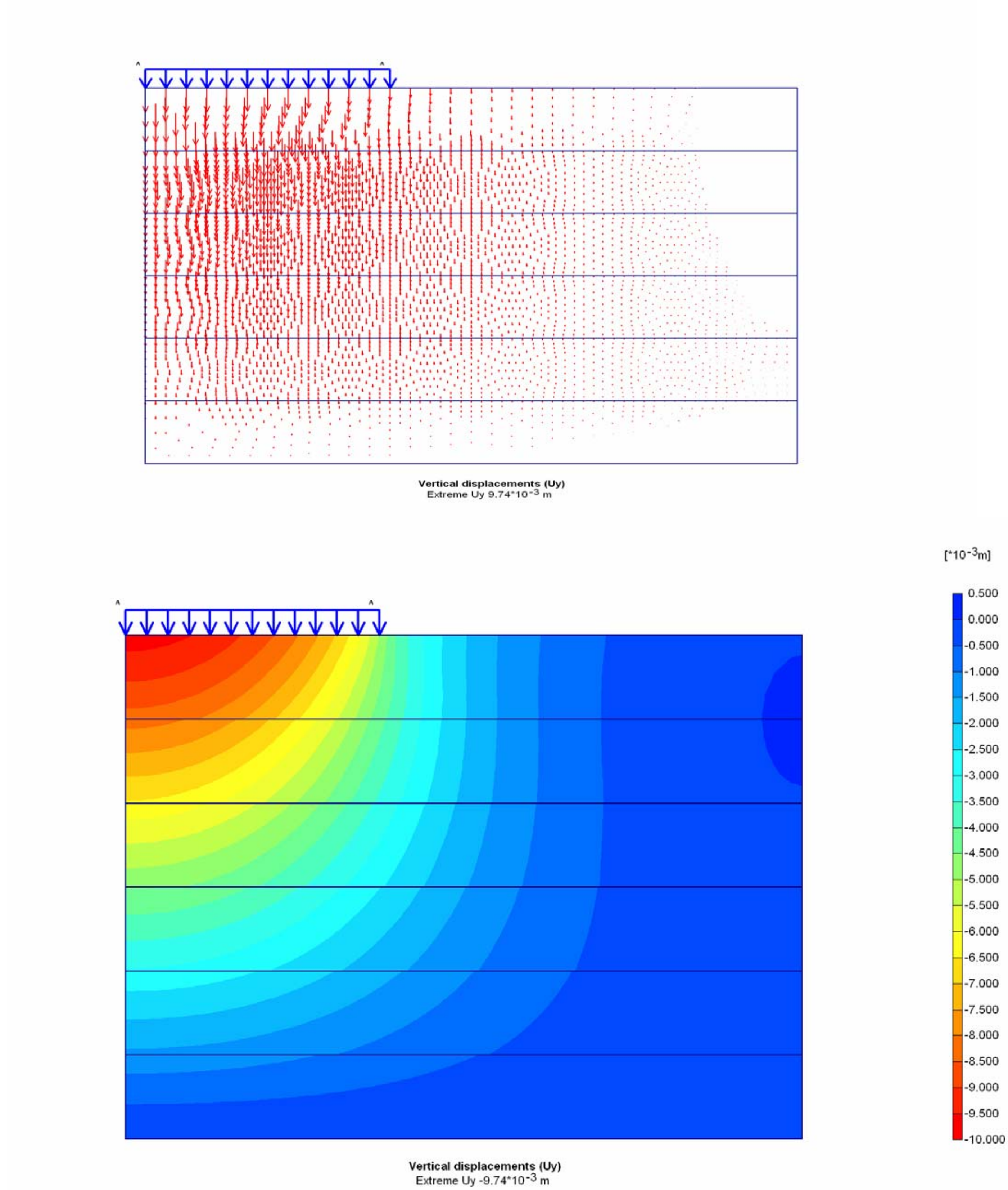
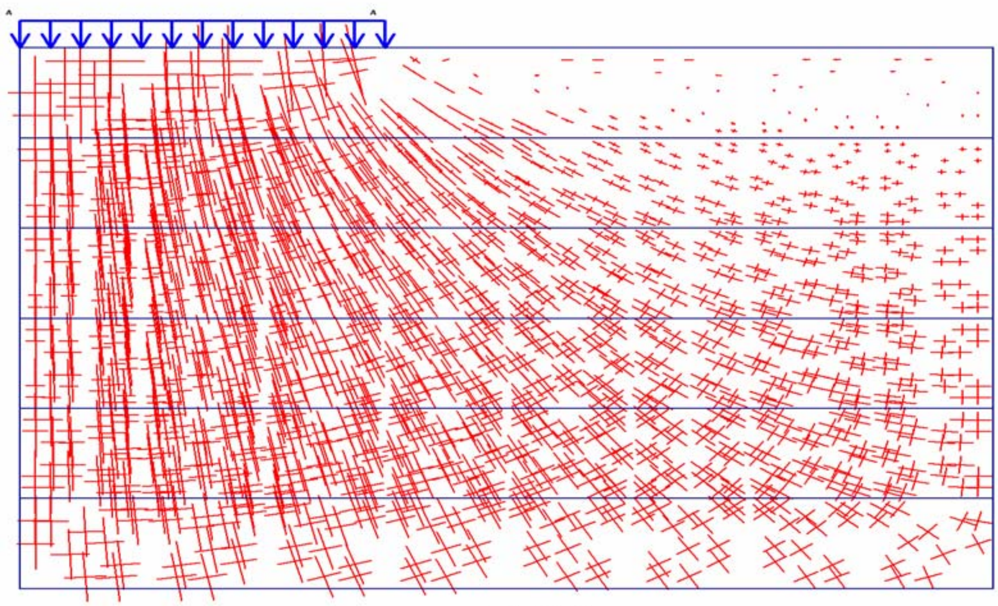


Fig.7.64 Vertical settlement in soil type B for circular footing (r=1.5m).



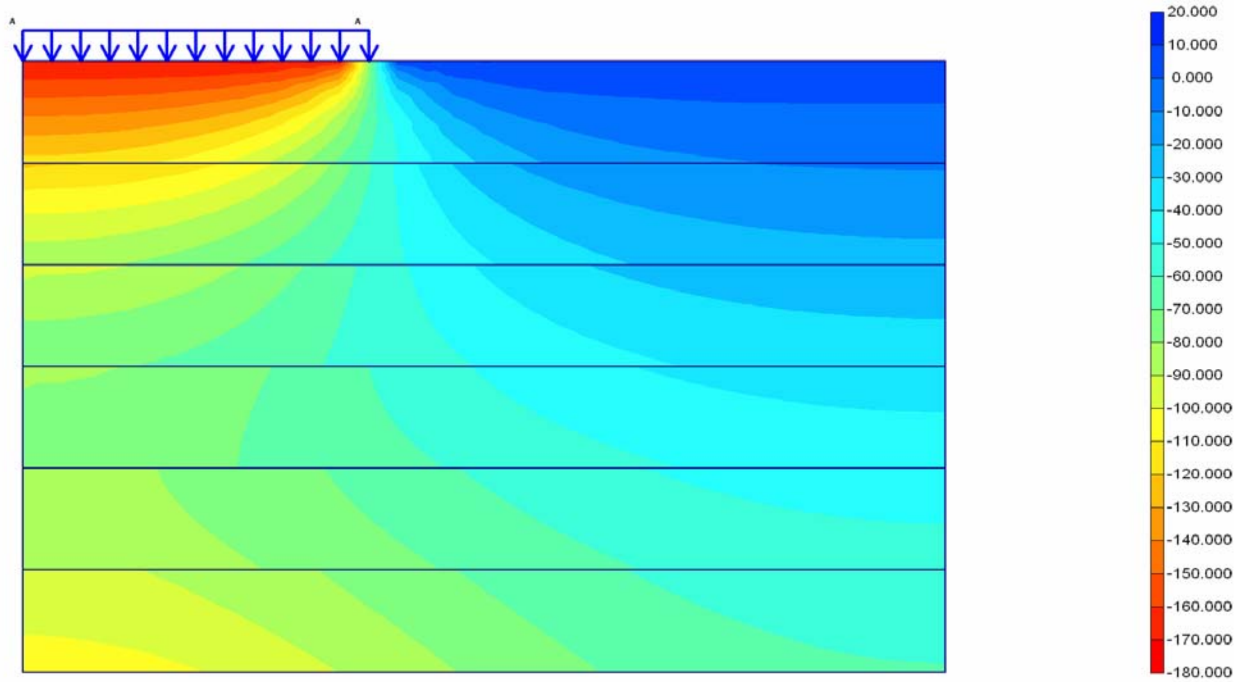


Fig.7.65 Mean stresses in soil type B for circular footing (r=1.5m).

CHAPTER-8

CONCLUSIONS AND SCOPE OF FURTHER STUDY

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CONCLUSIONS AND SCOPE OF FURTHER STUDY

8.1 Conclusions

Study has been carried out on the circular footing foundation system using conventional method and Finite Element Method (PLAXIS). The influence of the soil having constant value of elastic modulus, having variation of elastic modulus with vertical stress in the soil, diameter of the foundation and composition of the soil, on the settlement response of the circular foundation system has been examined. The conclusion arrived at based on the present study is listed below.

1. Soil elastic properties change with depth below footing. Elastic modulus decrease non-linearly with depth.

2. Variation in the value of the settlement by considering the soil has variation in elastic modulus with vertical stress to a constant value of elastic modulus is shown in table 8.1.

3. With increase in the size of the circular footing, increase in the settlement occur in the foundation and decease in the load carrying capacity of the foundation irrespective of the properties of the soil (soil type A or B) and nature of the foundation (elastic or rigid) in foundation as shown in table 8.1.

4. With Increase in the silt content in sand; decrease the settlement in soil for the same elastic properties under study for a given diameter of footing as given in table 8.2.

5. PLAXIS gives the comparable result on soil type A for different diameter of foundation; but gives quite different result for soil type B under study.

6. It shows that the Finite Element Method Software analyze the foundation by considering the soil as having constant value of elastic modulus, and not consider the variation of elastic modulus of soil.

Table 8.1 Settlement for soil A and soil B

PARAMETERS	SETTLEMENTS (mm) at a/r=0,z/r=0		PERCENTAGE CHANGE IN SETTLEMENT
Size of foundation (Radius)	Soil type A	Soil type B	
0.5m	3.624	4.985	37.55%
1.0m	9.421	7.569	24.44%
1.50m	11.455	8.242	38.98%

Table 8.2 Settlement for silty sand and soil A

PARAMETERS	SETTLEMENTS (mm) at a/r=z/r=0 & r=0.5m		PERCENTAGE CHANGE IN SETTLEMENT
Silt content (%)	Silty sand	Soil type A	
0	7.451	3.624	105.6%
5	5.884	3.624	62.36%
10	5.425	3.624	49.69%
15	4.856	3.624	33.99%

8.2 Scope of further study

The work can be extended to study the effect of the moisture content, variation in the water table depth, analyze by taking plastic behaviour of soil under study, for better understanding on the performance of foundation. As made clear, detail numerical understanding can only come from the three-dimensional modeling with properly developed model for soil. It is obvious to say that the Numerical modeling is not enough to study the behaviour of foundation soil and soil foundation- interaction problems, in soil but it is essential to physically study the behaviour through experiment.

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