ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

EVALUATING BEHAVIOR OF WAFFLE MAT FOUNDATION ON EXPANSIVE SOIL

A Thesis Submitted to Addis Ababa Institute of Technology in Partial Fulfillment of the Requirements for the Degree of Master of Science in Geotechnical Engineering

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ABSTRACT

In Ethiopia, expansive soils are found in wide areas. Those soils cause significant damage to structures built on them. Particularly lightly loaded building with different foundation types such as isolated and mat foundations. Are more prone to damage there is a general increase in building construction with increasing population at a very faster rate and large coverage of expansive soil. Therefore there is need to understand the characteristics of these soil type and appropriate foundation types.

The focus of this thesis is to evaluate the factors for moisture distribution with depth of soil, edge moisture distance and amount of soil heave by using proposed formula by Remon I. Abdelmalak (2007), the formula depends on pioneer work of Mitchell (1979). For the determination of deflection, moment and shear of waffle mat foundation assumed as interconnected T-beam. For soil heave the beam to assumed as simply supported and design as beam on elastic foundation using formula by M. Hetenyi (1946) to determine subgrade reaction of the soil. When soil shrinks the beam has no contact with soil and assumed as cantilever beam by using beam formula.

Diffusivity coefficient has great sensitivity to moisture distribution with depth, edge moisture distance and soil heave. It has more than 100% increasing effect for moisture distribution with depth, edge moisture distance and soil heave when the soil goes from uncracked condition to crack. Change of surface edge suction has secondary sensitivity. Drying period has little effect on the swelling characteristics of expansive soil. Differential soil heave and edge moisture distance increase when the slab dimension increase.

Beam depth has great sensitivity to deflection of the T-beam for dry soils. The deflection of the beam reduce by 75.6% when the beam depth increases two fold. Flange thickness has secondary sensitivity to deflection and flange width has small effect. For soil heave beam width has great sensitivity to deflection. Increases contact area and swelling pressure when the beam width increases. But other parameters have almost constant sensitivity for deflection.

Waffle mat foundation has reduced construction cost and time comparing with isolated footing. It has also high stiffness. Waffle mat foundation is a better foundation alternative for light loaded structure on expansive soil.

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LIST OF SYMBOLS AND ABBREVIATION

CEC: Cation Exchange Capacity

C_{mi}: Matrix suction index for layer i

C_{ti}: Effective stress index for layer i

C_W: Climatic rating index

D: The plate flexural rigidity

E: Modulus of elasticity of the concrete

EI: Beam flexural stiffness

ESP: Exchangeable Sodium Percentage (ESP)

e_o: The initial void ratio of the soil specimen at the seating pressure

e_{se}: Void ratio after stabilized swell at seating pressure

f: Ratio of the vertical strain to the volumetric strain

F: Reduction factor for surcharge pressure

G: Effective soil shear modulus

H: Thickness of expansive layer

H_g: Gravitational potential

H_m: Matrix potential

H_o: Overburden potential

H_p: Pneumatic (Gas) potential

H_s: Osmotic (solute) potential

H_T: Total potential

I: Moment of inertia of the slab cross-section

K:Effective subgrade modulus

L: Slab length

L' : Slab width

LL : Liquid Limit

m: The mound exponent

PE= Potential expansiveness

PI : Plasticity Index

PL: Plastic limit in percent

s : Slope of the suction water content curve

S_F : Potential vertical rise based on the free swell test

S_L :Potential vertical rise based on the loaded swell test

Sp : Percent swell,%

U_a : Pore air pressure

U_w : Pore water pressure

ω : Weather periodic time

wo : Initial water content

$\Delta H(x)$: The surface soil movement under the impervious cover

σ: Total stress

γ_h : Suction compression index

α_{field} : Field diffusion coefficient

ΔU_{edge} : Edge suction change

1. INTRODUCTION

1.1 Background

Expansive soils are clay soils with high plasticity. These soils are known for their peculiar nature of expanding or shrinking when exposed to moisture change. In dry state, the soils exhibit a high bearing capacity which is gradually lost with increase in moisture content. Expansive soils are found in large areas of Ethiopia. It covers nearly 40% surface area of the country Uba.B (2017).

These soils caused significant damage to structures that are founded on them, particularly the lightly loaded buildings which are commonly founded on various foundation types such as isolated and mat foundations. Although there is not an organized economic survey, it is assumed that most of the economic loss due to failures associated with civil engineering construction are attributed to expansive soils. Hence a vital need arises for foundation alternatives that can interact with this type of soil, and combat safely the soil swelling problems in Ethiopia. The present study propose the waffle mat foundation as one of foundation alternatives with the modest expenditure that may suit conditions in Ethiopia.

1.2 Objectives of this Study

1.2.1 General Objective

The main objective of this thesis is to evaluate waffle mat foundation on expansive soil and to provide reliable approach to make a safe design for practical effect of expansive soil on waffle mat foundation.

1.2.2 Specific Objective

This study is concerned with:-

- Reviewing, assessing and evaluating the available design methods for the slab-on – ground on expansive soils that have been used worldwide.
- Parametric evaluation for the soil structural parameters that are involved in the soil-structure interaction problem.
- Evaluating the waffle mat foundation as technically and financial viable solution for foundation resting on expansive soils in Ethiopia.

1.3 Scope of the study

The study is limited to the analysis of waffle mat foundation on expansive soil subjected to selected common and practical loadings. This work does not include analysis of a plate effect of foundation. The foundation is also assumed as interconnected T-beam on homogeneous expansive soil.

1.4 Methodology

This thesis has been divided into three parts. The first part evaluating the existent design methods for slab – on– grade comprises a pertinent literature review.

The second part of the study contains parametric evaluation for shrinkage and swelling characteristic of expansive soil by using proposed formula Remon I. Abdelmalak (2007), the formula depends on pioneer work of Mitchell (1979) and parametric evaluation for deflection of T-beam. Analysis of the T-beam assumed as simply supported beam when the soil is swell by using beam on elastic foundation formula M.Hetenyi (1946), and cantilever beam when the soil is dry.

The last part deals with how to design waffle mat foundation on expansive soil and cost comparison with isolated footing.

1.5 Organization

This document is organized into five Chapters. The first Chapter deals with the background of the study, objective, scope, and methodology employed. The second Chapter briefly reviews the major literature works related to foundation on expansive soil and characteristics of expansive soil. The third Chapter deals with evaluating of existent design methods for slab on ground. In the fourth Chapter, study with parametric evaluation for swell and shrink characteristics of expansive soil, structural parameters for deflection of T-beam, how to design waffle mat foundation on expansive soil and cost comparison with isolated footing. Finally, the fifth Chapter presents the conclusions drawn and the recommendations for further study.

2. LITERATURE REVIEW OF CHARACTERISTICS FOR EXPANSIVE SOIL

2.1 Introduction

This chapter reflects the literature review on expansive soils from different aspects, origin, structural and mineralogical, also the methods that are used globally for identification and classification of expansive soils together with evaluation of swelling potential, the approaches developed for predicting expansive soil heave and heave patterns.

Most soil classification systems arbitrary define clay particles as having an effective diameter of two microns (0.002mm) or less. Particle size alone doesn't determine clay mineral. Probably the most important grain property of fine-grained soils is the mineralogical composition. For small size particles, the electrical forces acting on the surface of the particles are much greater than the gravitational force. These particles are in a colloidal state. The colloidal particles consist primarily of clay minerals that were derived from parent rock by weathering (Chen, 1975).

2.2 Expansive Soil

2.2.1 Overview

Expansive soil is susceptible to volume changes in response to variations in moisture content. Expansive soils swell on wetting and shrink on drying by significant amounts and are often termed "reactive soil" (Walsh and Cameron, 1997).

Expansive soils are those soils which have the capacity to undergo considerable volumetric changes when subjected to variance in water content, they swell by increasing their moisture content and shrink when water removed from the soil. The degree of shrink/swell capacity is related or relevant to clay mineralogy (active minerals such as montmorillonite) (Krohn and Slosson, 1980).

Thus from the above one can easily apply the term “Expansive Soils” to those soils which predominantly contain clays, and have ability to expand or to shrink according to the change in soil water content.

The clay consists of fine grained material with particles smaller than 0.002 mm (ASTM-D422, 1998). As such, clay content of the expansive soils governs the reactivity (Gray and Allbrook, 2002). Clay is a general term including many combinations of one or more minerals with traces of metal oxides and organic matter (Guggenheim and Martin, 1995). Geologic clay deposits are mostly composed of sheet silicate minerals with water trapped in between the mineral structure (Nelson and Miller, 1992). These smaller particles combined with the layered crystalline composition produce properties of plasticity during wet conditions and significant strength during dry conditions (Nelson and Miller, 1992).

2.2.2 Soil Property

Type and Amount of Clay mineral

The type and amount of clay mineral are the factors which determine if the clay mass will shrink or expand when soil moisture conditions change. The three most important groups of clay minerals from the geotechnical engineering viewpoint are kaolinite, illite and montmorillonite. The mineralogical composition of the clay minerals determine the amount of volume change. Some clay minerals expand due to their swelling structure. Lattice structures of some minerals such as montmorillonite have a great ability to expand. Therefore, clayey soils having a considerable amount of expanding clay minerals in their composition should be considered as swelling soils. Essential clay properties that control the degree to which the clay minerals swell are (Barden, 1973):-

- Percent montmorillonite.
- Cation Exchange Capacity (CEC).
- Exchangeable Sodium Percentage (ESP).

Origin and Mineralogy of Expansive Soil

Chen (1975) reported that two origins from which montmorillonite is formed:-

(I) From the products of weathering and erosion of the rocks transferred with time and sedimented on plain areas, or

(II) From the products of volcano eruptions i.e. volcanic ashes that accumulated with time. He clarified that montmorillonite is basically formed with time from minerals that consist of Ferro-magnesium minerals, calcic feldspars and volcanic glass. He concluded that in semi-arid regions lack of leaching has assisted the formation of montmorillonite.

Weathering and climate

The weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering, which include hydrolysis, hydration, oxidation, carbonation and solution, of parent rock mineral which generally consists of ferromagnesium mineral, calcic feldspars, volcanic glass, volcanic rocks and volcanic ash. The formation is aided in alkaline environment, presence of magnesium ion and lack of leaching. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceed precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rainwater.

Clay mineralogy

The clay minerals are classified as follows (Snethen et al. 1975):

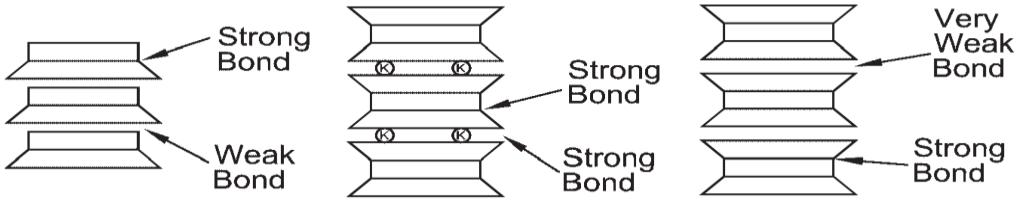
- Two-layer clays which consists of one tetrahedral layer bounded to one aluminum octahedral layer.

  Kaolinite is the most common mineral under this category.
- Three-layer clays which consists of one octahedral layer sandwiched between two tetrahedral layers. Illite, montmorillonite and vermiculite are the common mineral under this catagory.
- Mixed-layer clays which consists of interstratifications of the two and three layer clay minerals previously described. The mixing may be regular or random. Common mineral under these classes are chlorite, montmorillonite-chlorite.

The clay mineral Kaolinite exhibits very minor interlayer swelling. This is explained by the virtual absence of ionic substitution in either the tetra or octahedral layers which results in more or less complete electrical neutrality and the absence of compensating cations. Also, the individual two layer structures are more tightly bonded together by the opposing electrical charges on the adjacent octa and tetrahedral layers.

Illite a three-layer clay mineral, it exhibits very minor interlayer swelling. This result from the presence of non-hydrated K+ ions in the interlayer positions within the hexagonal openings of the tetrahedral layer. The K+ satisfies charge deficiencies residing mainly on the tetrahedral layer and is thus tightly bonded.

The clay mineral responsible for the most damage of expansive soil is montmorillonite. Montmoridonite also a three-layer clay mineral. The layers are held together by very weak oxygen-to-oxygen and cation-to-oxygen linkages. Exchangeable cations and associated water molecules are attracted between layers which causes expansion of the crystal lattice. The movement of exchangeable cations and associated water molecules into the interlayer spaces of the montmorillonite crystal causes a very large internal surface.



(a) Kaolinite (b) Illite and (c) Montmorillonite

Fig. 2.1 Schematic diagrams of the structure

Table 2.1 Properties of Three Major Types of Silicate Clay.
(After Brady, 1984)

Property	Kaolinite	Illite	Montmorillonite
Size (μm)	0.1-5.0	0.1-2.0	0.01-1.0
Shape	Hexagonal Crystals	Irregular Flakes	Irregular Flakes
Specific Surface(m^2/g)	5-20	100-120	700-800
External surface	Low	Medium	High
Internal surface	None	Low	Very High
Cohesion, Plasticity	Low	Medium	High
Swelling capacity	Low	Medium	High
Cation Exchange Capacity(cmol/kg)	3-10	15-40	80-120

Adsorbed cations and cation hydration

The negative charges on the face of the clay particle are balanced by positively charged cations. The type of cations in the environmental will influence the nature of the clay soil. The cations in the soil can exist in different stage. For a completely dry soil, the cations would be at a very low stage of hydration. A water become available to the soil, it will bond with the sodium cations by process of hydration.

Frank and Wen (1957) present a well-accepted concept of hydration. They depicted a simple model of a hydrated ion as shown in Fig. 2.2. Here, region A is termed the region of immobilization. In that region, the water molecules are strongly held in the field of the ion and are immobile. In region B, the water molecules have less structure but are held to the ion. Region C contains water with normal structure but the water molecules are polarized by the weak ionic field. The water held in regions A, B, and C is termed water of hydration. For purposes of considering the role of ion hydration in soil expansion, the water in region A can be assumed to be “fixed” and forming a permanent part of the ion. Region C can be considered as part of the water that is adsorbed to the clay particle and that can be removed easily by air drying. Region B is considered to be that part of the water of hydration that is removed by heat or desiccation of the soil.

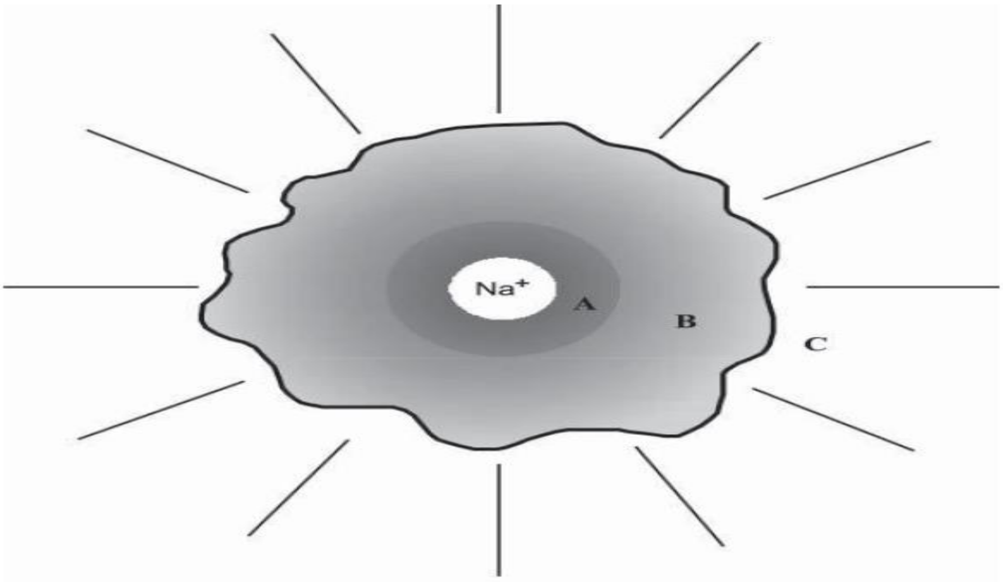


Fig. 2.2 Ion water interaction (modified from Frank and Wen 1957).

A conceptual model to depict the role of cation hydration on soil expansion is shown in Fig. 2.3. In figure (A), two parallel clay particles are separated by cations that have a very low level of hydration. These would be cations that have water in region A and perhaps some of region B. The energy of hydration is quite large such that if water is added to the soil, it is drawn into region B. Figure (B) shows the cations as having been hydrated to the stage where region B is completed. As more water is added, it goes into region C, as shown in Figure (C).

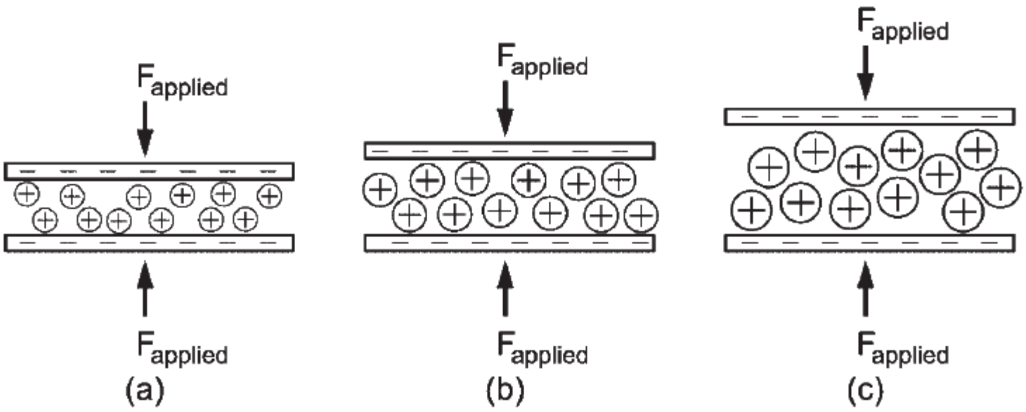


Fig. 2.3 Role of cation hydration on soil expansion:
(a) low hydration; (b) partial hydration and (c) full hydration.

Lambe and Whitman (1969) give the radii of unhydrated and hydrated sodium ions as 0.98Å and 7.8Å, respectively. Thus, in going from an unhydrated state to a fully hydrated state the ion grows in size by more than 700%. If the hydration of the cations were to cause even a 10% increase in the diameter of cations, this would represent a 10% change in spacing between particles, which would correlate to a significant amount of soil expansion.

Migration of moisture

When the moisture content of expansive soil changes, volume change occurs both in the vertical and horizontal directions. The amount of swelling pressure which causes volume change is dependent on the initial moisture content of the soil. The amount of volume change decreases with increasing initial moisture content and vice versa.

The soil-water system will allow moisture migration until a new state of equilibrium is reached. Total suction of the soil-water system creates an energy gradient that induces moisture migration.

Total suction of the soil is the amount of work required for migration of moisture from a soil at equilibrium conditions. It is comprised of matrix suction, osmotic suction, gravitational suction, pneumatic suction and overburden suction.

$$H_T = H_m + H_s + H_p + H_g + H_o \qquad (2.1)$$

H_T = Total potential
H_m = Matrix potential
H_s = Osmotic (solute) potential
H_p = Pneumatic (Gas) potential
H_g = Gravitational potential
H_o = Overburden potential

Matrix suction is capillary flow in soils. Capillary flow can occur upward, downward or lateral. Osmotic suction occurs from the differences in concentrations of soluble salts in the pore fluid. Therefore, osmotic suction causes moisture migration in the direction of increasing concentration. Gravitational and overburden potential causes downward moisture migration. Pneumatic potential can cause moisture migration in all directions.

The change of any component of the total soil suction can cause moisture migration through the soil. Matrix and osmotic suctions are most important for moisture migration in comparison to other suction components Therefore, Gravitational, pneumatic and overburden potentials are considered to be negligible for most engineering applications. Total soil suction is a function of water content, surface tension and osmotic forces in the free pore water and adsorbed water in the soil. A potential for swelling exists in soil beneath a building foundation if the initial pore water pressure or soil suction is negative. Soil suction can be used to characterize the effect of moisture on volume change behavior of soil (Snethen, 1980).

$$H_T = H_m + H_s \qquad (2.2)$$

Osmotic suction occurs due to osmotic potential of the water which is dependent on the presence of dissolved salts. The effect of solutes is to reduce the free energy of water because the solute ions attract the water molecules. The osmotic suction also affects the movement of water vapor since water vapor pressure is reduced by the presence of solutes (Piconell and Lytton, 1984).

The matrix suction results from soil structure such as capillary tension and attractive forces attracting dipole soil water. Matrix suction can be described as pressure produced when the free water has the same chemical concentrations as the soil's pore fluid. The variation of matrix suction is influenced by the density and pore size distribution.

Matrix suction occurs between water surrounding clay particles and clay interlayer water which cause the volume change in expansive soils. The amount of water drawn into clay layers is dependent on the potential of the surrounding water. Attractive forces include hydrogen bonding, clay particle attraction, cation hydration, etc.

Matrix suction can be expressed as:

$$H_m = U_a - U_w \qquad (2.3)$$

H_m = Matrix suction U_a = pore air pressure U_w = pore water pressure

Another factor that affects moisture migration is change in temperature. Moisture migration occurs in the direction of decreasing temperature.

Donaldson (1965) identified three sources of causes for moisture migration beneath structures mainly:-

- The first is caused by the erection of the structure which produces changes in soil moisture upon long period until equilibrium state is achieved.
- The Second is the fluctuation which is mainly caused by the seasonal climatic changes and water table fluctuation, and
- The Third is due to broken man - made services such as leaking water pipes, broken sewers

Depth of active zone

The soil moisture active zone can be defined as the zone of major seasonal moisture variations. Climate, depth to the groundwater table, type and amount of clay minerals, soil profile, temperature, and vegetation are the factors affecting soil moisture in the active zone. If seasonal moisture changes are large, greater depths of active zone will occur. The determination of the depth of the soil moisture active zone is an important parameter for the design of foundations on expansive soil. If the depth of the active zone is deeper, a greater amount of soil will experience volume change. If the depth to the ground water table is close to the surface, the depth of active zone is also shallow. A long period of dry climate produces a deep active zone. On the other hand, a long period of wet climate usually reduces depth of active zone. Change in soil temperature also influence the depth of active zone. Moisture will move close to the surface during the colder seasons and will return to deeper depths during the warmer seasons. The depth of active zone depends on the type and physical properties of the soil (Snethen, 1980).

2.3 Identification and classification of expansive soil

2.2.3 Expansive soil in Ethiopia

Distribution of expansive soil is generally a result of geological history, sedimentation and local climatic conditions. Arid climatic conditions and severe weathering environment prevailing in north eastern part of Africa promote the widespread occurrence of expansive soils. In Ethiopia, covering nearly 40% surface area of the country, expansive soils are observed in area such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa - Debere Berehan, Addis Ababa - Gohatsion, Addis Ababa - Mojo. Also the cover the area like Mekelle, Bahirdar, Gambela, Arba Minch and the most Southern, South-west and south-east part of the capital Addis Ababa area in which the most major recent construction are being carried out (Uba.B, 2017). The distribution of expansive soil in Ethiopia show in Fig. 2.4.

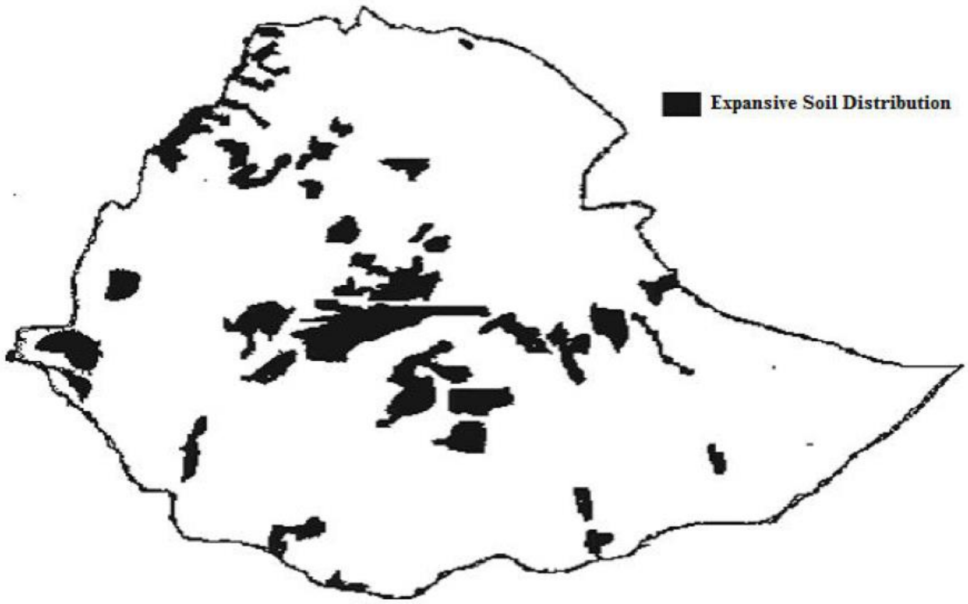


Fig. 2.4 Distribution of Expansive Soils in Ethiopia (Uba. B, 2017)

2.3.1 Swell pressure and swell potential

Swelling pressure is a very useful index of the trouble potential of an expansive soil. This pressure is the maximum force per unit area that needs to be applied over a swelling soil to prevent volume increase. The swell potential of a soil in comparison is the magnitude of heave of a soil for a given final water content and loading condition. Specially designed oedometer tests have been found quite useful to determine the magnitudes of these parameters for expansive soils (Amer Ali and Mattheus F.A, 2006).

Oedometer test procedure

The swell potential and swell pressure of a soil specimen can be determined by any of the three methods specified by ASTM Standards (ASTM D 4546–90 Standard Test Method for One Dimensional Swell or Settlement Potential of Cohesive Soils).

Method A

The seating pressure (at least 1 kPa) is applied to the clay specimen. After the initial deformations at the seating pressure are complete, the specimen is inundated with water in the oedometer cell and is allowed to swell vertically.

The time-swell curve typically consists of three regions. An initial swell region, primary swell region, and secondary swell region Fig.2.5. The minor initial swell is attributed to swelling of the macrostructure, while the major primary swell and minor secondary swell is attributed to microstructural swelling (Rao *et al.*, 2006). The specimen is stepwise loaded after primary swell is complete using a load increment ratio of unity. The loading process is continued till the swollen specimen regains its initial void ratio/height. The external pressure needed to regain the initial void ratio e_0, defines the swell pressure of the specimen (Amer Ali and Mattheus F.A, 2006).

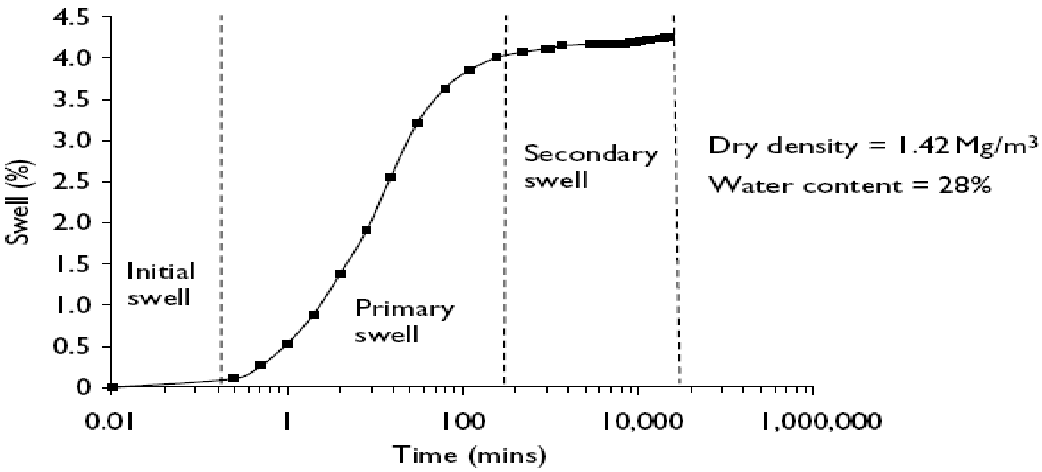


Fig. 2.5 Time-swell behavior of compacted black cotton soil

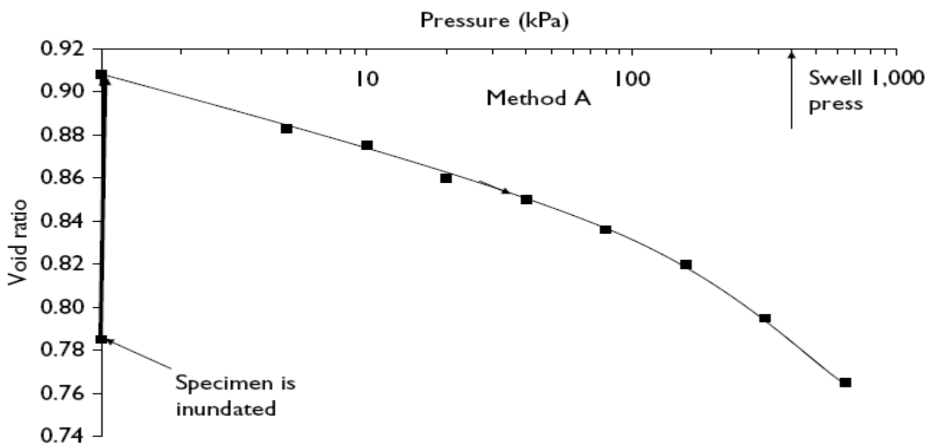


Fig. 2.6 Estimation of swell potential and swell pressure by Method A

The well potential at the seating pressure relative to the initial void ratio e_0 is given as:

$$Swell\ \text{potential}\ (\%) = \frac{e_{se} - e_o}{1 + e_o * 100} \qquad (2.4)$$

Where: e_{se} is void ratio after stabilized swell at seating pressure and e_o is the initial void ratio of the soil specimen at the seating pressure.

This test method measures:-

- The percent heave of the specimen at the seating pressure and
- The swell pressure of the specimen

Method B

A vertical pressure exceeding the seating pressure is applied to the specimen. The magnitude of vertical pressure (σ_{v0}) is usually equivalent to the in situ overburden pressure, or structural loading or both. After the axial deformations under the vertical pressure (σ_{v0}) are complete, the specimen is inundated with water and axial swelling deformations of the specimen are recorded until primary swell is complete. After completion of primary swell, the specimen is stepwise loaded till the pre-wetting void ratio (e_{v0}) corresponding to the vertical pressure σ_{v0} is attained. The pressure needed to regain the void ratio (e_{v0}) defines the swell pressure of the specimen.

This test method measures

- The percent heave for vertical pressure usually equivalent to the estimated in situ vertical overburden pressure and other vertical pressures up to the swell pressure and
- The swell pressure of the specimen

The swell potential at the vertical pressure σ_{v0}, relative to e_{v0} is given as:

$$Swell\ \text{potential}\ (\%) = \frac{e_{se} - e_{vo}}{1 + e_{vo} * 100} \qquad (2.5)$$

Where e_{se} the void ratio is after stabilized swell at the vertical pressure σ_{v0} and e_{vo} is the initial (pre-swollen) void ratio at the vertical pressure σ_{v0}.

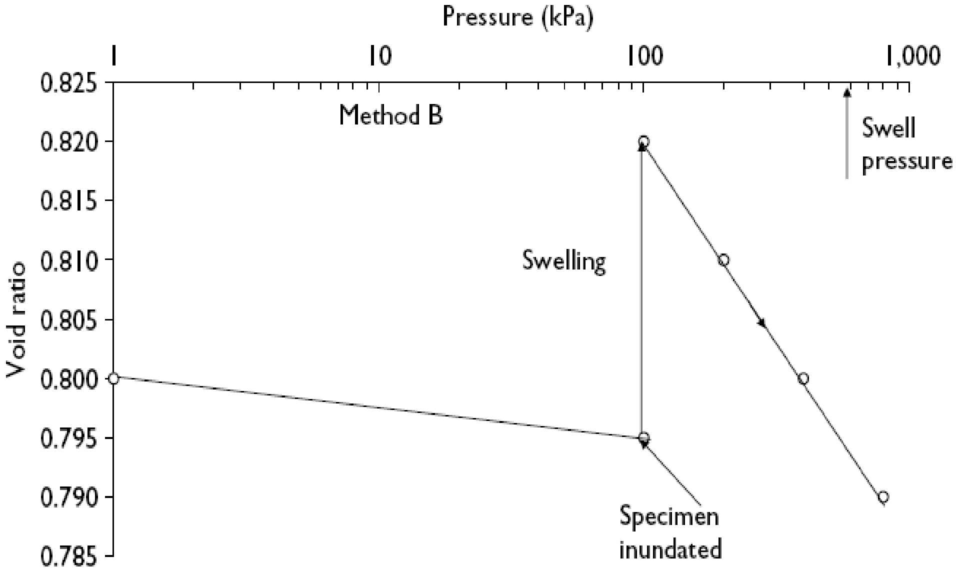


Fig. 2.7 Estimation of swell potential and swell pressure by method B

Method C

A vertical pressure (σ1) equivalent to the estimated vertical in situ pressure or swell pressure is applied to the specimen. After completion of axial deformations under the vertical pressure (σ1), the specimen is inundated with water. Increments of vertical stress are applied to the wetted specimen to prevent any swell. Variations from the dial gage readings at the time the specimen is inundated at stress σ1 shall be preferably kept within 0.005 mm and not more than 0.01 mm. The vertical pressure at which the wetted specimen shows no further tendency to swell defines the swell pressure of the specimen. The specimen is stepwise loaded following no further tendency to swell. The applied load increments should be sufficient to define the maximum curvature on the consolidation curve and to determine the slope of the virgin compression curve.

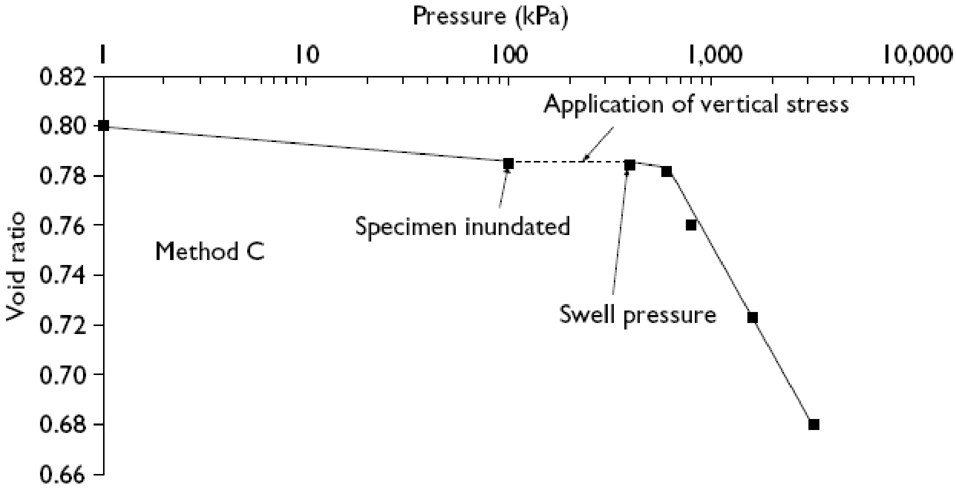


Fig. 2.8 Estimation of swell pressure by Method C

Table 2.2 Classification of expansive soil

Degree of expansion	*Holtz and Gibbs' (1956) classification of percent swell*	*Seed et al's (1962) classification of percent swell*
Low	0–10	0–1.5
Medium	10–20	1.5–5
High	20–35	5–25
Very high	>35	>25

Table 2.3 Identification criteria for expansive clay

Colloid content (%)	*Plasticity index (%)*	*Shrinkage limit (%)*	*Degree of expansion*	*Probable expansion (% total volume change)*
<15	<18	<10	Low	<10
13–23	15–28	10–20	Medium	10–20
20–31	25–41	20–30	High	20–30
>28	>35	>30	Very high	>30

Source: After Holtz and Gibbs, 1956.

2.3.2 The Prediction of Heave for Expansive Soils

After proper soil investigations have been conducted on a specified expansive soil through the reviewed identification and classification procedures, and accurate qualitative and quantitative soil parameters were obtained, the next essential step is using them to predict the expected heave that may be exhibited from this soil.

Methods Used for Heave Prediction

Numerous investigators developed different techniques for predicting soil heave, conveniently they can be categorized into three methods, empirical methods, oedometer methods, and soil suction methods.

Empirical method

Researchers attempted to establish by regression some verified empirical equations and charts correlating swell, swell Pressure with soil index properties such as Liquid Limit, Plastic Limit and Natural Moisture Content, mainly to approximate soil heave. Taking into consideration all proposed relationships have some constrains or limitations in their application to soil outside the geographical area. They may suit the local soils where they were developed Van der Merwe (1964) proposed an empirical relationship between the degree of expansive (PE), the plasticity index (PI), percent clay (%) and surcharge pressure. This method doesn't consider variation in initial moisture conditions. He found that the total heave experienced from expansive soil layers can be directly computed from the following equation.

$$\Delta H = \sum_{D=1}^{D=N} F * PE \qquad (2.6)$$

Where: ΔH = Total heave

D= Depth of soil layer

F= Reduction factor for surcharge pressure, $F = 10^{-D/20}$

PE= Potential expansiveness

Nagaraj and Muthy (1987) proposed a semi-empirical method for soil heave prediction, they based their method of predicting the swelling from the re-bound line from the equivalent pre-consolidation pressure.

$$\Delta H = \sum_{i=1}^{n} \frac{0.0463e}{1+e_o} H_i \log\left(\frac{p_{si}}{p_{oi}}\right) \quad (2.7)$$

Where: ΔH = is the total heave. p_{si} = Swelling pressure for Layer (i)

H_i = is the thickness of layer (i) p_{oi} = Overburden pressure for layer (i)

e = Void ratio at Liquid Limit for Layer (i)

e_o = Initial void ratio for layer (i)

Fredlund and Rao (1987) developed an empirical equation for computing heave within soil active zone , they suggested to subdivide the active depth into m number of layers of equal thickness (i.e., hi=h)

$$\Delta h_i = \frac{C_s}{1+e_o} \frac{H}{m} \log\left(\frac{\gamma g H (2i-1)}{2m} \frac{1}{gH}\right) \quad (2.8)$$

Where: H = active depth

h = H/m = is the thickness of a soil layer

Δh_i = Heave in a layer i

C_s = Swelling index

e_o = Initial void ratio.

γ = Total density of the soil which is assumed to remain constant.

g = Gravitational acceleration

Heave Prediction by Oedometer Method

According to Nelson and Miller (1992), there are three Oedometer test meth-ods used commonly for predicting heave in the laboratory: the consolidation-swell test, the constant volume test and the double Oedometer test, but the former is widely used for its simplicity.

The constant volume test was recommended by Porter and Nelson (1980) and Fredlund (1983) as the appropriate test for predicting expansive soil heave. Initial and final void ratio, swelling index, corrected swelling pressure are those parameters obtained from this test and simply can be used in the following equation for computing heave.

$$H = \sum_{i=1}^{n} \frac{C_s}{\left(1+e_o\right)_i} \log\left(\frac{\sigma'_f}{\sigma'_{sc}}\right) \qquad (2.9)$$

Where: H = is total heave

C_s = Swelling Index

σ'_f = Final effective stress state

σ'_{sc} = corrected swelling pressure from constant volume test.

n = is the number of layers

.

Soil Suction Methods

As it was mentioned earlier; the quantitative characterization of expansive soil is extremely necessary to predict the amount of anticipated volume change. Suction method is one of these procedure used for estimating soil heave.

Nelson and Miller (1992) reported that; suction test depends upon soil response to suction changes through the initial suction condition which can be measured directly and the final one that must be assumed, taking in the consideration that the soil suction value is usually greater than zero above ground water table, thus they may or may not be saturated.

Total heave can be computed using the following equation which is based on suction parameters. This equation consists of two terms, the first one, heave due to matric suction changes the second, heave due to effective stress changes.

$$H = \sum_{I=1}^{n} \frac{Z_i}{(1+e_o)_i} \left(C_{mi} \Delta \log (Ua\text{-}U_w) + C_{ti} \Delta \log (\sigma - Ua) \right) \qquad (2.10)$$

Where: H = total heave

Z_i = thickness of layer i

$\Delta = (e_f - e_o)_i$= void ratio change for layer i

C_{mi} = matrix suction index for layer i

C_{ti} = effective stress index for layer i

σ = total stress

Ua = pore air pressure

Uw = pore water pressure

Factors Affecting Heave

Jennings (1969) listed essential factors upon which heave and heave rate depend:-

- Thickness of expansive soil.
- The depth to the water table.
- The nature and degree of desiccation as existing in the profile.
- The characteristics of the soil.
- The initial stress condition in the soil.
- The stresses applied by the structure σz vertically and σx and σy horizontally.
- The soil properties which determine its volume change.
- Rate of change of volume:-
    (a) In case of heaving depends on the rate of absorption.
    (b) In case of shrinking depends on soil permeability.
- The depth of seasonal moisture content.

2.4 Foundations on Expansive Soil

It is believed that once the foundation problem at a site has been recognized in its overall perspective, it is possible to propose and provide economical, practical solution which will well cope with the ultimate heave movements, thus reliable estimates of heave are extremely requisite for the selection of appropriate foundation to the soil swelling.

In this portion some of the foundation types universally are adopted to found structures in and on expansive soils such as pad foundation, strip, pile foundation, stiffened raft and waffle mat foundation will be reviewed briefly.

A comprehensive review will be given to the waffle mat foundation option aiming at reflecting its performance advantages and disadvantages through some comparisons and discussions.

Selection of Foundation

The proper selection for suitable and appropriate foundation type among foundation design alternatives can be achieved by dealing with adequate soil parameters in order to minimize distortion and damage to structure. Table 2.4 gives guidelines for selecting a foundation system based on heave parameters.

Nelson and Miller (1992) stated two fundamental different strategies for selecting foundations on expansive soils to minimize the differential movement.

- Isolate completely the superstructure from the expansive soil movement (drilled pier and beam) or
- Design a suitable foundation to be stiff enough to combat soil differential movement (stiffened raft foundation and continuous footings).

Foundation system can be grouped into two major classes shallow and deep foundation. Shallow foundation includes mats, continuous and spread footings, while deep foundation involves piled raft and piles.

Table 2.4 Guidelines for selected Foundation Systems on Expansive soils (U.S.A. Eng. Corp. Tech. Manual 1983)

Predicted Differential Movement, inches	Effective Plasticity Index, PI	Foundation System	Remarks		
½	<15	Shallow individual continuous wall strip ReReinforced and stiffened thin mat.	Lightly loaded buildings and residences Residences and lightly loaded structures; on-grade 4- to 5-in. reinforced concrete slab with stiffening. Beams; maximum free area between beams 400 ft^2; ½ percent reinforcing steel; 10-12in thick beams; external beams thickened or deepened, and extra steel stirrups added to tolerate high edge forces as needed; dimensions adjusted to resist loading. Beams positioned beneath corners to reduce slab distortion.		
			Type of Mat	**Beam Depth, in**	**Beam spacing, ft**
½ to 1	15 to 25		Light	16 to 20	20 to15
1 to 2	26 to 40		Medium	20 to 25	15 to 12
2 to 4	> 41		Heavy	25 to 30	15 to 12
No limit		Thick, reinforced mat	Large, heavy structures; mats usually 2ft or more in thickness.		
No Limit		Deep foundations pile or drilled shaft.	Foundations for any light or heavy structure; grade beams span between piles or shafts 6 to 12in above ground level; suspended floors or on-grade slabs isolated from grade beams and walls. Concrete drilled shafts may be underreamed or straight , reinforced, and cast in place with 3000=psi concrete of 6-in slump.		

Spread Pad Footing

This type of foundation is generally used to found a structure on low to moderate potential soils of swell percent ranges from 2.5 to 12.7 and swelling pressure from 144 to 240kN/m^2, and should be design to a dead load pressure as high as the specified swelling pressure (Chen 1975).

Katzir and David (1969) advised to locate the spread footing at a depth below the zone of seasonal variation of moisture content (Active zone) so as to avoid the influence of heave. They added that; drying of the soil during excavation should be prevented.

The use of non-expansive soil of low permeability in refilling pits is extremely necessary so as to reduce pressure effectiveness. (Katzir and David (1969), Chen (1973).

Grade beams should be elevated from the ground surface with a sufficient clear height greater than the maximum anticipated heave.

Strip Footing

It is sometimes termed long or modified continuous footing or shallow individual continuous wall strip. It is preferable used for lightly loaded structure (e.g. residences) supported by a foundation soil of low swelling potential. The location of the strip footing were usually found where the building walls are.

The strip footing should be as narrow as possible to concentrate the stress in order to combat the soil swelling pressure (Chen 1975).

Chen (1975), Holland and Richard (1984) indicated the major disadvantage of the strip footing is the lack of the three dimensional rigidity which obviously leads to instability particularly when constructed on moderate to high soil swell potential.

Pile Foundation

Generally a pile foundation is an element (commonly cylindrical shape) that transfers the superstructure load to the soil either through the bearing end point (pile tip) or by its surface frictional transformation.

There are several types of piles that basically depend on the materials from which the pile is made (concrete, steel, or timber) and on the methods of installation driven (precast, and prestressed) or bored piles (cast in-situ).

Chen (1975) defined the drilled pile in expansive soils as a type of foundation that is used to transfer the structural load from the upper unstable soil to the lower stable soil.

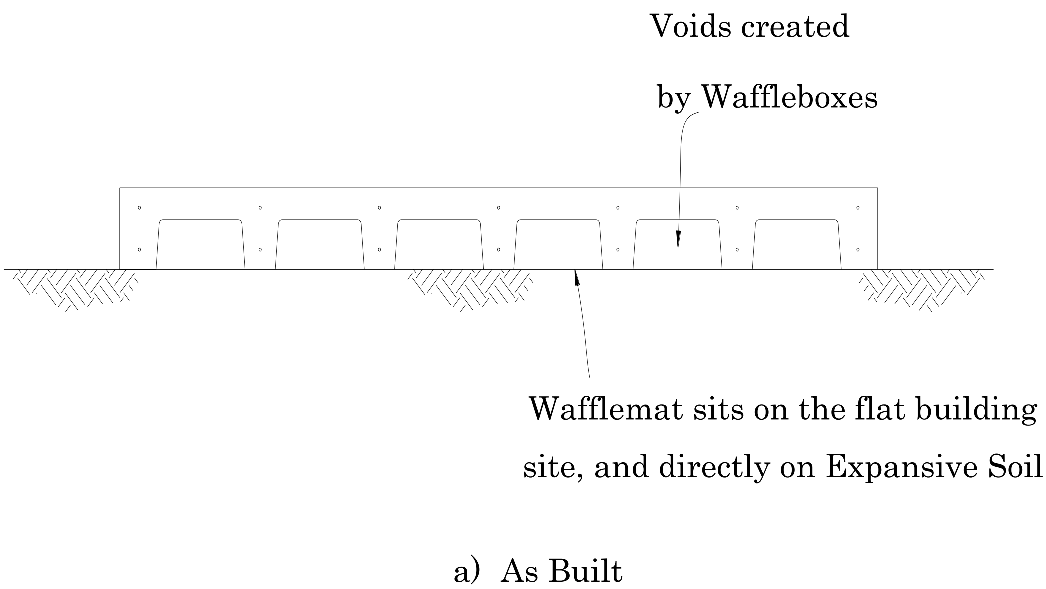
Piles are the most suitable foundations to combat the high soil swelling and the unexpected differential movement in order to transfer superstructure loads safety to deep layers, and to provide an anchor to resist sufficiently the uplift pressure upon the active zone area. (Chen 1975-1988).

Waffle mat foundation

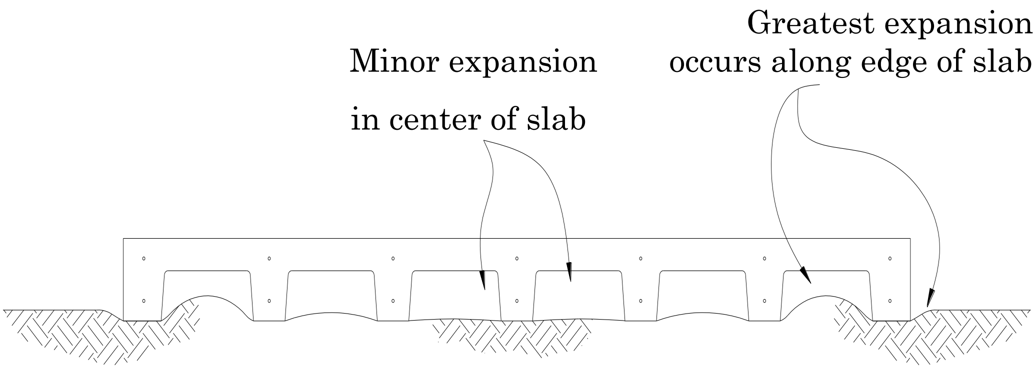
General Description

Waffle mat foundation a type of foundation that consists of a thin concrete slab with underlying stiffening cross beams in order to provide an additional stiffness for the slab. It has been successfully used since the early 1990's. A grid of narrow reinforced beam contact ground. The ribs created by the Waffle boxes, a contact area much less than the total slab area. Reduction of the contact area is significant in that there is less surfaces upon which swelling soils can exert force and the superimposed bearing stress is increased. The resulting higher bearing stress from the structure counteracts soil swelling pressures

The waffle mat foundation is based on the concept of a thick solid raft regularly hollowed at its base forming a grid of underlying cross beams topped with a constant thin slab. This huge reduction in concrete volume is mainly for economical purposes. It also creates a much more rigid matrix of beams that resist the bending forces exerted on the foundation by the upwardly expanding soil. Figure (2.9) will illustrate the concept of waffle mat foundation (Greg Carr and John Maier, 2011).



a) As Built



b) During Expansion of Soil

Fig. 2.9 View of the Waffle mat foundation (Greg Carr, 2011)

The Design Concept

The waffle mat foundation is considered as an alternative solution in designing rigid building capable of tolerating soil movement. Systematically the slab transmits the loading forces from the super-structure to the stiffening beams which resist moments and shear due to differential heave of the expansive soil.

The waffle mat foundation is designed to resist both positive and negative moments from the superstructure loads and from the pressure due to under slab soil swelling. Usually negative moment controls the design of the stiffened raft.

The Construction Techniques of waffle mat Foundation

Advisable construction procedures that may be followed in constructing waffle mat foundation as recommended by Post-Tension Institute (PTI) are:-

- Preparing the building pad- cleaning and leveled building surface. Presoaking non necessary with the system.
- Install underground utilities
- Place the formwork on the ground.
- Lay out waffle boxes and connects with plastic clips to supply waffle mat system.

- Install reinforcing steel, the waffle mat plastic clips supporting rebar in the correct position
- pouring the concrete

Photo 2.1 Lay out waffle boxes and connects with plastic clips

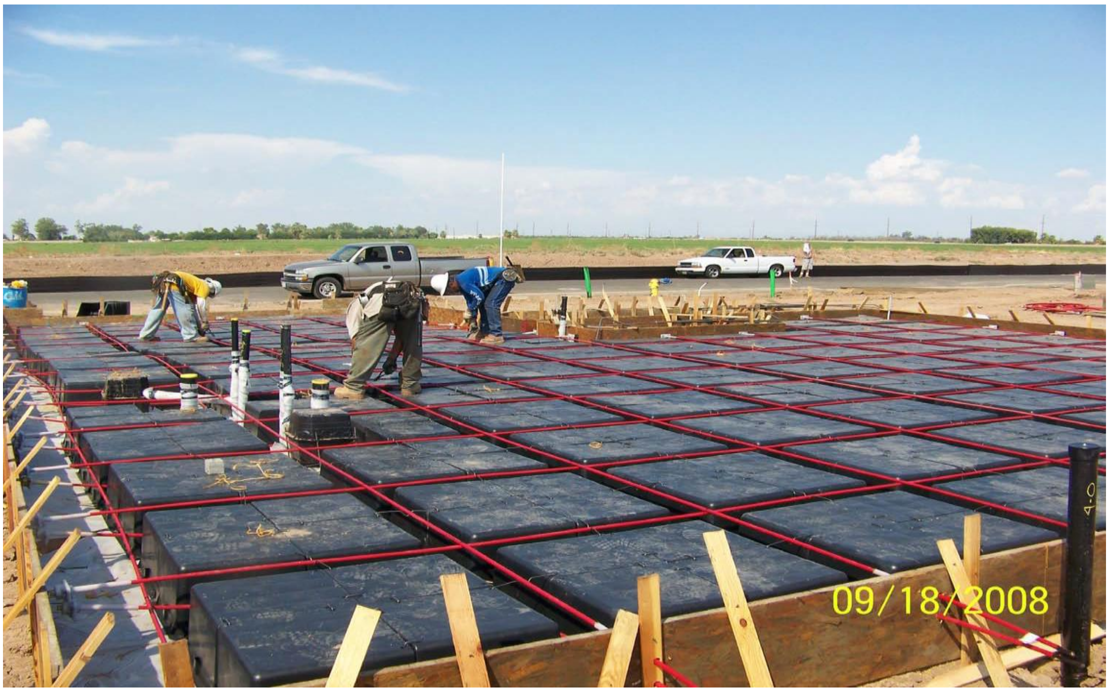


Photo 2.2 Install reinforcing steel

Photo 3.3 Pouring the concrete

Photo 2.4 after pouring concrete

Waffle box

Plastic Dome waffle box are stay-in-place forms to create unreinforced or reinforced concrete slabs on grade and other concrete structures. Concrete is poured over the modular dome forms to create floating or structural slabs on grade. It made from 100% recycled Polypropylene (PP) plastic; the forms provide the maximum performance and guarantees superior characteristics of stability and resistance in its structure to allow operations that are completed directly above the plastic. Waffle mat form(Waffle box) is designed to support the weight of given thickness of concrete slab plus any common work loading required for placing the concrete slab, during the entire curing period. The materials used are inert and non-toxic. Waffle box is unaffected by water, snow or ice so, in wet weather or when foundations are below the water table, work can continue without reducing its effectiveness (Greg Carr, 2011).

3 EVALUATION OF PRESENT SLAB DESIGN METHOD

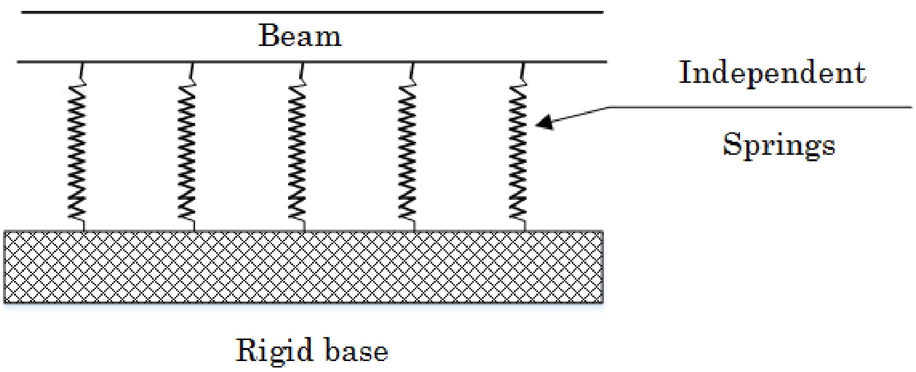
It is intended in this chapter to highlight the available design procedures pertaining to the slab foundation on expansive soil from different sources of published literature. There are more than 13 design methods for slab-on-ground foundations constructed over expansive soils (Wray, 1978; Holland, 1980; Post-Tensioning Institute, 1980; Wire Reinforcement Institute, 1981), but the Budding Research Advisory Board, Lytton, Walsh, Fraser and Wardle, Swinburne, Wire Reinforcement Institute and the Post Tensioning Institute methods are the most rational design methods for slab foundation design.

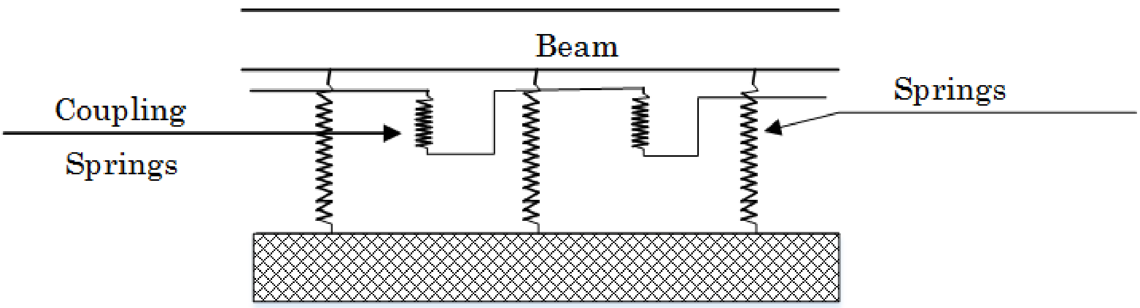
3.1 Lytton method

3.1.1 Lytton (1970) Method

Lytton (1970) procedure based on the mechanics of beams on elastic foundation which is represented by an elastic mathematical model of the soil-structure interaction problem. The design procedure is based on two principles:-

- Mechanism of distortion, and its state which produces the worst design quantitative values, and
- Should be rational to ensure the mathematical model concept.





Rigid base

Fig. 3.1 Winkler spring foundation model

He assumed that the soil properties can be physically specified by two numbers:-

1. Representing soil compressive stiffness (center lift analysis) by independent springs "Winkler Foundation Pattern" and,

2. Representing soil shear stiffness (edge lift analysis) by "Coupled Spring Foundation Pattern", in order to link the activity of the adjacent springs.

Lytton modified the general beam equation by including the effects of shearing resistance, which was represented by the couple springs, of the foundation soil. The differential equation, which was put forward to represent a beam on a coupled spring mound, is given by equation (3.1). The first equation is Plate on Mound equation: for shear stiffness as an isotropic elastic plate rested on a coupled spring mound

$$D\nabla^4 w - \nabla(Gh\nabla(w-y)) + k(w-y) = P \qquad (3.1)$$

Where:- D is the plate flexural rigidity.

w is beam or plate deflection

y is the distance below the highest point of the mound or is the location of pressure – free mound surface in relation to its high point

P is pressure acting on the plate

$$\nabla = \frac{\partial}{\partial x} + \frac{\partial}{\partial y} \qquad \nabla^4 = \frac{\partial^4}{\partial x^4} + 2\frac{\partial^2}{\partial x^2 \partial y^2} + \frac{\partial^4}{\partial y^4}$$, Laplace operation (3.2)

The shape of the curved mound was chosen fit experimentally determined or observed field shapes and was given in the form

$$y = cx^m \tag{3.3}$$

Where:

m = the mound exponent,

c = a constant

x = distance along the beam, and

y = distance below the highest point of the mound.

The second equation is Beam-on-Mound equation (solved by using finite element method)

$$\frac{d^2}{dx^2}\left(EI\frac{d^2w}{dx^2}\right)-\frac{d}{dx}\left(GhB\frac{d}{dx}(w-y)\right)+kB(w-y)=q \tag{3.4}$$

Where:

EI = beam flexural stiffness,

w = transverse deflection of the beam,

y = position of mound,

G = effective soil shear modulus,

h = effective depth within which soil shearing resistance is mobilized,

B = effective width within which soil support for the beam is mobilized

k = effective subgrade modulus, and

q = distributed load on the beam.

Lytton proposes that the beam equation can be applied to a slab when the slab is assumed to take a cylindrical deflection pattern, however, it is also pointed out that if two dimensional bending becomes the primary mode of distortion, then the assumption of the cylindrical deflection pattern is not valid. This differential equation applies only in the region where the beam is in contact with the soil, and a second equation, in which kB and GhB are put equal to zero, applies from the points not in contact with the soil. An iterative process is required to locate these points. A rigid beam solution was also developed to determine maximum moment and shear envelopes. The main benefit gained from these studies is an appreciation of the relative importance of the different design variables and the rational mathematical models of soil-structure interaction.

Beam on Mound equation requests ten essential factors to be determined in order to use them in the analysis of beams on mound. These are summarized here under:

Soil Properties:-

1- Compressibility kB

2- Shear stiffness, GhB

3- Maximum differential heave ym (field measurement)

4- Shape of soil profile (mound exponent) m (ranges from 2 to 4).

Superimposed load on beam:

1- Two concentrated loads, P.C.

P is the edge load on beam.

C is the center load on beam

2- Uniformly distributed load, q

Beam Properties:

1- Beam flexural stiffness, EI.

2- Beam length, L.

3- Effective supporting width of beam, B.

In order to determine the design values Lytton (1970) proposed two critical cases of loading for stiffened raft when subjected to edge and center cylindrical support modes.

- ✓ Dividing the stiffened raft foundation into strips comprising edge and internal grillage beams.
- ✓ Using the beam differential equation (3.4) to determine design values through computer programs.

Lytton adopted the Winkler foundation pattern to represent the rigid beam rested on a mound in order to produce a maximum moment (M_{max}) according to the worst loading pattern either for simple beam or cantilever beam. Moment correction (ΔM) which depends upon soil stiffness (kB) and maximum differential mound movement (y_m), should be subtracted from the maximum moment to obtain the design moment.

$$M_{design} = M_{\max} - \Delta M \tag{3.5}$$

$$\Delta M = \alpha \left(\frac{1}{(kb)(y)} \right)^{\left(\frac{1}{m+1}\right)} \tag{3.6}$$

α = is the collection of constants pertaining to loads, beam length,

3.1.2 Lytton's Method (1972)

Lytton continued to improve his 1970's empirical design procedure to step forwards with progressive modifications. Herein Lytton's design method (1972) was similar to BRAB in their concept that; the slab is initially analyzed as a beam supported by one of the two worst cylindrical support modes, but Lytton considered two types of concentrated loads q_e at the beam two ends and q_c at the beam center in addition to the uniformly distributed load (w) along the beam. Basically he assumed the slab and the soil are both rigid, and accordingly the maximum moment is calculated in each direction.

The maximum positive moment due to edge heave in the direction under consideration, L, can be calculated from

$$M = \frac{q_c L' L}{4} + \frac{L^2}{8}\left(2q_e + wL'\right) \tag{3.7}$$

The maximum negative moment due to center heave in the direction under consideration, L, can be calculated from

$$M = \frac{q_c L' L}{2} + \frac{L^2}{8}\left(2q_e + q_c + wL'\right) - \frac{cTL}{8} \tag{3.8}$$

Where: T=total load on the rectangle plate

L=length in the direction under consideration

c=support index.

The one-dimensional design moments in each direction are adjusted to account for plate action in both directions:

In the long direction: $M_L = M\left(1.4 - 0.4\dfrac{L}{L'}\right) \geq M\left(1.5 - c\right)$ (3.9)

In the short direction: $M_S = M\left(1 + 0.9\left(1.2 - c\right)\left(\dfrac{L}{L'} - 1\right)\right)$ (3.10)

Shear and beam stiffness are obtained from the same equations as proposed in BRAB

$$\text{Shear} = \frac{4M}{L} \quad (3.11)$$

$$\text{Beam stiffness} = EI = \frac{ML^2}{12\Delta} \quad (3.12)$$

Lytton (1972) experimentally developed an empirical relationship for support index "c" based on a rational interaction analysis, as follows:

$$c = \frac{m+1}{m+2}\left(\frac{m+1}{m}\frac{1}{ky}\frac{T}{A}\right)\frac{1}{m+1} \quad (3.13)$$

Where: T=total load acting.

A=total slab (rectangle) area.

k=Winkler foundation modulus

y=maximum differential movements of supporting soil

m=mound exponent.

The maximum differential heave(y) and Winkler modulus (k), can be estimated by using Fig. 3.4 and Fig. 3.5. Mound exponent can be determined by using the following equation:

$$m = \frac{L}{Z} \quad (3.14)$$

Where: L=length in the direction of bending

Z=depth to the active zone

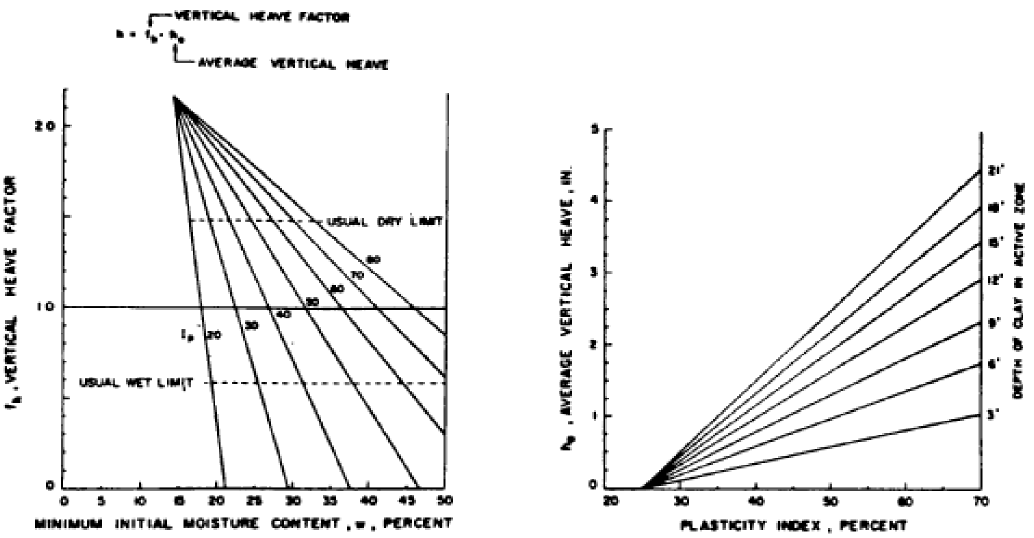


Fig. 3.2 Design aid for estimating differential swell. (After Lytton, 1972)

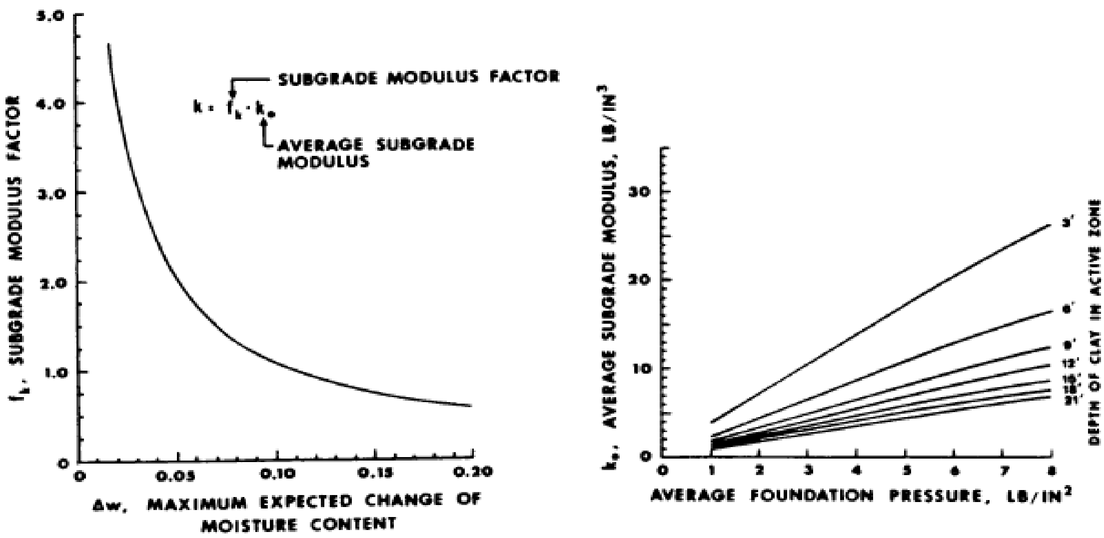


Fig. 3.3 Design aid for estimating subgrade modulus. (After Lytton, 1972)

3.1.3 Lytton and Woodburn (1973)

Lytton and Woodburn reported that the design of stiffened mat on expansive clays is affected mainly by the changing of supporting soil moisture regime beneath the slab. They found the predicted soil movement in laboratory for some Australian expansive soils too closely matches the observed field movement. They built their idea on that soil mat contact is not uniform due to soil moisture variations, beneath the mat. The mat interacts with the non-uniform soil surface within the contact area, pressing down the high spots and bridging the low spots.

This soil-structure interaction can be analytically expressed as a partial differential equation for the deflections of an isotropic plate (mat) rested on a coupled spring mound (soil).

$$D\nabla^4 w - \nabla\left(Gh\nabla\left(w-y\right)\right) + k\left(w-y\right) = P \tag{3.15}$$

Where:- D = is the plate flexural rigidity = EI

Gh = is supporting soil shear stiffness.

k = is the effective subgrade modulus.

w = is the plate deflection.

y = is the elevation of the mounded soil relative to its high point.

P = is the pressure acting on the plate.

The solution for the plate differential equation was based on trial and error depending on the locations where the mat lifts off the supporting soil. A computer analysis was performed by Lytton (1970b) leading to design empirical coefficients which related the applied loads and loading patterns to the maximum design quantities (moment, shear and differential deflection). Lytton re-used the same computer studies in the new (1973) design method, however, before proceeding to the design procedure some important comments reported by (Lytton and Woodburn 1973) concerning mat design are illustrated hereunder:-

- Usually, the stiffened mat is constructed under-reinforced and the mats fails, mainly due to the tensile cracking which occurs in the high moment location.
- Moment design based on the assumption of a cracked section is not economical for two important reasons:-
  1. Cracked section design obviously needs large quantity of steel reinforcing and that is expensive from economical aspect.

2. Occasionally under-reinforced mats crack and bend forming the mound shape in tension, subsequently a great loss of mat flexural stiffness resulting in sharp curvatures which induce cracks in the superstructure.

Lytton & Woodburn (1973) Design procedure

- This design procedure is used for mats with a maximum dimension of 80ft (~ 20m) (Lytton & Woodburn 1973; Nelson & Miller (1992).
- In case of irregular shaped mat, the mat should be divided into rectangles, each designed separately. Subsequently calculating the maximum moment for both directions as follows: bear in mind the both mat and supporting soil are assumed to be rigid.

For maximum positive moment (Edge heave)

$$M_{\max} = \frac{cL}{4} + \frac{q(L')^2 L}{8} \qquad (3.16)$$

For maximum negative moment (center heave)

$$M_{\max} = -\frac{PL'}{2} - \frac{q(L')^2 L}{8} \qquad (3.17)$$

To consider the influence of the soil compressibility a moment correction (ΔM) should be subtracted from the maximum moment to obtain the one dimensional design moment M.

$$M = M_{\max} - \Delta M \qquad (3.18)$$

Moment correction $\Delta M = c\dfrac{TL}{8}$

Where:- T = is the total load on the rectangle plate

c = is the soil support index.

L = is the length in the direction under consideration.

L'= is the width of slab in the other direction of L

Support index c can be determined from Fig. 3.4.

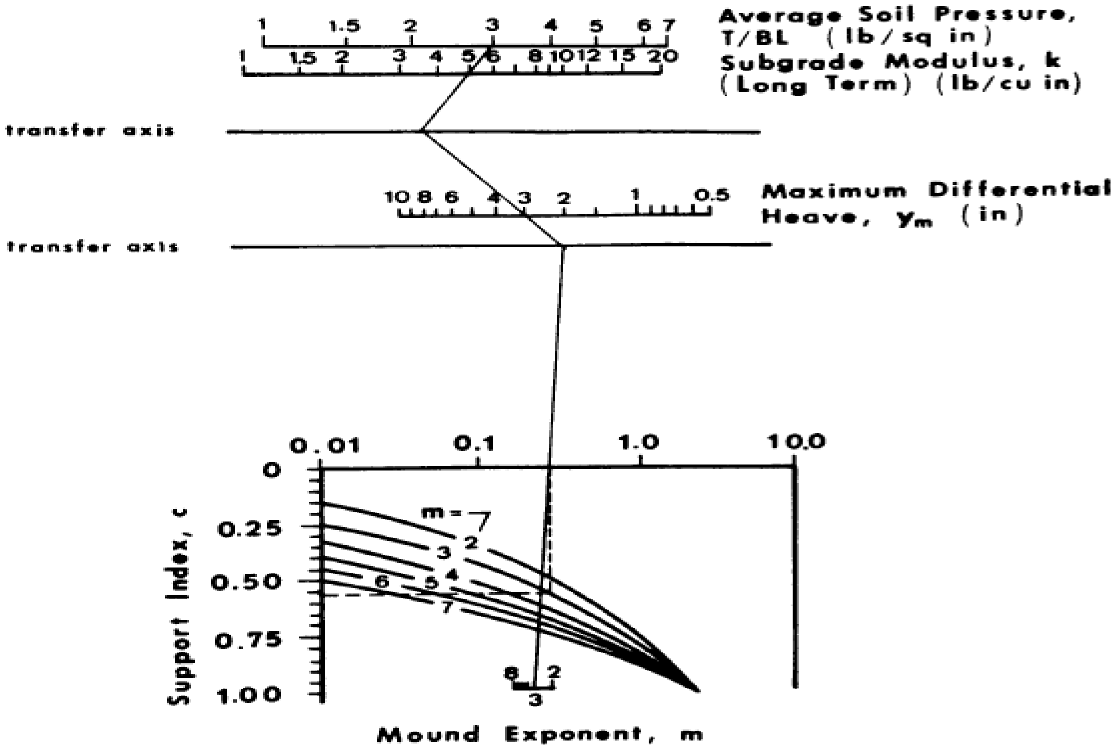


Fig. 3.4 Support index nomograph. (After Lytton and Woodbum, 1973)

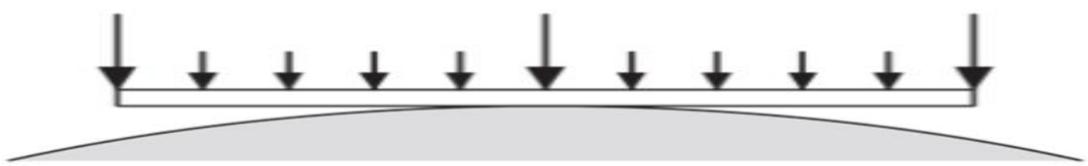
3.2 Walsh's Method

Walsh (1974) method is a combination of Building Research Advisory Board (BRAB) (1968) and Lytton (1970a) procedures. Walsh experimentally attempted to rationalize the determination of support index "c" following the BRAB procedure. He used the same BRAB (1968) design concept that involves splitting up the irregular slab into overlapping rectangles (for analysis simplicity purposes) each one is individually analyzed in both short and long direction assuming the slab rectangular strip as a beam rested on either center or edge mound patterns Fig.3.5.With the aid of finite element analysis method, equation (3.3) can be conveniently solved by representing the beam element and the soil mound by a coupled spring foundation. The solution requires an estimation of four parameters. The soil stiffness constant (k), moisture variation distance (e), soil maximum differential heave (y_m) and the beam allowable deflection Δ/L from Table 3.1 and then from the analysis arranged in non-dimensional terms $\frac{w}{ky_m}, \frac{e}{L}, \frac{\Delta}{y_m}$

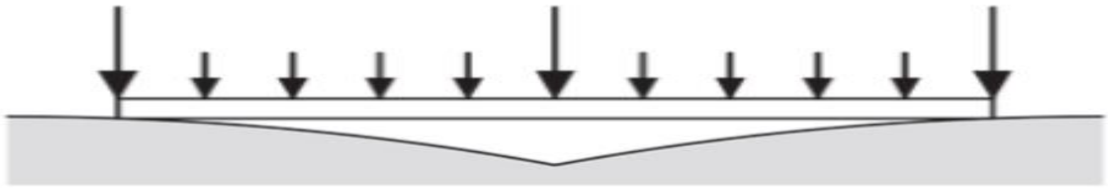
Table 3.1 Allowable differential deflection ratios for slab-on-grade to limit Damage to superstructure

Type of Construction	Maximum Allowable Deflection Ratio (Δ /L)	Reference
Wood Frame	1/200	BRAB(1968)
Nonmasonry, timber or prefabricated	1/200	Woodburn (1974)
Unplastered masonry or gypsum wallboard	1/300	BRAB (1968)
Nonmasonry, frame and panel	1/300	Mitchell (1980)
Stucco or plaster	1/360	BRAB (1968)
Brick veneer (articulated)	1/300	Woodburn (1974)
Brick veneer	1/480	Walsh (1978)
Brick veneer (standard)	1/500	Woodburn (1974)
Masonry (completely articulated)	1/500	Mitchell (1980)
Masonry (partially articulated)	1/800	Woodburn (1974)
Fully articulated solid brick	1/1000	Holland and Lawrence (1980)
Masonry, solid or cavity wall	1/1500-1/2000	Woodburn (1974); Holland (1980)

Finally the design values moment and stiffness can be easily determined by using equations which are similar to BRAB. Walsh continued his attempts by modifying Lytton mound shape to be flat at the middle portion with smooth parabolic ends fig. 3.5.a and 3.5.b in order to simplify soil-structure interaction analysis, and to overcome the problem that occurs while using Lytton's method with large differential heave values (y_m).



a) Center lift



b) Edge lift

Fig. 3.5 Soil-structure interaction with initial mound assumed by Walsh method. (After Walsh, 1974).

3.3 Fraser and Wardle Method

The design of slab foundations on swelling soils requires the consideration of immediate settlement, long term settlement, and settlement or heave because of moisture migration in the supporting soil. Fraser and Wardle (1975) developed a rational solution method by using immediate and long term settlement using a three-dimensional semi-infinite elastic soil foundation numerical model instead of a Winkler foundation or coupled spring foundation model. The computer program, FOCALS, was developed for the design of raft foundations on a cross anisotropic layered system.

Through the use of surface elements, the discretization of a full three-dimensional foundation was reduced down to two dimensions. The design method did not require the slab to be divided into rectangular areas and simplified loading conditions as was required in the Lytton and Walsh methods. Conventional plate and beam finite elements represent the structure and foundation in the Fraser and Wardle method. Surface elements represent a layered soil system beneath the foundation. The immediate and long-term settlements can be determined by using undrained and drained moduli for the soil layers.

Heave profile has been considered as the most severe deformation case to provide an upper bound solution. The method has the same problem as the Lytton and Walsh methods for defining mound shape and edge moisture distance.

3.4 Swinburne Method

It was developed by Holland et al. (1980) from an exhaustive analysis of a modified version of Fraser and Wardle (1975) method and the observed behavior of experimental and housing slabs. The principal assumptions of the method are:

1. The slab-soil interaction model is assumed as an elastic mound with parabolic edges.
2. Edge distance, e, and maximum differential mound heave, y_m, should be estimated from the following equations (Holland and Lawrence, 1980):

$$e = (S_F - S_L)^2 \text{ in feet} \quad (3.19)$$

$$y_m = (S_F - S_L) \text{ in inch} \quad (3.20)$$

Where: S_F = potential vertical rise based on the free swell test

S_L = potential vertical rise based on the loaded swell test.

3. Soil modulus of elasticity and Poisson's ratio are assumed equal to 1150 psi and 0.4, respectively.
4. Long term concrete modulus of elasticity and Poisson's ratio are assumed equal to ($30440(fc^{0.5})$) psi and 0.2, respectively.
5. Concrete shear stresses are assumed to not be significant.
6. Normal single and double-story loading have the same effect on the soil-slab interaction model.
7. Although edge heave is the most likely case for actual housing slabs, it does not lead to the case for critical design.
8. A conservative effective prestress of 75 psi has been assumed.
9. Concrete tensile strength, f_t, is assumed as shown below:

   Steel bar (rebar): $4.2(fc^{0.5})$, psi

   Post-Tensioned: $4.2(fc^{0.5}) + 75$, psi

   Steel Fiber: Modulus of rupture/factor of safety depend on mix design

10. Steel bar (rebar) slab beam steel, A_{sb}, is assumed equal to 0.2bd/100, in $in.^2$/beam

The Swinburne design method is limited with a maximum dimension of slab length 100 feet and maximum mound differential heave 5 in.

The Swinburne design method can be summarized as follows:

1. Divide the slab into overlapping rectangles.
2. Choose 28 day laboratory concrete compressive strength, Fc'(Fc'<2600 psi), beam width, b (6in<b<16in), and slab panel thickness, t (3in<t<6in.).
3. Select the appropriate Δ/L ratio from Table 3.2.
4. Select the beam spacing from Table (3.3).
5. Estimate edge distance (e) and mound differential heave, y_m, and determine the moment from figure (3.6) (Chart 1).
6. Calculate the section modulus, Z, as:

$$Z = \frac{M}{f_t} \tag{3.21}$$

Where: M = moment

f_t = concrete tensile strength.

Table 3.2 Allowable Curvature Deflection Ratios (Δ/L).
(After Holland et al, 1980)

Code	Superstructure Type	Δ/L
A	Stucco, Timber and Articulated Brick Veneer	1 in 250
B	Brick Veneer	1 in 500
C	Fully Articulated Solid Brick	1 in 1000
D	Solid Brick	1 in 2000

Table 3.3. Recommended Beam Spacing and Slab Panel Reinforcement. (After Holland et ah, 1980)

Edge Distance	Steel bar (rebar) Slab		Post-Tensioned Slab		Fibre Steel
	Steel in.^{2}/in.x10^{-3}	Maximum Internal Beam Spacing (ft)	Cable Spacing (ft)	Maximum Internal Beam Spacing (ft)	Maximum Internal Beam Spacing (ft)
e < 1.5	7.4	No Internal Beams	6.6	No Internal Beams	Depends on the mix
1.5 < e < 3.0	9.7	20	6.0	26	
e > 3.0	9.7	15	5.0	20	

7. Determine actual Δ/L ratio from Fig, 3.6 (Chart II). If Δ/L ratio exceeds the allowable Δ/L ratio, then increase Z accordingly.
8. Calculate the Width Factor, W, for each rectangle as shown below:

$$W = \frac{L}{nb} \qquad (3.22)$$

Where: L=crossing the rectangle dimension
n=number of beams
b=beam width.

Use the maximum W value for the entire slab design.
9. Calculate factors F_z and F_s as shown below:

$$F_z = \frac{ZW}{0.2}\text{ , in } (inch^3/inch) \qquad F_s = \frac{t(W-1)}{0.2}\text{ , in (inch)}$$

10. Using factors F_z and F_s values, determine the beam depth, d, directly from Fig.3.6 (Chart 3).
11. Proportion the steel reinforcing from Table (3.3).
12. If required beam depth is greater than about 30 inches, consideration should be given to reducing the edge distance value, e, and redesigning the slab using a new edge distance value.

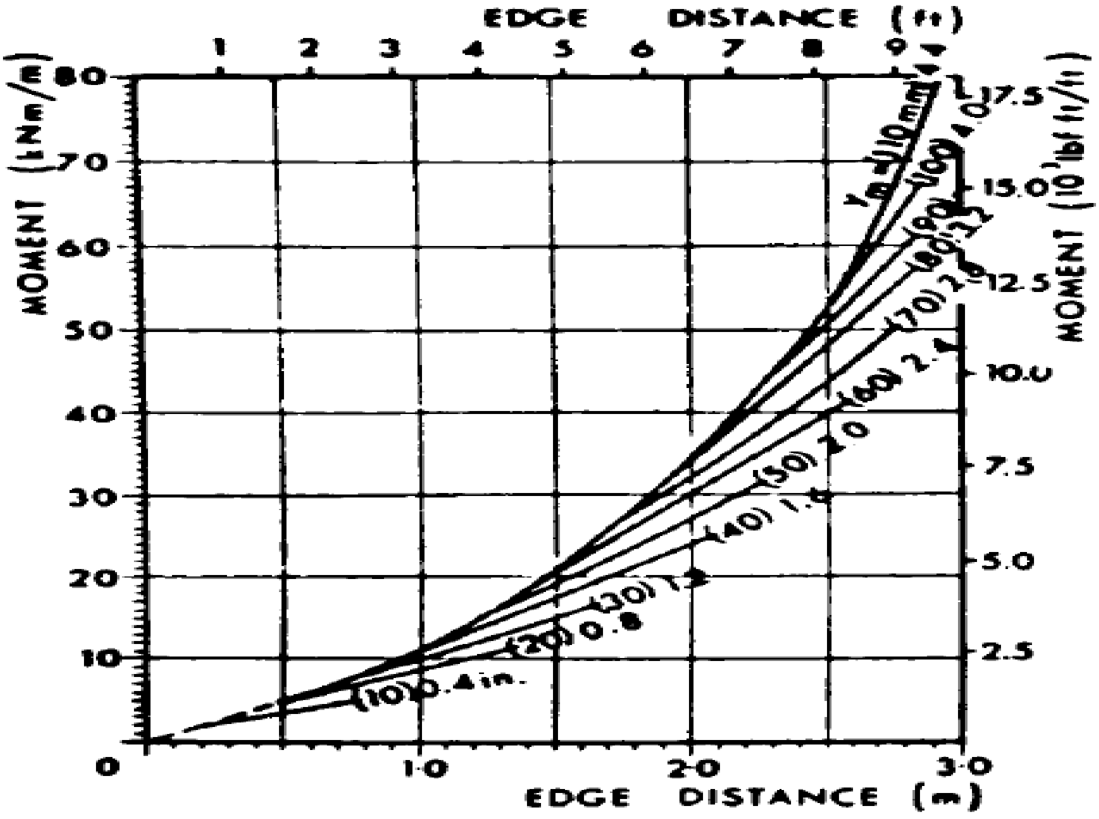


Chart 1

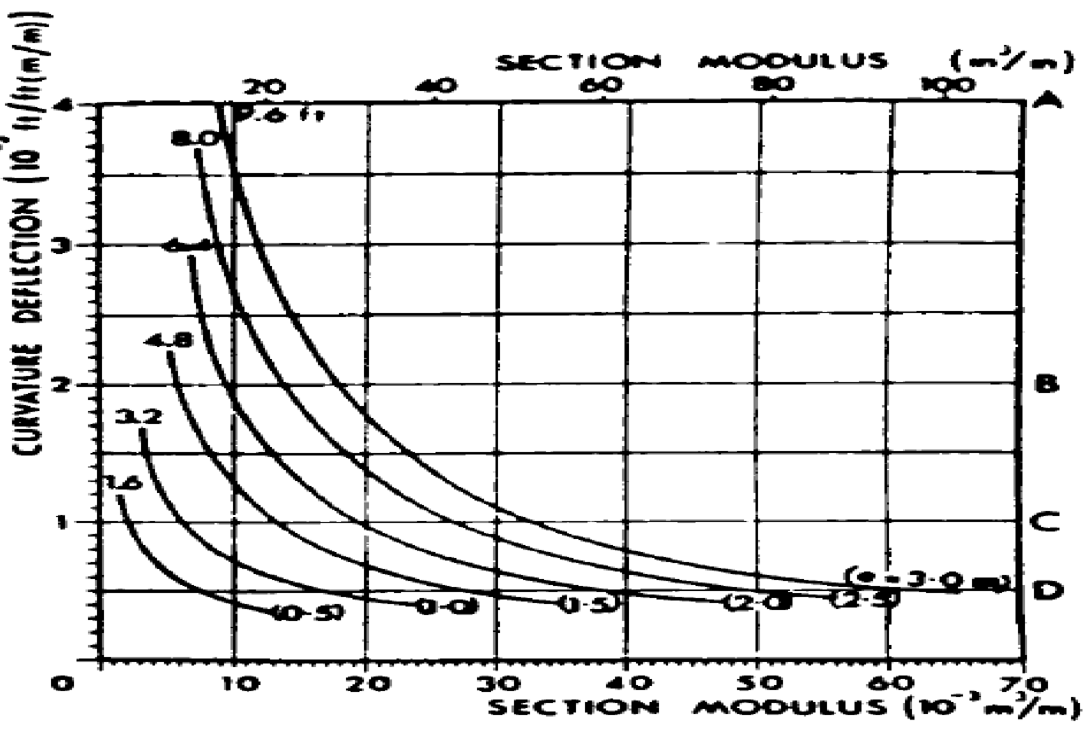


Chart 2

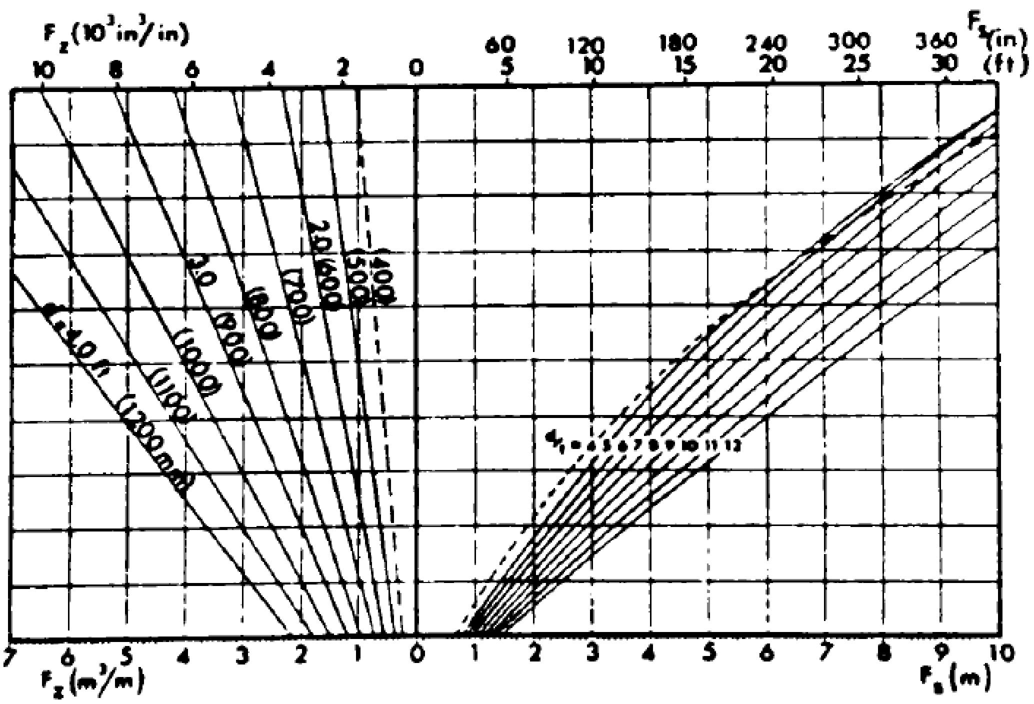


Chart 3

Fig.3.6 Swinburne design method charts. (From Holland et al, 1980)

3.5 Summary

Lytton methods are more realistic but it is not useful for any arbitrary concentrated load on the beam and simple to determine deflection of slab it needs computer program. Use for center and edge concentrated load and uniformly distributed load on the beam. Walsh method following the BRAB procedure to determine the supporting index, so it is localized and Lytton procedure to determine deflection, it needs computer aid. Fraser and Wardle method is more rationalize but it needs FOCAL computer program to design. For this paper to design waffle mat foundation assumed as interconnected T-beam. The beams are sectioned at the middle because of symmetry and have two concentrate load (one load at the edge and the other one is arbitrary location of the beam), uniformly distributed load and linear swelling pressure of soil. These loads found on the beam simultaneously when the soil is swell and the beam assume as simply supported beam as show Fig. 3.7. To determine moment, shear and deflection by using beam on elastic foundation formulas superposition M.Hetenyi (1946). For dry case the beam assume as cantilever beam and to determine moment, shear and deflection use beam formula as show Fig. 3.8.

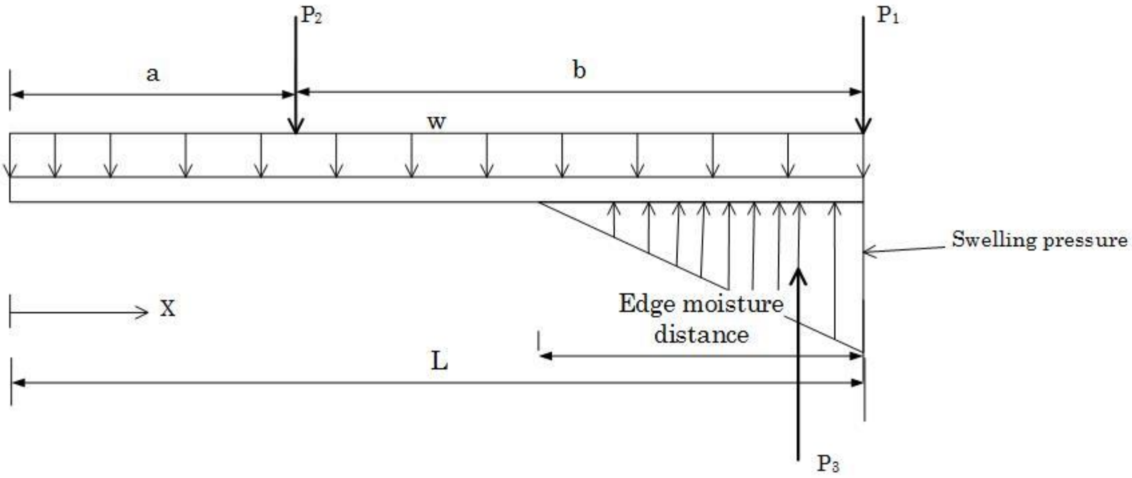


Fig.3.7 Proposed beam model for heave soil

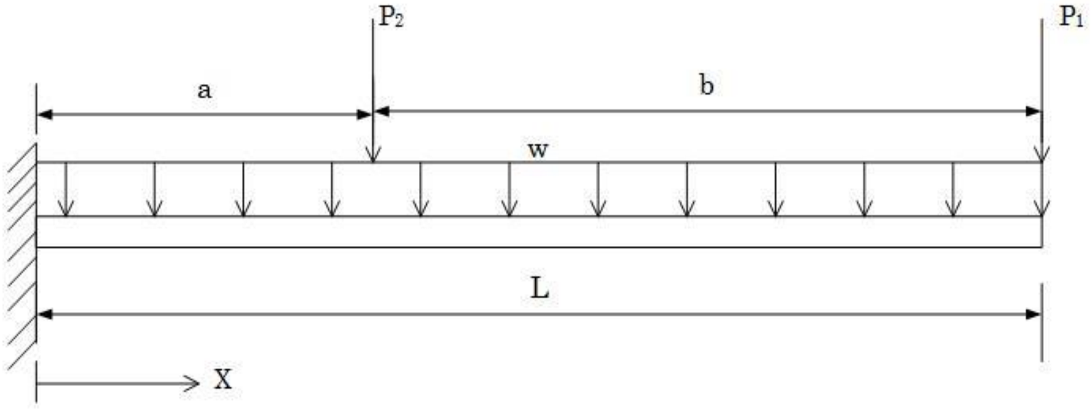


Fig. 3.8 Proposed beam model for dry soil

4 ANALYSIS AND DISGN METHOD OF WAFFLE MAT FOUNDATION

4.1 Volume change of soil beneath a slab foundation

Measurement of soil water diffusivity

1. Estimation of Empirical Diffusivity α

An empirical diffusion coefficient α can be estimated from (Lytton 1994):

$$\alpha = 0.0029 - 0.000162S - 0.0122\gamma_h \qquad (4.1)$$

Where: γ_h = suction compression index

S = Slope of the suction water content curve

$S = -20.29 + 0.1555(LL\%) - 0.117(PI\%) + 0.0684(\% - \#200)$

$$\gamma_h = \gamma_o \left(\frac{\% - 2micron}{\% - No200sieve} \right) \qquad (4.2)$$

$$A_c = \frac{PI\%}{\left(\frac{\% - 2micron}{\% - No200sieve} \right) * 100} \qquad (4.3)$$

PI = plasticity index in percent

$$CEA_c = \frac{CEC}{\left(\frac{\% - 2micron}{\% - No200sieve} \right) * 100} \qquad (4.4)$$

CEC = the cation exchange capacity in mill equivalents per 100 gm of dry soil

CEC = $(LL\%)^{0.912}$ meq/100 g, LL- Liquid limit in percent.

CEC = $(PL\%)^{1.17}$ meq/100 g, PL- Plastic limit in percent

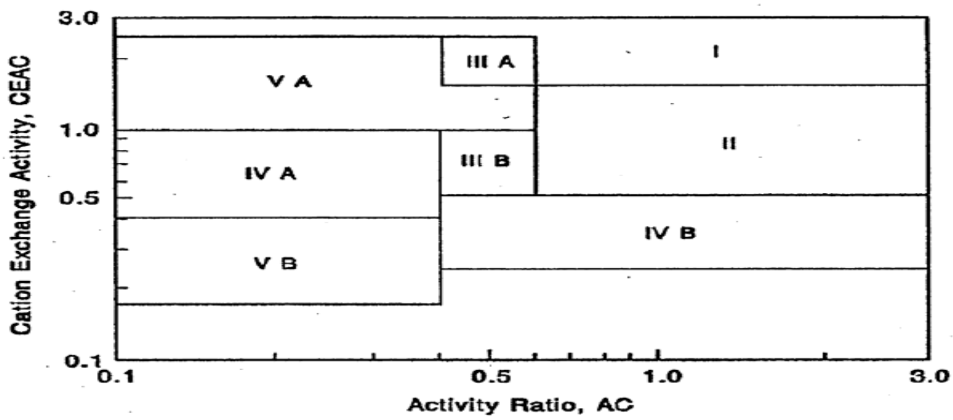


Fig. 4.1 Chart for Prediction of Suction Compression Index Guide Number. (Mckeen, 1981)

Table 4.1 Values for a Soil with 100% Fine Clay Content.

Region	Volume Change γ_0 Guide Number
I	0.220
II	0.163
IIIA	0.096
IIIB	0.061
IVB	0.061
VA	0.033
VB	0.033

To determine the suction compression index in terms of low cost and easily available testing methods (Atterberg limits and soil particle size distributions) to predict soils properties and behavior. This method was developed based on the soil data of the Soil Survey Laboratory (SSL) of the National Soil Survey Center. To get soil suction compression index needs only Liquid Limit (LL), Plastic index and fine clay fraction (%).

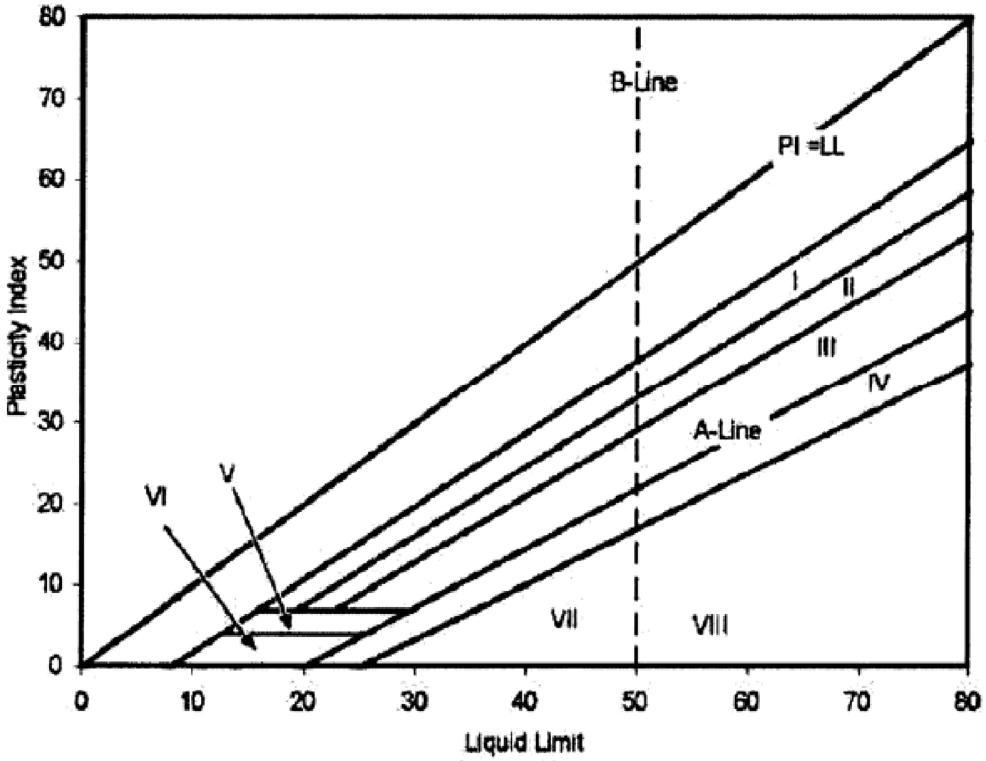


Fig. 4.2 Data Filter for Partitioning Database on Mineralogical Types. (After Casagrande (1948) and Holtz and Kovacs (1981)

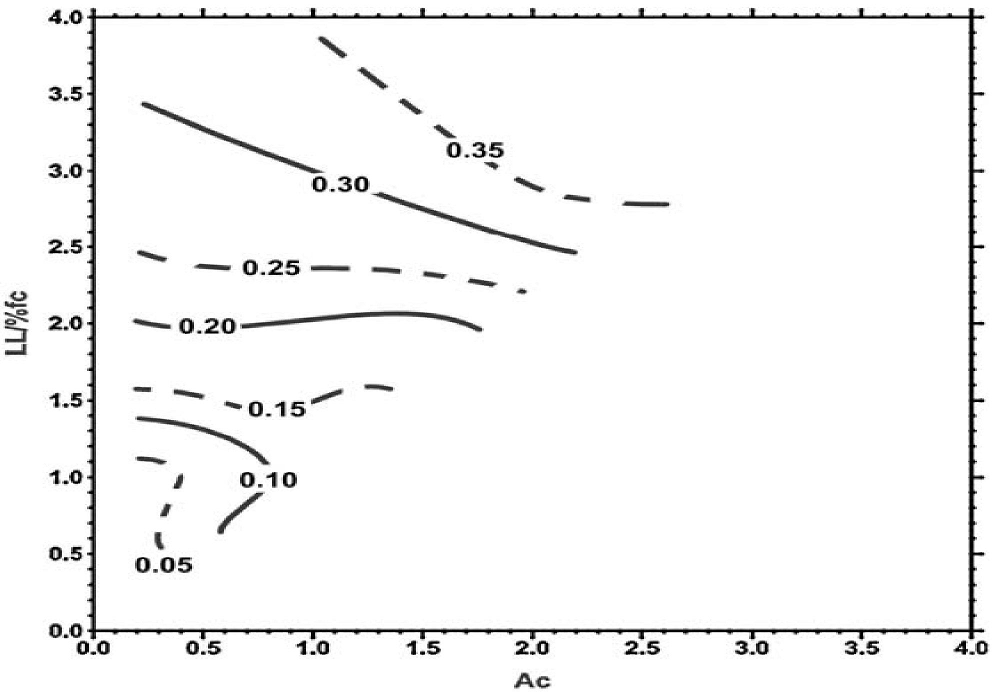


Fig. 4.3 Zone I Chart for Determining γo . (Covar and Lytton, 2001).

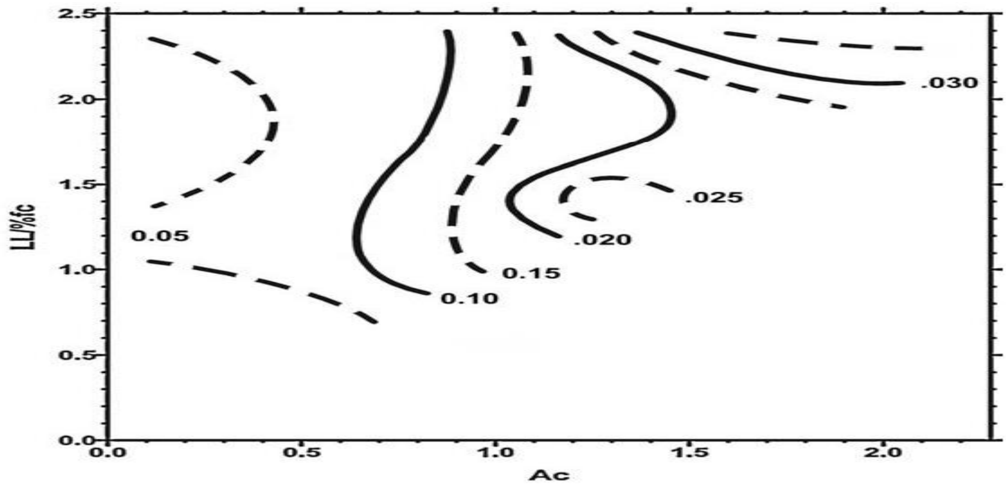


Fig.4.4 Zone II Chart for Determining γ_o . (Covar and Lytton, 2001)

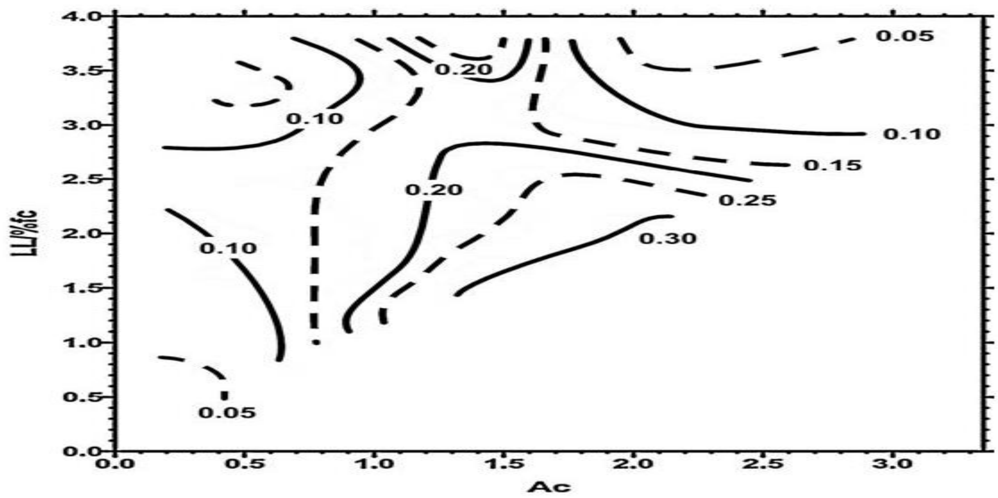


Fig. 4.5 Zone III Chart for Determining γ_o . (Covar and Lytton, 2001)

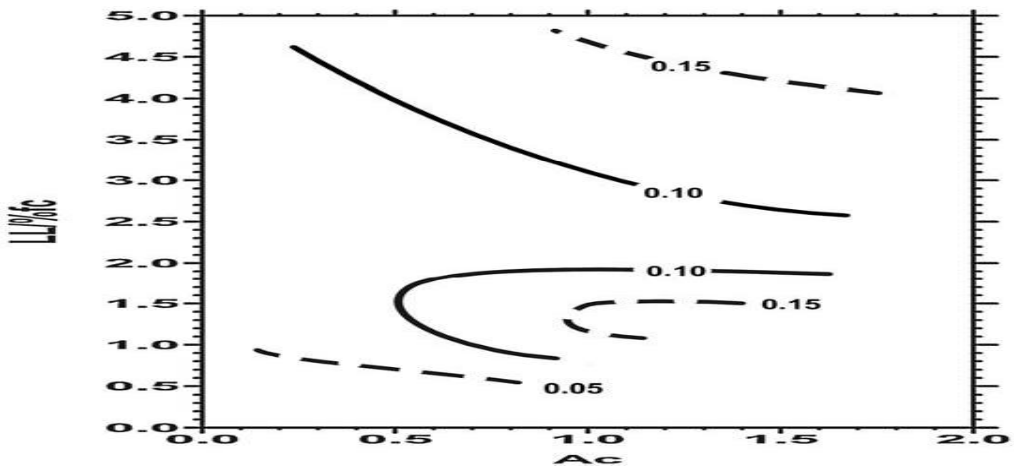


Fig. 4.6 Zone IV Chart for Determining γ_o . (Covar and Lytton, 2001)

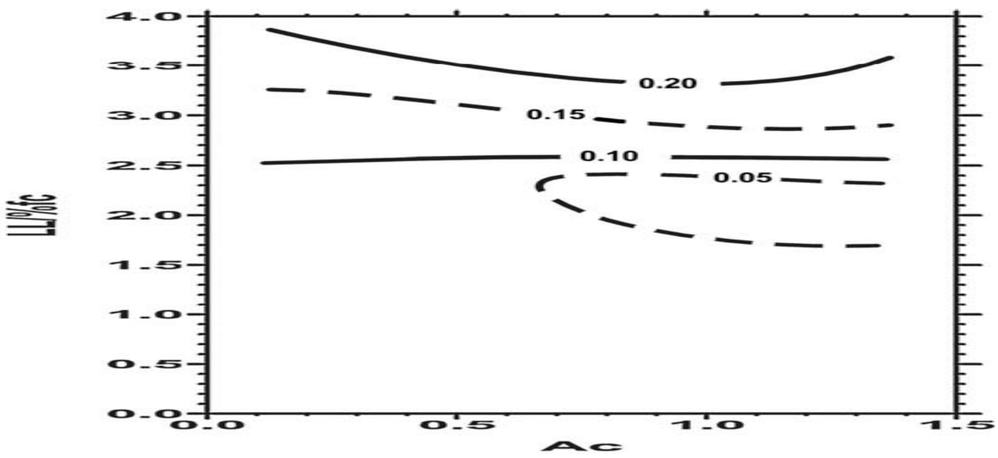


Fig. 4.7 Zone V Chart for Determining γ_o . (Covar and Lytton, 2001)

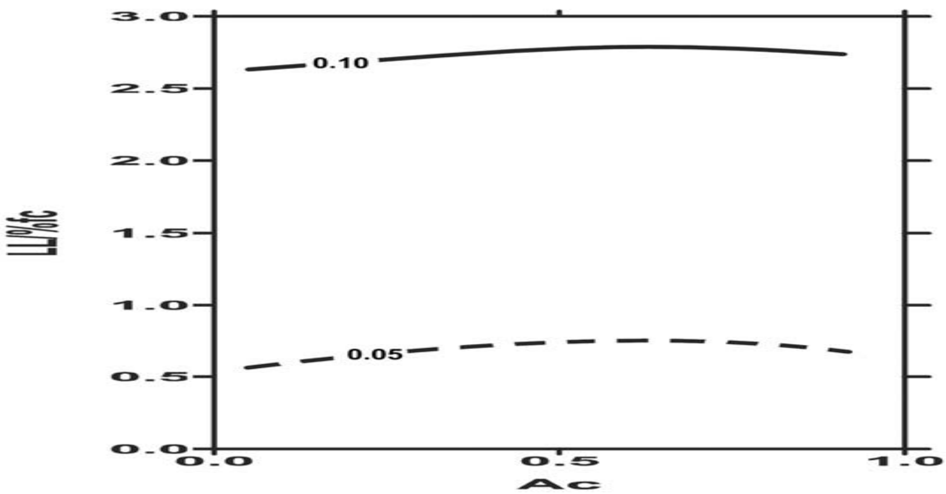


Fig. 4.8 Zone VI Chart for Determining γ_o . (Covar and Lytton, 2001)

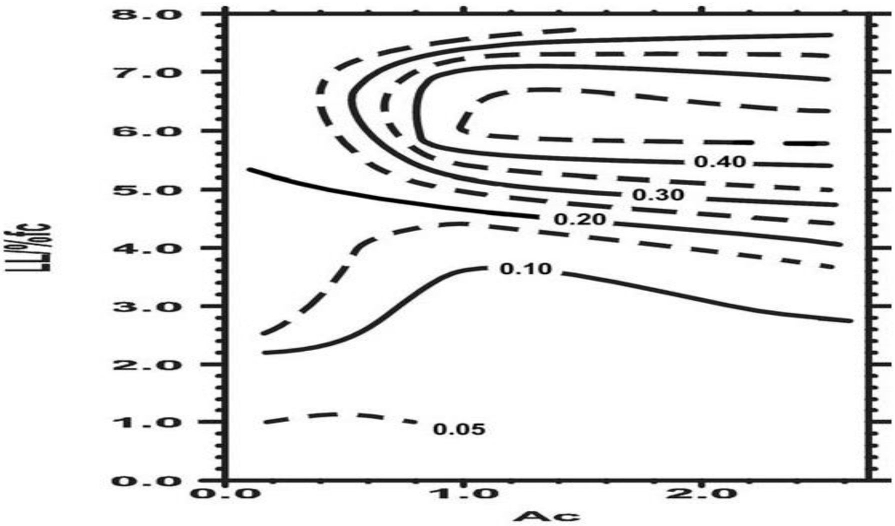


Fig. 4.9 Zone VII Chart for Determining γ_o . (Covar and Lytton, 2001)

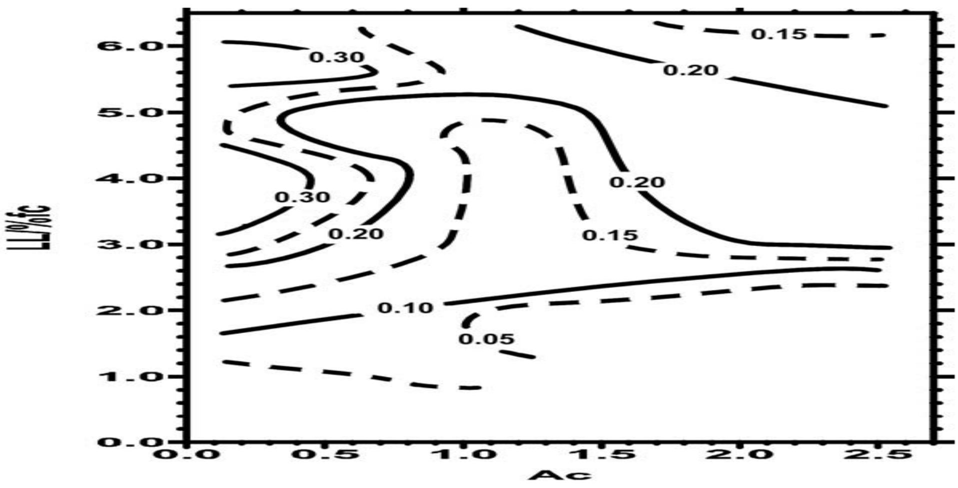


Fig. 4.10 Zone VIII Chart for Determining γ_o . (Covar and Lytton, 2001)

The suction compression index obtained from Equation (4.2) is corrected to compensate for the different initial volume of soil mass during a wetting or drying process (Lytton R. 2004)

$$y_{h(swelling)} = y_h e^{\gamma h} \tag{4.5}$$

$$y_{h(shrinkage)} = y_h e^{-\gamma h} \tag{4.6}$$

The ratio of vertical to volumetric strain, coefficient f

The mobilized volumetric strain, due to moisture movements, to a vertical strain, an important parameter for waffle mat foundation to consider the void portion of the foundation coefficient, f, is needed. f is defined as the ratio between the mobilized vertical strain to the mobilized volumetric strain (i.e. f = ξver / ξvol).

Now let us consider a soil mass at the surface of soil- structure interaction.

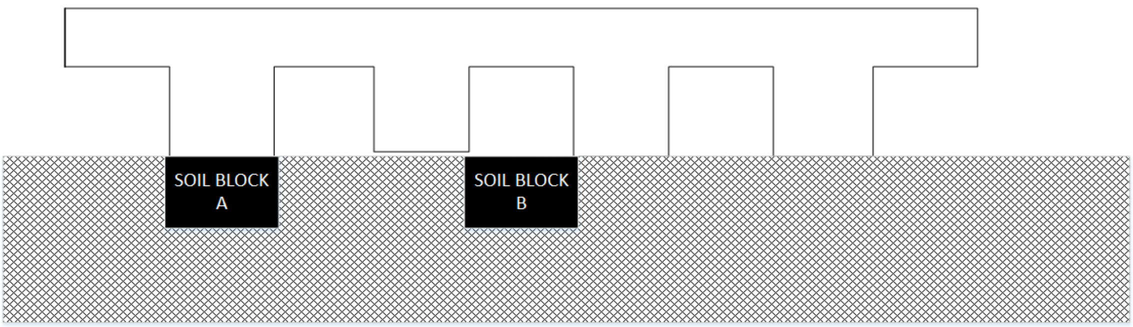
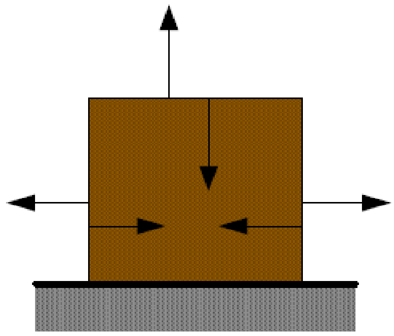


Fig. 4.11 Model of waffle mat foundation on ground

a) For soil block A there are four conditions that depend on vertical and horizontal subgrade reaction. Subgrade reaction also depends on stress and module of elasticity of the soil according to vertical and horizontal direction. If the soil assume to homogenous soil vertical and horizontal direction of module of elasticity are equals. So subgrade reaction only depends on vertical stress due to structural imposed load and horizontal stress due to confinement of soil R. Abdelmalak (2007).



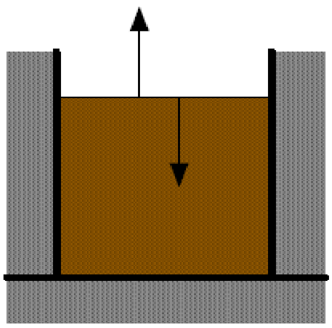
i) $If \ \ 0 < \frac{\sigma_v}{\sigma_h} < 1 \ \rightarrow \ 1 > f > 0.33$

ii) $If \ \ \frac{\sigma_v}{\sigma_h} = 1 \ \rightarrow \ f = 0.33$

iii) $If \ \ \frac{\sigma_v}{\sigma_h} > 1 \ \rightarrow \ 0.33 > \mathrm{f} > 0$

iv) $If \ \ \frac{\sigma_v}{\sigma_h} \cong \infty \ \rightarrow \ \mathrm{f} = 0$

b) For soil block B that found void place of the foundation are only vertical strain because of laterally confined soil. So the vertical strain equal to volumetric strain and the coefficient of $\mathrm{f} = 1$



4.2 Result and discussion

Proposed mound shape equation

The proposed equation derived by Remon I. Abdelmalak (2007), he depends on pioneer work Mitchell (1979) for suction distribution on soil. Assume the soil surface was partially covered by a weightless impervious flexible cover and exponential decay suction distribution under the cover edge.

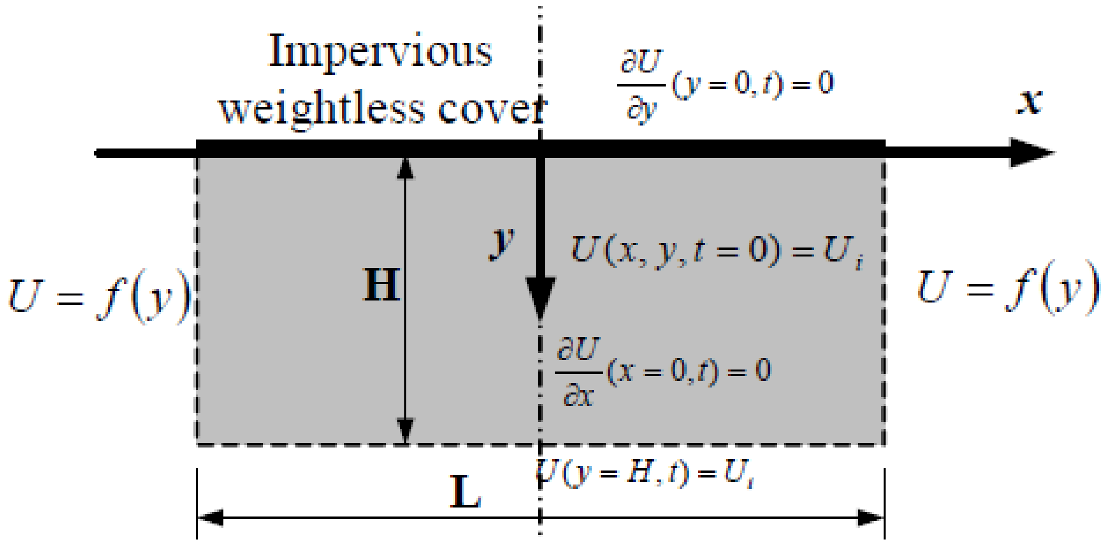


Fig. 4.12 Boundary conditions for the impervious weightless cover problem.

$$U(y) = U_i + \Delta U \exp\left(-\sqrt{\frac{\omega}{2\alpha_{field}}}\,y\right) \qquad (4.7)$$

4.2.1 Parametric Sensitivity Study for Soil

For moisture distribution with depth

A parametric study was conducted for equation (4.7) for predicting moisture profiles in the depth of subgrade soils. The main parameters involved in equation (4.7) are the field diffusion coefficient (α_{field}), weather periodic time (ω), edge suction change (ΔU_{edge}). It is noted that the final suction boundary condition is specified on the ground surface, and assuming an exponential decay moisture distribution under the cover edge with depth within the subgrade. Initial (equilibrium) suction profile is considered to be constant with depth within the subgrade and covered edge suction change (ΔU_{edge}) is equal to half of uncovered surface suction change (ΔU). In the analysis, the initial suction profile is assumed to be 3.5 pF. The other parameters are varied.

a) Surface edge suction

The moisture distribution profile using equation (4.7) by varying surface edge suction (ΔU_{edge}) in 0.1 pF increments while considering an average constant diffusion coefficient (4.02×10^{-3} cm^2/min) and one year of drying period. Fig. 4.13 depicts the moisture distribution profiles using the data in Table 4.2. It shows the effect of various surface suctions on the moisture distribution profile as they form over a period of one year under an average constant diffusivity value. The figure also indicates that the depth of moisture distribution vary from 1m to 1.25 m when change of surface edge suction increase from 0.1PF (soil with no crack) to 0.5PF (soil with crack). So surface edge suction has less effect on moisture distribution with depth.

Table 4.2 Moisture Distribution vs Depth at Different Surface Edge Suctions

α_{field} = 0.00402 cm^2/min		ω=0.0000119	U_i = 3.5PF	T = 1 year	
Depth in m	Δu_{edge}(0.5PF)	Δu_{edge}(0.4PF)	Δu_{edge}(0.3 PF)	Δu_{edge}(0.2PF)	Δu_{edge}(0.1PF)
0	4.00	3.90	3.80	3.70	3.60
-0.25	3.69	3.65	3.61	3.58	3.54
-0.5	3.57	3.56	3.54	3.53	3.51
-0.75	3.53	3.52	3.52	3.51	3.51
-1.00	3.51	3.51	3.51	3.50	3.50
-1.25	3.50	3.50	3.50	3.50	3.50
-1.50	3.50	3.50	3.50	3.50	3.50
-1.75	3.50	3.50	3.50	3.50	3.50
-2.00	3.50	3.50	3.50	3.50	3.50
-2.25	3.50	3.50	3.50	3.50	3.50
-2.50	3.50	3.50	3.50	3.50	3.50
-2.75	3.50	3.50	3.50	3.50	3.50
-3.00	3.50	3.50	3.50	3.50	3.50

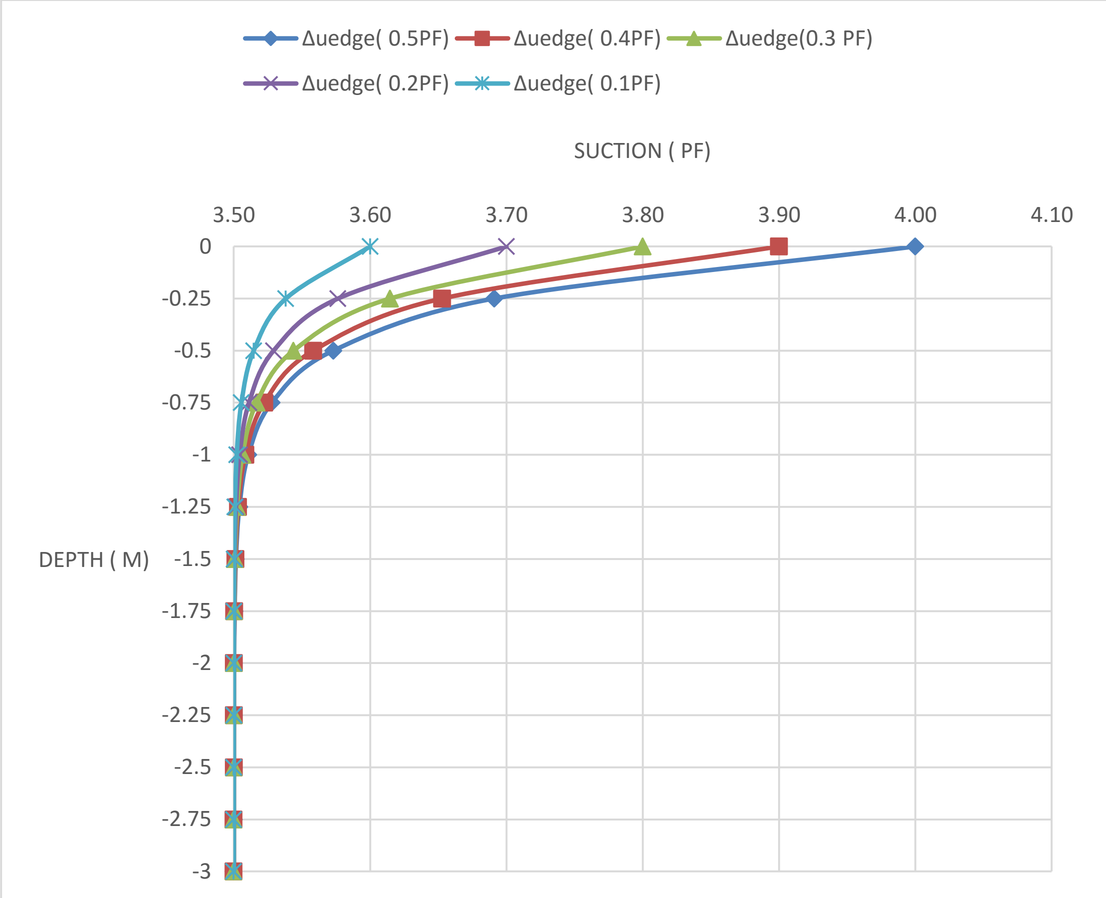


Fig. 4.13 Moisture Distributions with Depth at Different Edge Suctions

b) Drying Period

The effects of various drying periods on the moisture distribution profiles at constant edge surface suction of 0.5 pF and diffusivity coefficient of 4.02×10^{-3} cm²/min. Fig. 4.14 shows the moisture distribution profiles using the data in Table 4.3. As the surface of the subgrade is exposed to a high suction value (4.5 pF which is close to the wilting point of vegetation), the suction envelopes expand laterally with increasing times. The depth of moisture distribution vary from 0.75m, 0.75m and 1.25m when the drying period increase from 3 month, 6month and 1 year respectively. So the drying period also has less effect on moisture distribution with depth.

Table 4.3 Moisture Distribution vs Depth at Different Drying Times

α_{field} = 0.00402 cm2/min		Δu_{edge}= 0.5PF	U_i =3.5PF		
y in m	1year	9month	6month	3month	1month
0	4.00	4.00	4.00	4.00	4.00
-0.25	3.69	3.66	3.63	3.57	3.52
-0.5	3.57	3.55	3.53	3.51	3.50
-0.75	3.53	3.52	3.51	3.50	3.50
-1	3.51	3.51	3.50	3.50	3.50
-1.25	3.50	3.50	3.50	3.50	3.50
-1.5	3.50	3.50	3.50	3.50	3.50
-1.75	3.50	3.50	3.50	3.50	3.50
-2	3.50	3.50	3.50	3.50	3.50
-2.25	3.50	3.50	3.50	3.50	3.50
-2.5	3.50	3.50	3.50	3.50	3.50
-2.75	3.50	3.50	3.50	3.50	3.50
-3	3.50	3.50	3.50	3.50	3.50

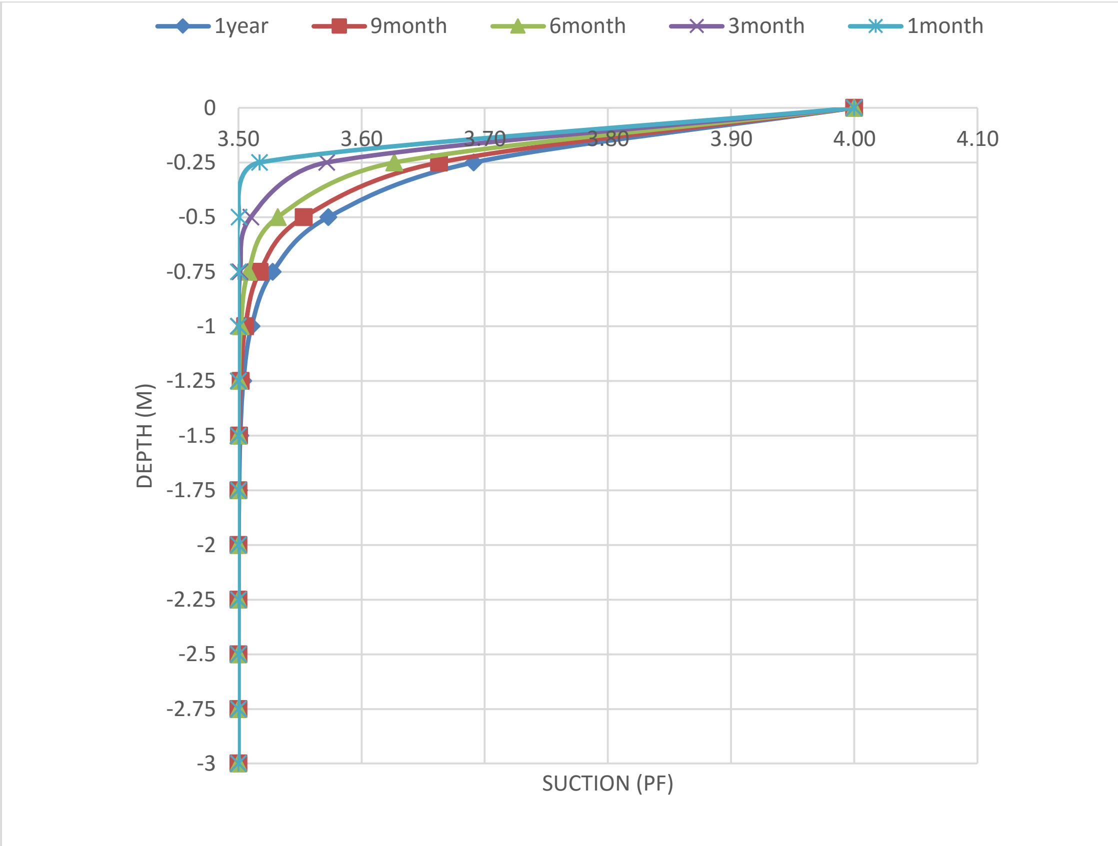


Fig. 4.14 Moisture Distributions with Depth at Different Drying Times

c) Diffusivity coefficient

The effects of the diffusivity parameter in predicting the moisture distibution profile using equation (4.7) are undertaken for a constant suction boundary condition on the surface of the subgrade during a drying period of 1 year. The diffusivity parameter was changed from a small value (1.0×10^{-6} cm2/min) representing a tight soil with no cracks and to a large value (1.0×10^{-1} cm2/min) representing a loose soil with cracks. Table 4.4 and Fig. 4.15 give the moisture envelopes showing the effects of diffusivity. The depth of moisture distribution vary from 0.75m to exceed 3m when diffusitive coefficient increase from 0.001 cm^2 /min (soil with no crack) to 0.1 cm^2 /min (soil with crack). This indicate that moisture distribution with depth greatly affected by diffusitive coefficient.

Table 4.4 Moisture Distribution vs Depth at Different Diffusive Coefficients

Δuedge=0.5PF	Ui= 3.5PF		T = 1 year			
y in m	α=0.1 (cm2/min)	α=0.01 (cm2/min)	α=0.001 (cm2/min)	α=0.0001 (cm2/min)	α=0.00001 (cm2/min)	α=0.000001 (cm2/min)
0	4.00	4.00	4.00	4.00	4	4
-0.25	3.91	3.77	3.57	3.50	3.5	3.5
-0.5	3.84	3.65	3.51	3.50	3.5	3.5
-0.75	3.78	3.58	3.50	3.50	3.5	3.5
-1	3.73	3.54	3.50	3.50	3.5	3.5
-1.25	3.69	3.52	3.50	3.50	3.5	3.5
-1.5	3.66	3.51	3.50	3.50	3.5	3.5
-1.75	3.63	3.51	3.50	3.50	3.5	3.5
-2	3.61	3.50	3.50	3.50	3.5	3.5
-2.25	3.59	3.50	3.50	3.50	3.5	3.5
-2.5	3.57	3.50	3.50	3.50	3.5	3.5
-2.75	3.56	3.50	3.50	3.50	3.5	3.5
-3	3.55	3.50	3.50	3.50	3.5	3.5

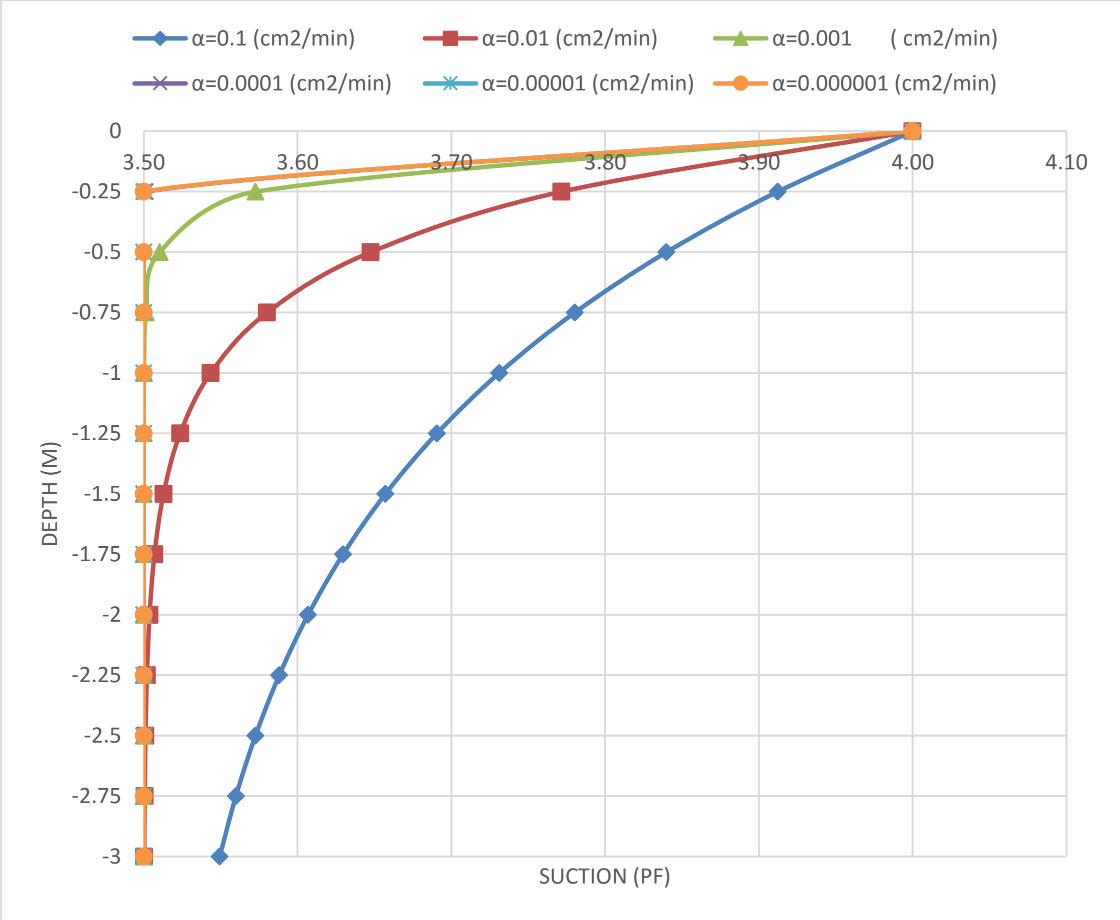


Fig. 4.15 Moisture Distributions with Depth at Different diffusive Coefficient

Table 4.5 Summary of moisture distribution with depth due to different parameters

			Suction distribution with depth		
	Increase		Increase		Increase in percent (%)
Parameters	From	To	From	To	
Change of surface edge suction	0.1PF	0.5PF	1m	1.25m	25.00
Drying period	6 month	1 year	0.75m	1.25m	66.67
Diffusitive coefficient	0.001 cm2/min	0.1 cm2/min	0.75m	exceed 3m	more than 100 percent

The above table indicate that deffusitive coefficient has more sensitivity to moisture distribution with depth than change of surface edge suction and drying period.

For soil mound and edge moisture distance

Remon I. Abdelmalak (2007) also proposed new mound shape equation (4.8) of soil partially covered by a weightless impervious flexible as shown in figure (4.12). A parametric study was undertaken to evaluate equation (4.8) for predicting surface mound shape profiles in the horizontal direction of subgrade soil.

$$\Delta H\left(x\right)=f\gamma_h H\Delta U_{edge}\sum_{n=1}^{\infty}\left(\left[\frac{2\pi\left(n-\frac{1}{2}\right)\left(-1\right)^{n-1}\exp\left(-\sqrt{\frac{\omega H^2}{2\alpha_{field}}}\right)+2\sqrt{\frac{\omega H^2}{2\alpha_{field}}}}{\frac{\omega H^2}{2\alpha_{field}}+\pi^2\left(n-\frac{1}{2}\right)^2}\right] * \left[\frac{\cosh\left(\left(n-\frac{1}{2}\right)\pi\frac{x}{H}\right)\left(-1\right)^{n-1}}{\cosh\left(\left(n-\frac{1}{2}\right)\pi\frac{L}{2H}\right)\pi\left(n-\frac{1}{2}\right)}\right]\right) \tag{4.8}$$

Where; $\Delta H\left(x\right)$ is the surface soil movement under the impervious cover.

f is ratio of the vertical strain to the volumetric strain.

γ_h is suction compression index .

ΔU_{edge} is the change in suction, which = $U(x,y)-Ui$

ω is frequency = $\frac{2\pi}{T}$, T is weather periodic time

α_{field} is field diffusion coefficient

n is number of weather frequency

d) Active depth

The effects of active depth parameter in predicting the soil mound shape profile in horizontal direction using equation (4.8) are undertaken for a constant suction boundary condition (ΔU=2.5PF) on the surface of the subgrade during a drying period of 1 year. The active depth parameter was changed from a small value 1m representing a tight soil with no cracks and to a large value 3m representing a loose soil with cracks. Table 4.6 and Fig. 4.16 give the soil mound envelopes showing the effects of active depth. Edge moisture distance vary from 1m to 3m and maximum soil heave vary from 33mm to 44mm when active depth increase from 1m (soil with no crack) to 3m (soil with crack).

Table 4.6 Mound shape vs horizontal distance from middle of beam at Different active depth

ΔU=2.5PF	α_{field} = 0.00402 cm²/min		f=1	T= 1year	γ_h=0.035		
X in m	H=1m	H=1.5m	H=2m	H=2.5m	H=3m	H=3.5m	H=4m
0	0.00	0.00	0.00	0.00	0.00	0.00	0.01
0.50	0.00	0.00	0.00	0.00	0.00	0.00	0.01
1.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
1.50	0.00	0.00	0.00	0.00	0.00	0.01	0.01
2.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
2.50	0.00	0.00	0.00	0.00	0.01	0.01	0.01
3.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01
3.50	0.00	0.00	0.00	0.01	0.01	0.01	0.01
4.00	0.00	0.00	0.01	0.01	0.01	0.02	0.02
4.50	0.00	0.01	0.01	0.01	0.02	0.02	0.02
5.00	0.01	0.01	0.02	0.02	0.02	0.02	0.03
5.50	0.01	0.02	0.02	0.03	0.03	0.03	0.03
6.00	0.03	0.04	0.04	0.04	0.04	0.04	0.05

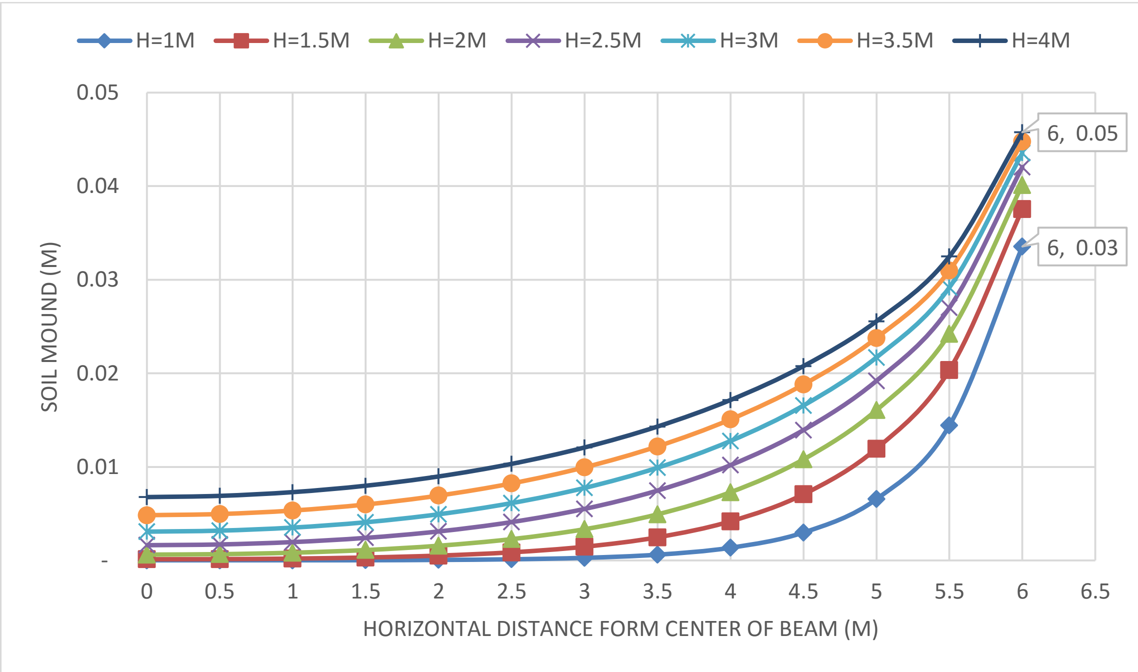


Fig.4.16 Mound shape with horizontal distance at Different active zone

e) Diffusivity coefficient

The effects of the diffusivity parameter in predicting the soil mound using equation (4.8) are undertaken for a constant suction boundary condition on the surface of the subgrade during a drying period of 1 year. The diffusivity parameter was changed from a small value (1.0×10^{-6} cm2/min) representing a tight soil with no cracks and to a large value (1.0×10^{-1} cm2/min) representing a loose soil with cracks. Table 4.7 and Fig. 4.17 give the soil mound envelopes showing the effects of diffusivity. . Edge moisture distances vary from 1.5. to 6m and soil mound 24mm to 138mm when diffusitive coefficient increase from 0.001cm2/min (soil with no crack) to 0.1cm2/min (soil with crack).

Table 4.7 Mound shape vs horizontal distance from middle of beam at Different diffusive coefficient

ΔU=2.5PF	H=3m	f=1	T= 1year	γ_h=0.035		
X(m)	α_{field}=0.1 cm^2/min	α_{field}=0.01 cm^2/min	α_{field}=0.001 cm^2/min	α_{field}=0.0001 cm^2/min	α_{field}=0.00001 cm^2/min	α_{field}=0.000001 cm^2/min
0	0.011433	0.004731	0.001558	0.000495	0.000156	4.95E-05
0.5	0.011829	0.004895	0.001612	0.000512	0.000162	5.12E-05
1	0.013042	0.005397	0.001777	0.000564	0.000179	5.64E-05
1.5	0.015158	0.006273	0.002065	0.000656	0.000207	6.56E-05
2	0.018322	0.007582	0.002496	0.000793	0.000251	7.93E-05
2.5	0.022754	0.009416	0.0031	0.000984	0.000311	9.85E-05
3	0.02876	0.011903	0.003919	0.001245	0.000394	0.000125
3.5	0.036759	0.015217	0.005012	0.001592	0.000504	0.000159
4	0.047311	0.019598	0.00646	0.002052	0.000649	0.000205
4.5	0.061175	0.02539	0.00839	0.002666	0.000844	0.000267
5	0.079423	0.033149	0.011033	0.003512	0.001111	0.000352
5.5	0.103783	0.044068	0.015036	0.004823	0.001528	0.000493
6	0.138495	0.062604	0.024149	0.008234	0.002635	0.000905

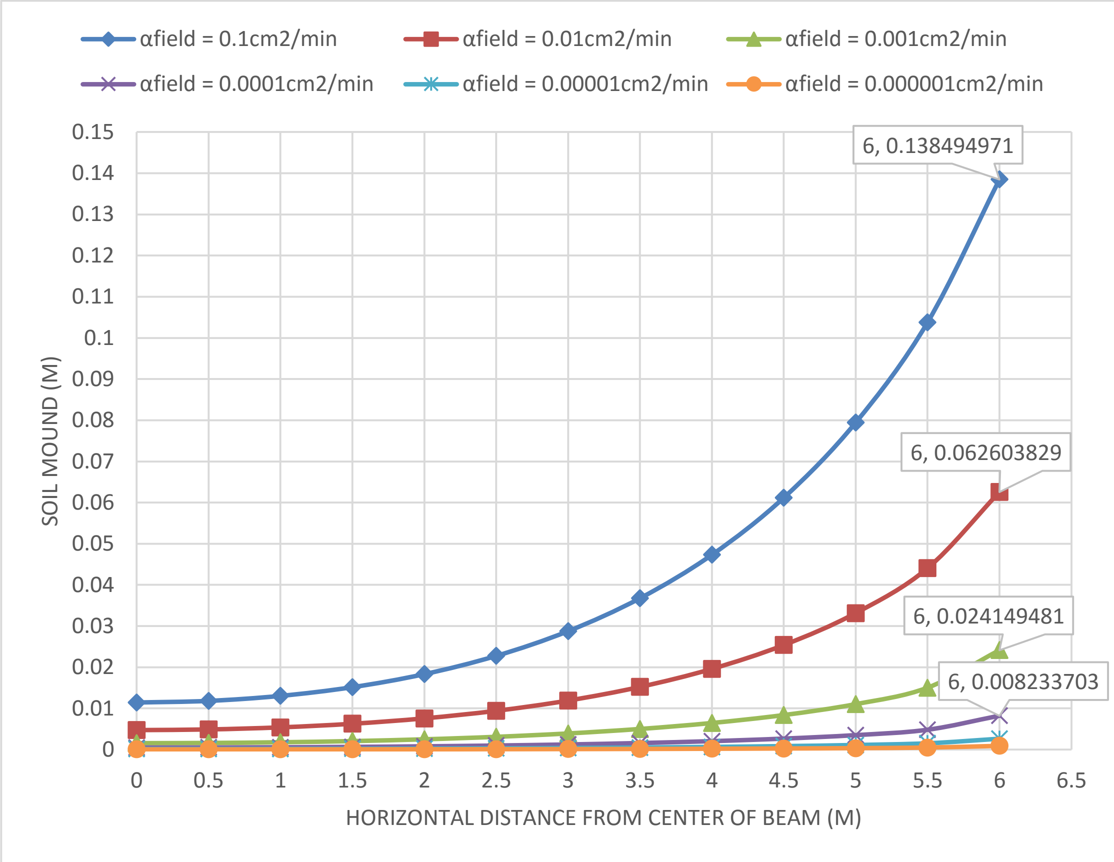


Fig. 4.17 Mound shape with horizontal distance at Different diffusive Coefficient

f) Change of edge suction

The effects of the change of edge suction in predicting the soil mound using equation (4.8) are undertaken for a constant diffusitive parameter condition on the surface of the subgrade during a drying period of 1 year. The change of edge suction was changed from a small value 0.4 PF and to a large value 2.5PF. Table 4.8 and fig. 4.18 give the soil mound envelopes showing the effect of change of edge suction. Edge moisture distances is negligible and 5mm maximum soil mound for 0.4PF but about 2m edge moisture distance from free end of the beam and 35mm maximum soil mound for 2.5 PF.

Table 4.8 Mound shape vs horizontal distance from middle of beam at Different change of edge suction

H=3m	α_{field} = 0.00402 cm²/min		f=1	T= 1 year	γ_h=0.035	
x	ΔU= 0.4PF	ΔU= 0.8PF	ΔU= 1.2PF	ΔU= 1.6PF	ΔU= 2PF	ΔU= 2.5PF
0	0.000391	0.000781	0.001172	0.001562	0.001953	0.002435
0.5	0.000404	0.0008081	0.001212	0.001616	0.00202	0.00252
1	0.000445	0.000891	0.001336	0.001782	0.002227	0.002778
1.5	0.000518	0.0010355	0.001553	0.002071	0.002589	0.003229
2	0.000626	0.0012517	0.001878	0.002503	0.003129	0.003903
2.5	0.000777	0.0015545	0.002332	0.003109	0.003886	0.004847
3	0.000983	0.0019651	0.002948	0.00393	0.004913	0.006127
3.5	0.001256	0.0025126	0.003769	0.005025	0.006282	0.007834
4	0.001619	0.0032373	0.004856	0.006475	0.008093	0.010094
4.5	0.0021	0.0041991	0.006299	0.008398	0.010498	0.013093
5	0.002751	0.0055018	0.008253	0.011006	0.013755	0.017155
5.5	0.003699	0.007399	0.011099	0.014821	0.018497	0.02307
6	0.00552	0.0110407	0.016647	0.022417	0.027602	0.034427

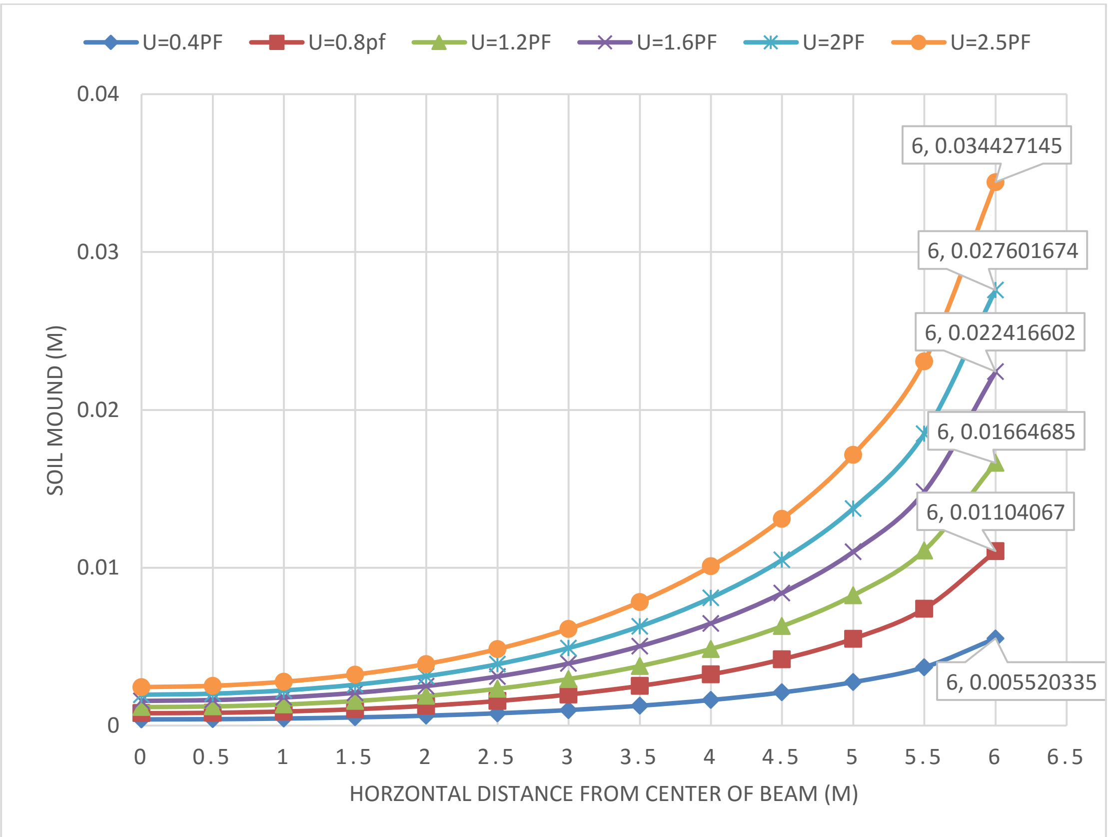


Fig. 4.18 Mound shape with horizontal distance at Different Change of edge suction

g) Drying period

The effects of the drying period in predicting the soil mound using equation (4.8) are undertaken for a constant diffusitive parameter condition on the surface of the subgrade and change surface edge suction. The drying period was changed from a small value 1month and to a large value 1 year. Table 4.9 and Fig. 4.19 give the soil mound envelopes showing the effect of drying period. Edge moisture distances vary from 1.5m to 2m and maximum soil mound vary from 25mm to 35mm when the drying period increase from 6 month to 1 year.

Table 4.9 Mound shape vs horizontal distance from middle of beam at Different drying period

H=3m	α_{field}=0.00402 cm²/min		f=1	ΔU= 2.5PF	γ_h=0.035
x	1month	3month	6month	9month	1 year
0	0.000711	0.001226	0.001728	0.0021084	0.002435
0.5	0.000736	0.001268	0.001787	0.0021813	0.00252
1	0.000811	0.001398	0.001971	0.0024051	0.002778
1.5	0.000943	0.001625	0.00229	0.0027953	0.003229
2	0.00114	0.001965	0.002769	0.0033789	0.003903
2.5	0.001415	0.00244	0.003438	0.0041964	0.004847
3	0.001789	0.003084	0.004347	0.0053049	0.006127
3.5	0.002288	0.003944	0.005558	0.0067831	0.007834
4	0.00295	0.005084	0.007163	0.0087406	0.010094
4.5	0.003832	0.006602	0.009298	0.011341	0.013093
5	0.005046	0.008683	0.012209	0.0148749	0.017155
5.5	0.006913	0.011834	0.016548	0.0200747	0.02307
6	0.011555	0.019019	0.025687	0.0304701	0.034427

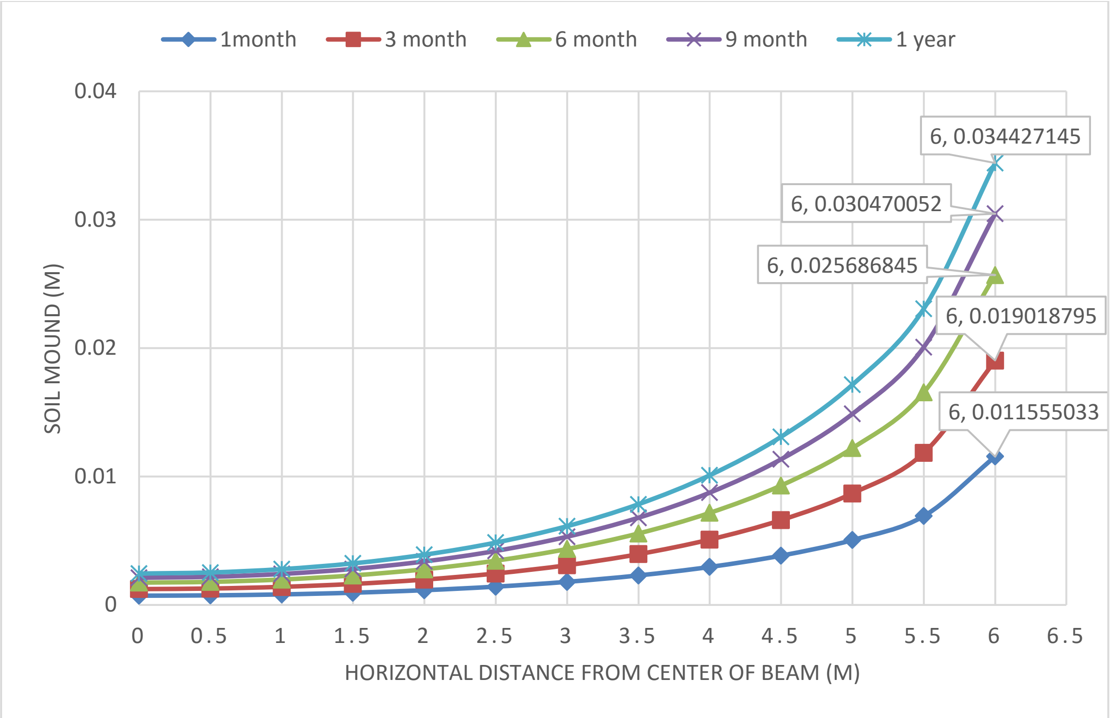


Fig. 4.19 Mound shape with horizontal distance at Different Drying period

Table 4.10 Summary of edge moisture distance and soil mound due to different parameters

Parameters	Increasing		Edge moisture distance Increasing		Soil mound Increasing	
	From	To	From	To	From	To
Change of surface edge suction	0.4PF	2.5PF	0	2m	5mm	35mm
Drying period	6 month	1 year	1.5m	2m	25mm	35mm
Diffusivity coefficient	0.001 cm2/min	0.1 cm2/min	1.5m	6m	24mm	138mm
Active depth	1m	3m	1m	3m	33mm	44mm

The above table indicate that diffusitive coefficient has more sensitivity to edge moisture distance and soil mound than the others parameters. The diffusitive coefficient is high when the soil is cracked or loos. The moisture easily migrate in the soil because of this the effect of moisture for soil heave is high. Therefor to predict the soil heave and effect on structure diffusive coefficient are important parameters.

Influence of slab dimension

The effects of slab dimension in predicting the soil mound shape profile in horizontal direction using equation (4.8) are undertaken for a constant suction boundary condition (ΔU=2.5PF) on the surface of the subgrade during a drying period of 1 year. The slab dimension parameter was changed from a small value 8x8m to a large value 20x20m. Table 4.11 and Fig. 4.20 gives the normalized soil mound envelopes showing the effects of slab dimension by using normalized equation (4.9) and (4.10). When the slab dimension small the normalized soil heave profile is flat this means the differential soil heave and edge moisture distance are small. The curvature of the normalized soil heave profile are increase when the slab dimension increase, this means the differential soil heave and edge moisture distance increase when the slab dimension increase, because of this the swelling pressure on the slab are also increase.

$$X_N = \frac{x}{L/2} \tag{4.9}$$

$$\Delta H_N = \frac{\Delta H(x) - \Delta H_{center}}{\Delta H_{edge} - \Delta H_{center}} \tag{4.10}$$

Where:

L = Slab dimension

x = Horizontal distance from center of slab

$\Delta H(x)$ = Soil heave at x distance from center of slab

ΔH_{center} = Soil heave at center of slab

ΔH_{edge} = Soil heave at edge of slab

Table 4.11 Normalized soil heave shape

	α=0.00402 cm2/min	f=1	γh=0.096	T= 1year	H= 3m	ΔU=2.5PF	
X_N	Slab dimension (8x8m) ΔH_N	Slab dimension (10x10m) ΔH_N	Slab dimension (12x12m) ΔH_N	Slab dimension (14x14m) ΔH_N	Slab dimension (16x16m) ΔH_N	Slab dimension (18x18m) ΔH_N	Slab dimension (20x20m) ΔH_N
0	0	0	0	0	0	0	0
0.075	0.0030909	0.0029874	0.0026329	0.0021845	0.0017348	0.001332	0.0007836
0.15	0.0124418	0.0120818	0.0107139	0.0089563	0.0071758	0.0055661	0.0032746
0.225	0.0282907	0.0276867	0.0248023	0.0209915	0.0170646	0.0134588	0.0079178
0.3	0.0510451	0.0504959	0.0458737	0.0394923	0.0327494	0.0264201	0.015543
0.375	0.0813011	0.0815304	0.0753907	0.0663077	0.0563689	0.046766	0.0275124
0.45	0.1198795	0.1221989	0.1154126	0.1041232	0.091146	0.0781328	0.0459646
0.525	0.1678927	0.1744064	0.168774	0.1567522	0.1418415	0.1261347	0.0741972
0.6	0.2268784	0.2407713	0.2394135	0.2296313	0.2154919	0.1994317	0.1172601
0.7	0.3266483	0.325137	0.3331523	0.3309239	0.3229331	0.3118074	0.1828964
0.8	0.4598429	0.4339764	0.4601305	0.4752412	0.4840525	0.4892208	0.2832363
0.9	0.6511773	0.6272396	0.6450444	0.6551765	0.6610342	0.6644532	0.4922633
1	1	1	1	1	1	1	1

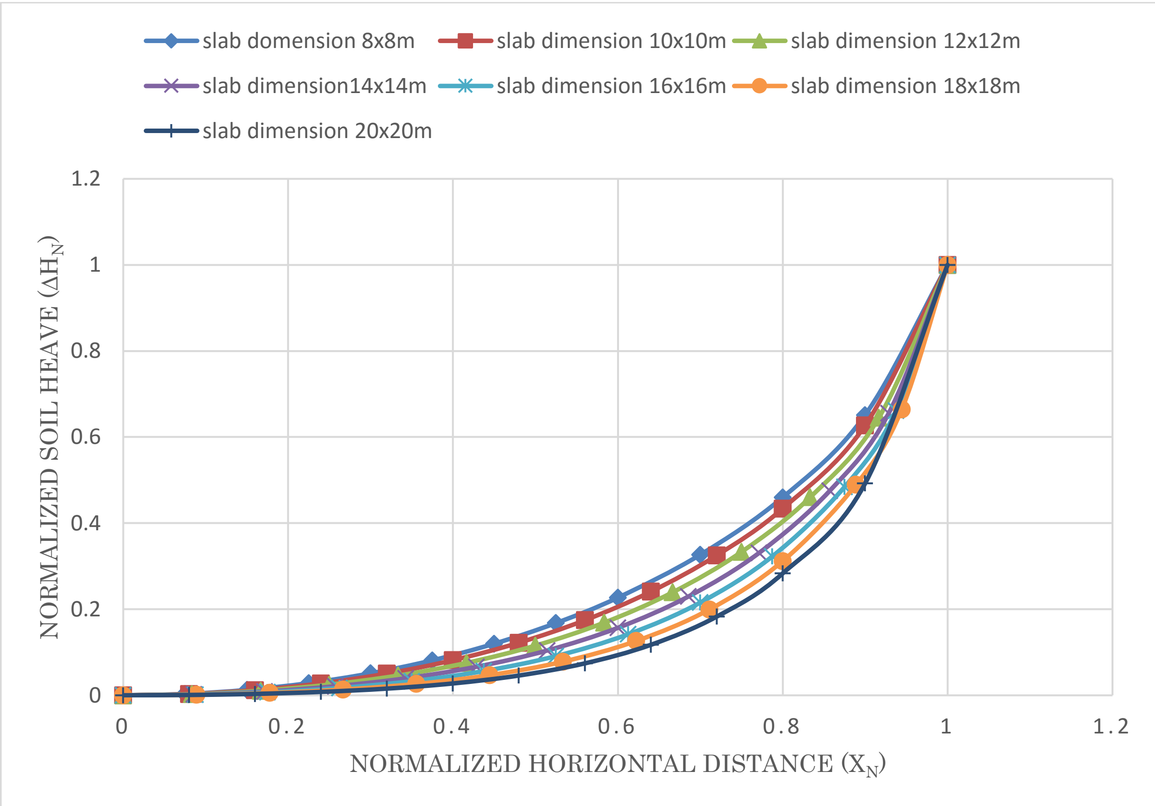


Fig. 4.20 Normalized soil heave shape

By take edge moisture distance and differential soil heave values from appendix A (table A17) prepare Table 4.12. Fig 4.21 and Fig 4.22 shows the relationship between slab dimension vs edge moisture distance and slab dimension vs differential soil heave respectively. A second order curve fitting shows strong correlation between slab dimension vs edge moisture distance with a coefficient of determination, $R^2 = 0.9999$ as shows in Fig 4.21 and slab dimension vs differential soil heave with a coefficient of determination, $R^2 = 0.9904$ as show in Fig 4.22.

Table 4.12 Slab dimension vs Edge moisture distance and Slab dimension vs Differential soil heave

Slab dimension (m)	Edge moisture distance (m)	Differential soil heave (cm)
8x8m	2.13	6.57
10x18m	2.6	7.33
12x12m	3	7.79
14x14m	3.33	8.06
16x16m	3.57	8.23
18x18m	3.72	8.32
20x20m	3.8	8.38

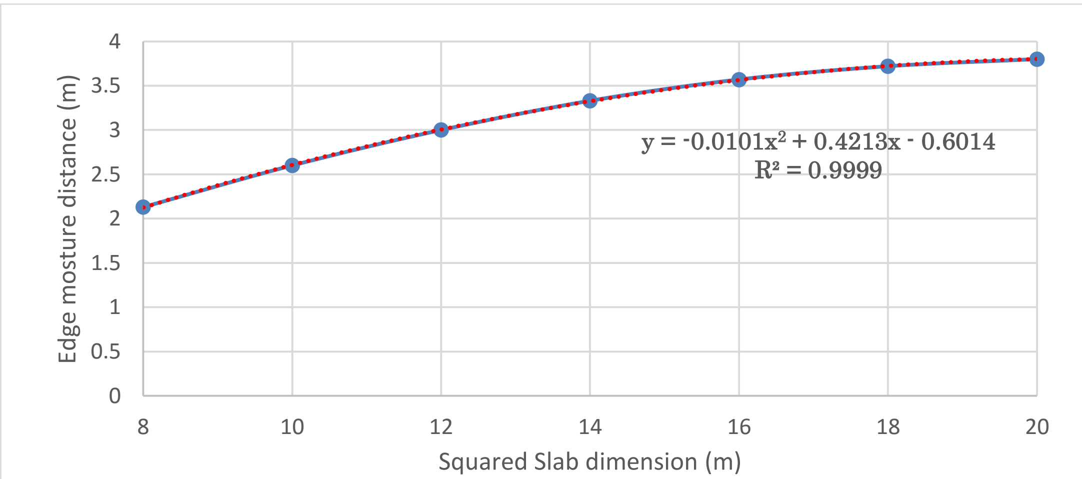


Fig. 4.21 Slab dimension vs Edge moisture distance

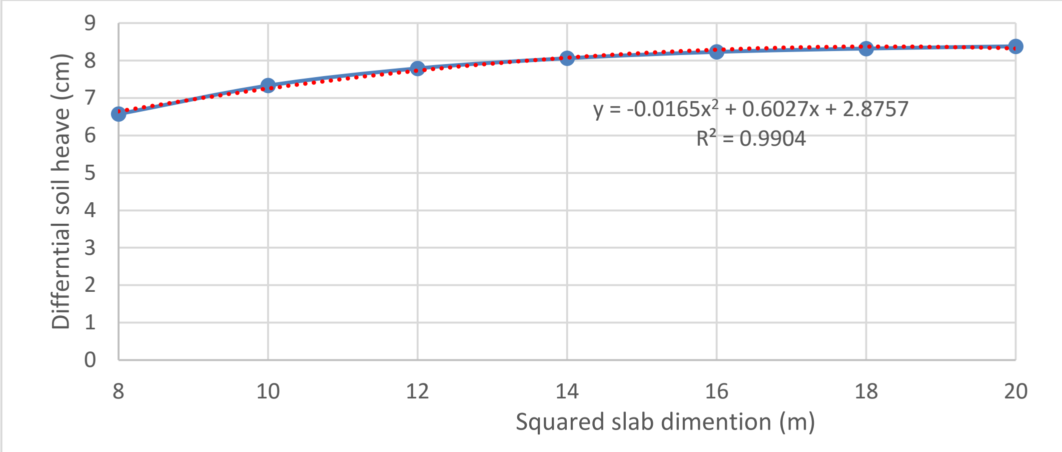


Fig.4.22 Slab dimension and Differential soil heave

4.2.2 Parametric Sensitivity for T-Beam Structural

Heave case

Waffle mat foundation structure analyses are assume as T- beams resting on expansive soil and they are simply supported beam as show in Fig. 3.7. There are four load on beam P_1 = 66.67kN at outer edge of beam, P_2 = 75kN at 2m distance from center of beam, distribution load on entire beam length its value depends on beam spacing and linearly decreased distributed up-ward swelling pressure started from outer edge beam to 2.5m distance from the edge of beam and analyses by use superposition. However, assumed on slab area load w=10kN/m^2 distributed load transfer to T-beam when consider waffle mat foundation as interconnected T-beam. Use equation (4.11) to cal-culate distributed area load on each interconnected T-beam and 400 kPa swelling pressure at the edge of beam. The swelling pressure on the beam depends on the beam width.

A parametric study was undertaken to evaluate the above load system by us-ing hetenyi (1946) formula superposition for each load case to predicting beam deflection profiles on expansive soil surface for heave case.

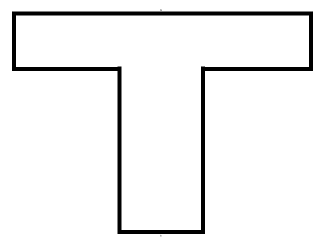


Fig. 4.23 Section view of cantilever T-beam

$$w_{equivalent} = \frac{ws}{2} \qquad (4.11)$$

Where: $w_{equivalent}$ = imposed line load on T- beam

W= imposed area load on waffle mat foundation

S= beam spacing transvers direction of T-beam

To evaluate structural parameter the above load system case by using superposition each below beam load type.

Case 1

Evaluate deflection of concentrated load at edge of T-beam by following equation (4.12).

$$y = \frac{2p\lambda}{k} \frac{\mathrm{Sinh}(\lambda l)\cos(\lambda x)\mathrm{Cosh}(\lambda x') - \sin(\lambda l)\mathrm{Cos}(\lambda x)\cos(\lambda x')}{\mathrm{Sinh}^2(\lambda l) - \sin^2(\lambda l)} \qquad (4.12)$$

$$M = -\frac{P}{\lambda} \frac{\mathrm{Sinh}(\lambda l)\mathrm{Sin}(\lambda x)\mathrm{Sinh}(\lambda x') - \mathrm{Sin}(\lambda l)\mathrm{Sinh}(\lambda x)\mathrm{Sin}(\lambda x')}{\mathrm{Sinh}^2(\lambda l) - \mathrm{Sin}^2(\lambda l)} \qquad (4.13)$$

$$Q = \frac{-P}{\mathrm{Sinh}^2(\lambda l) - \mathrm{Sin}^2(\lambda l)} * \begin{bmatrix} \mathrm{Sinh}(\lambda l)(\mathrm{Cos}(\lambda x)\mathrm{Sinh}(\lambda x') - \mathrm{Sin}(\lambda x)\mathrm{Cosh}(\lambda x')) \\ -(\mathrm{Sin}(\lambda l)(\mathrm{Cosh}(\lambda x)\mathrm{Sin}(\lambda x') - \mathrm{Sinh}(\lambda x)\mathrm{Cos}(\lambda x')) \end{bmatrix} \qquad (4.14)$$

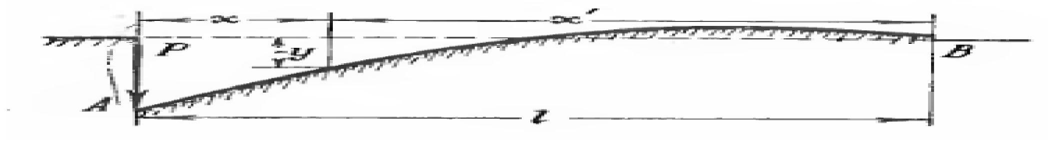


Fig. 4.24 simply supported beam with concentrated load at one end.

M. Hetenyi (1946)

Case 2

Evaluate deflection of concentrated load at 2m from center of T-beam by following equation (4.15).

$$y_3 = \frac{P\lambda}{k}\frac{1}{\sinh^2(\lambda l)-\sin^2(\lambda l)}\left\{\begin{array}{l} 2\operatorname{Cosh}(\lambda x)\cos(\lambda x)\begin{pmatrix}\operatorname{Sinh}(\lambda l)\cos(\lambda a)\operatorname{Cosh}(\lambda b)\\ -\sin(\lambda l)\operatorname{Cosh}(\lambda a)\cos(\lambda b)\end{pmatrix} \\ +\begin{pmatrix}\operatorname{Cosh}(\lambda x)\sin(\lambda x)\\ +\operatorname{Sinh}(\lambda x)\cos(\lambda x)\end{pmatrix}\left[\begin{array}{l}\operatorname{Sinh}(\lambda l)\begin{pmatrix}\sin(\lambda a)\operatorname{Cosh}(\lambda b)\\ -\cos(\lambda a)\operatorname{Sinh}(\lambda b)\end{pmatrix}\\ +\sin(\lambda l)\begin{pmatrix}\operatorname{Sinh}(\lambda a)\cos(\lambda b)\\ -\operatorname{Cosh}(\lambda a)\sin(\lambda b)\end{pmatrix}\end{array}\right]\end{array}\right\} \tag{4.15}$$

$$M = \frac{P}{2\lambda}\frac{1}{\operatorname{Sinh}^2(\lambda l)-\operatorname{Sin}^2(\lambda l)} * \left\{\begin{array}{l} 2\operatorname{Sinh}(\lambda x)\operatorname{Sin}(\lambda x)\begin{bmatrix}\operatorname{Sinh}(\lambda l)\operatorname{Cos}(\lambda a)\operatorname{Cosh}(\lambda b)\\ -\operatorname{Sin}(\lambda l)\operatorname{Cosh}(\lambda a)\operatorname{Cos}(\lambda b)\end{bmatrix} \\ +\left(\operatorname{Cosh}(\lambda x)\operatorname{Sin}(\lambda x)-\operatorname{Sinh}(\lambda x)\operatorname{Cos}(\lambda x)\right) \\ *\begin{bmatrix}\operatorname{Sin}(\lambda l)(\operatorname{Sin}(\lambda a)\operatorname{Cosh}(\lambda b)-\operatorname{Cos}(\lambda a)\operatorname{Sinh}(\lambda b))\\ +\operatorname{Sin}(\lambda l)(\operatorname{Sinh}(\lambda a)\operatorname{Cos}(\lambda b)-\operatorname{Cosh}(\lambda a)\operatorname{Sin}(\lambda b))\end{bmatrix}\end{array}\right\} \tag{4.16}$$

$$Q = \frac{P}{\operatorname{Sinh}^2(\lambda l)-\operatorname{Sin}^2(\lambda l)} * \left\{\begin{array}{l} (\operatorname{Cosh}(\lambda x)\operatorname{Sin}(\lambda x)+\operatorname{Sinh}(\lambda x)\operatorname{Cos}(\lambda x)) \\ *(\operatorname{Sinh}(\lambda l)\operatorname{Cos}(\lambda a)\operatorname{Cosh}(\lambda b)-\operatorname{Sin}(\lambda l)\operatorname{Cosh}(\lambda a)\operatorname{Cos}(\lambda b)) \\ +\operatorname{Sinh}(\lambda x)\operatorname{Sin}(\lambda x)\begin{bmatrix}\operatorname{Sinh}(\lambda l)(\operatorname{Sin}(\lambda a)\operatorname{Cosh}(\lambda b)-\operatorname{Cos}(\lambda a)\operatorname{Sinh}(\lambda b))\\ +\operatorname{Sin}(\lambda l)(\operatorname{Sinh}(\lambda a)\operatorname{Cos}(\lambda b)-\operatorname{Cosh}(\lambda a)\operatorname{Sin}(\lambda b))\end{bmatrix}\end{array}\right\} \tag{4.17}$$

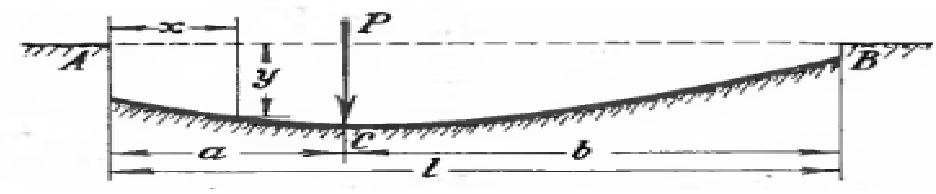


Fig. 4.25 simply supported beam with concentrated load at arbitrary point.
M. Hetenyi (1946)

Case 3

Evaluate deflection of uniformly distributed load of T-beam by following equation (4.18).

$$y_2 = \frac{q}{k}\left(1 - \frac{\mathrm{Cosh}(\lambda x)\cos(\lambda x') + \mathrm{Cos}(\lambda x')\cos(\lambda x)}{\mathrm{Cosh}(\lambda l) + \cos(\lambda l)}\right) \quad (4.18)$$

$$M = \frac{q}{2\lambda^2}\frac{\mathrm{Sinh}(\lambda x)\mathrm{Sin}(\lambda x') + \mathrm{Sin}(\lambda x')\mathrm{Sin}(\lambda x)}{\mathrm{Cosh}(\lambda l) + \mathrm{Cos}(\lambda l)} \quad (4.19)$$

$$Q = -\frac{q}{2\lambda}\frac{1}{\mathrm{Cosh}(\lambda l) + \mathrm{Cos}(\lambda l)}\begin{bmatrix}\mathrm{Sinh}(\lambda x)\mathrm{Cos}(\lambda x') - \mathrm{Cosh}(\lambda x)\mathrm{Sin}(\lambda x') \\ +\mathrm{Cosh}(\lambda x')\mathrm{Sin}(\lambda x) - \mathrm{Sinh}(\lambda x')\mathrm{Cos}(\lambda x)\end{bmatrix} \quad (4.20)$$

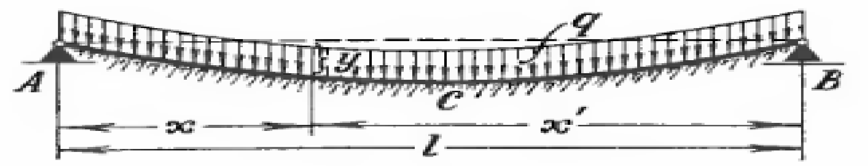


Fig. 4.26 simply supported beam with uniformly distributed load. M. Hetenyi (1946)

Case 4

Evaluate deflection of linearly distributed heave load of T-beam that comes from heave of soil. Using case 2 load system by changing linearly distributed load to concentrated load at 0.83 m from edge of beam and the load is negative because the load direction is up ward. This heave load varies that depends on the beam width that contact with soil.

a) Flange width.

The effects of the beam space parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant flange thickness 0.1m, beam depth 0.3m and beam width 0.3m. Assume the modulus of concrete 29GPa and length of beam span 12m. The beam spacing parameter was changed from a small value 0.75m and to a large value 2m. Table 4.13 and Fig. 4.27 give the beam deflection envelopes showing the effects of beam space. The maximum deflection of the beam found at edge of beam. However, beam space increased the deflection are almost the same so the beam space has less influence on the deflection of the beam.

Table 4.13 beam deflection (m) vs X (m) at Different beam space for soil swell.

x	S=0.75m	S=1m	S=1.25m	S=1.5m	S=1.75m	S=2m
0	0.00447391	0.0044952	0.00450787	0.004516227	0.004522	0.004526558
0.25	0.00425204	0.004245374	0.004236609	0.004227057	0.004217	0.004207226
0.5	0.00401428	0.003981675	0.003952865	0.003926452	0.003902	0.003878004
0.83	0.00364396	0.003584084	0.003533577	0.003488467	0.003447	0.003407429
1	0.00341545	0.00334612	0.003287564	0.003235151	0.003187	0.003140697
1.25	0.00303745	0.00295959	0.002892902	0.002832548	0.002776	0.002722501
1.5	0.00262327	0.002541995	0.002470794	0.002405299	0.002343	0.002283942
1.75	0.00218623	0.002105435	0.002032579	0.001964231	0.001899	0.001835223
2	0.00173765	0.00166021	0.001587912	0.001518564	0.001451	0.001385031
2.25	0.00128716	0.001215085	0.001145013	0.001076149	0.001008	0.000940755
2.5	0.00084299	0.000777559	0.000710916	0.000643699	0.000576	0.000508706
2.75	0.00041227	0.000354135	0.000291715	0.000227024	0.000161	9.43315E-05
3	1.37E-06	4.94119E-05	-0.00010719	-0.000168739	-0.00023	-0.00029758
3.25	6.2479E-05	-2.06276E-05	-3.10726E-05	-8.81217E-05	-0.00015	-0.00021052
3.5	-0.0003295	-0.000362594	-0.000407924	-0.000459925	-0.00052	-0.0005744
3.75	-0.0006811	-0.000707437	-0.000747314	-0.000794608	-0.00085	-0.0009009
4	-0.0009858	-0.001007962	-0.001043779	-0.001087055	-0.00113	-0.0011854
4.25	-0.0012394	-0.001260595	-0.00129408	-0.001334275	-0.00138	-0.00142526
4.5	-0.0014488	-0.001471559	-0.001504065	-0.001541866	-0.00158	-0.00162578
4.75	-0.001623	-0.001649108	-0.001681497	-0.001717264	-0.00176	-0.00179402
5	-0.0017703	-0.001800856	-0.001833586	-0.001867422	-0.0019	-0.00193664
5.17	-0.0019429	-0.001981793	-0.00201827	-0.002053257	-0.00209	-0.0021205
5.5	-0.0019311	-0.001966772	-0.001996929	-0.002023958	-0.00205	-0.00207277
5.75	-0.0021196	-0.002165658	-0.002199653	-0.002226851	-0.00225	-0.00226998
6	-0.0022229	-0.002274042	-0.002308245	-0.002332891	-0.00235	-0.00236643

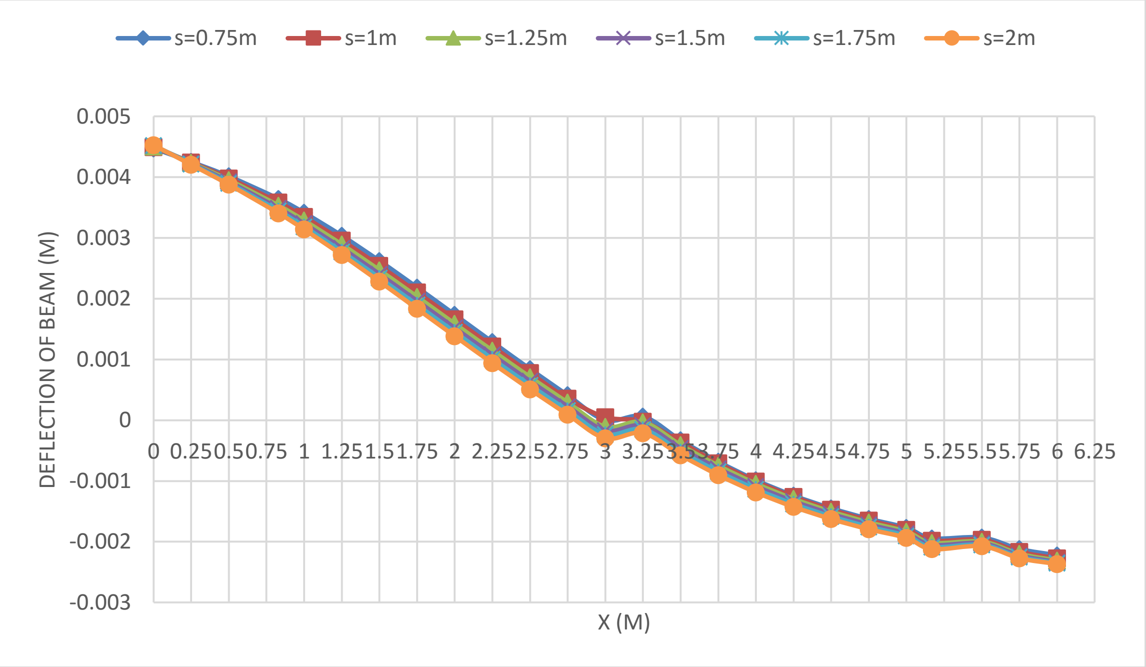


Fig. 4.27 Beam deflection (m) vs X (m) at Different beam space for soil swell.

b) Flange thickness

Considering flange thickness parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant beam spacing1.5m, rib depth 0.3m and beam width 0.3m. Assume modulus of concrete 29GPa and length of beam span 12m. The flange thickness parameter was changed from a small value 0.1m and to a large value 0.35m. Table 4.14 and Fig. 4.28 give the beam deflection envelopes showing the effects of flange thickness of T-beam. The maximum deflection of the beam found at edge of beam. When flange thickness increased the deflection of the beam almost the same. This shows that flange thickness has less influence on deflection of the beam.

Table 4.14 beam deflection (m) vs X (m) at Different flange thickness for soil swell.

x	t=0.1m	t=0.15m	t=0.2m	t=0.25m	t=0.3m	t=0.35m
0	0.004516227	0.004548725	0.00456143	0.004565586	0.004565943	0.004565
0.25	0.004227057	0.004233495	0.00423344	0.004231618	0.004229684	0.004228
0.5	0.003926452	0.003910439	0.0038999	0.003893642	0.003890474	0.003889
0.83	0.003488467	0.003455743	0.00343957	0.003432967	0.00343198	0.003434
1	0.003235151	0.0032025	0.00318892	0.003185859	0.003188557	0.003194
1.25	0.002832548	0.002809667	0.00280599	0.002812175	0.00282306	0.002835
1.5	0.002405299	0.002400547	0.00241192	0.002430718	0.002452026	0.002473
1.75	0.001964231	0.00198295	0.00201252	0.002045871	0.00207881	0.002109
2	0.001518564	0.001563565	0.00161278	0.001661409	0.001706306	0.001746
2.25	0.001076149	0.00114811	0.00121698	0.001280577	0.001337004	0.001385
2.5	0.000643699	0.000741491	0.00082878	0.000906162	0.000973042	0.001029
2.75	0.000227024	0.000347954	0.00045137	0.000540575	0.000616269	0.000679
3	-0.000168739	-2.87611E-05	8.7528E-05	0.000185918	0.000268292	0.000336
3.25	-8.81217E-05	7.06544E-05	0.00019934	0.000306341	0.000394807	0.000467
3.5	-0.000459925	-0.000286494	-0.0001484	-3.50956E-05	5.76572E-05	0.000133
3.75	-0.000794608	-0.000615263	-0.0004741	-0.000359295	-0.00026594	-0.00019
4	-0.001087055	-0.000912092	-0.0007752	-0.00066439	-0.00057458	-0.0005
4.25	-0.001334275	-0.001174976	-0.0010503	-0.000949394	-0.00086758	-0.0008
4.5	-0.001541866	-0.001407995	-0.0013025	-0.001216632	-0.0011467	-0.00109
4.75	-0.001717264	-0.001616576	-0.0015359	-0.001469198	-0.00141433	-0.00137
5	-0.001867422	-0.001805856	-0.0017542	-0.001710065	-0.00167278	-0.00164
5.17	-0.002053257	-0.002034211	-0.0020141	-0.00199421	-0.00197565	-0.00196
5.5	-0.002023958	-0.002038456	-0.0020433	-0.002043031	-0.00204012	-0.00204
5.75	-0.002226851	-0.002303332	-0.0023542	-0.002389415	-0.00241397	-0.00243
6	-0.002332891	-0.002458405	-0.0025458	-0.002609167	-0.00265573	-0.00269

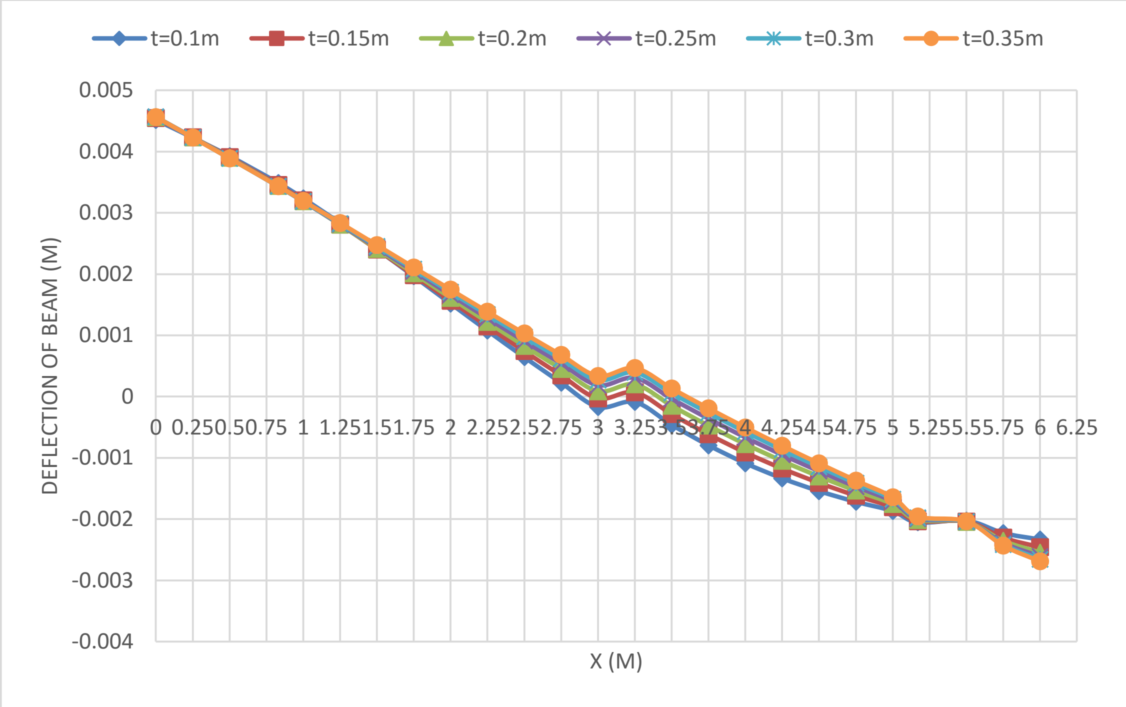


Fig.4.28 Beam deflection (m) vs X(m) at Different flange thickness for soil swell.

c) Beam width

Considering beam width parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant beam spacing1.5m, beam depth 0.3m and flange thickness 0.1m. Assume the modulus of concrete 29GPa and length of beam span 12m. The beam width parameter was changed from a small value 0.15m and to a large value 0.45m. Table 4.15 and Fig. 4.29 give the beam deflection envelopes showing the effects of beam width. The maximum deflection of the beam found at the edge of the beam. However, beam width increased the beam deflection also increase at the edge of the beam. So it has higher influence on the deflection of the beam because when increase the beam width also increase the swelling pressure of the soil.

Table 4.15 beam deflection (m) vs X (m) at Different beam width for soil swell.

x	b=0.15m	b=0.2m	b=0.25m	b=0.3m	b=0.35m	b=0.4m	b=0.45m
0	5.87668E-05	0.002266	0.003609	0.004516227	0.005174	0.005674188	0.006069
0.25	3.73127E-05	0.002114	0.003376	0.004227057	0.004843	0.005311612	0.005681
0.5	-5.51E-07	0.001948	0.00313	0.003926452	0.004502	0.004938772	0.005283
0.83	-0.00010441	0.001682	0.002762	0.003488467	0.004012	0.004407872	0.004719
1	-0.00019232	0.001514	0.002544	0.003235151	0.003732	0.004107704	0.004402
1.25	-0.00036292	0.001231	0.002191	0.002832548	0.003292	0.003638579	0.003909
1.5	-0.00057481	0.000916	0.00181	0.002405299	0.00283	0.003149159	0.003397
1.75	-0.00081945	0.000576	0.001411	0.001964231	0.002358	0.002651453	0.002879
2	-0.00108863	0.000222	0.001002	0.001518564	0.001884	0.002155358	0.002364
2.25	-0.00137429	-0.00014	0.000593	0.001076149	0.001417	0.001668948	0.001862
2.5	-0.00166847	-0.0005	0.000189	0.000643699	0.000963	0.001198765	0.001379
2.75	-0.00196314	-0.00086	-0.0002	0.000227024	0.000528	0.000750104	0.000919
3	-0.00225008	-0.0012	-0.00058	-0.000168739	-0.00012	0.00032728	0.000487
3.25	-0.00206706	-0.00107	-0.00048	-8.81217E-05	0.000184	0.000383052	0.000535
3.5	-0.00233988	-0.00139	-0.00083	-0.000459925	-0.0002	-1.17905E-05	0.000132
3.75	-0.00257571	-0.00168	-0.00114	-0.000794608	-0.00055	-0.00036823	-0.00023
4	-0.00276385	-0.00192	-0.00142	-0.001087055	-0.00085	-0.000682769	-0.00055
4.25	-0.00289668	-0.00211	-0.00164	-0.001334275	-0.00112	-0.000953797	-0.00083
4.5	-0.00298049	-0.00226	-0.00183	-0.001541866	-0.00134	-0.001186881	-0.00107
4.75	-0.00302452	-0.00237	-0.00198	-0.001717264	-0.00153	-0.001389132	-0.00128
5	-0.00303725	-0.00246	-0.0021	-0.001867422	-0.0017	-0.001567245	-0.00147
5.17	-0.00313436	-0.0026	-0.00227	-0.002053257	-0.00189	-0.001768542	-0.00167
5.5	-0.00289815	-0.00247	-0.00221	-0.002023958	-0.00189	-0.001786354	-0.0017
5.75	-0.00296072	-0.00261	-0.00238	-0.002226851	-0.00211	-0.002015067	-0.00194
6	-0.00291747	-0.00264	-0.00246	-0.002332891	-0.00223	-0.002151371	-0.00208

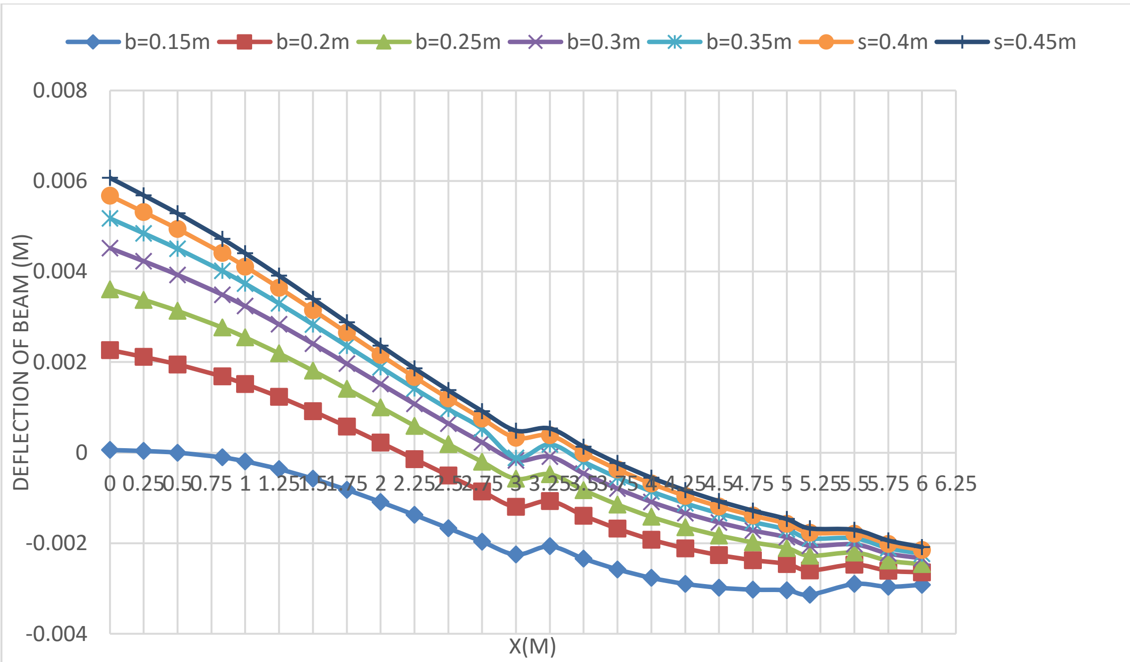


Fig. 4.29 Beam deflection (m) vs. X (m) at Different beam width for soil swell

d) Beam depth

The effects of the beam depth parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant flange thickness 0.1m, beam spacing 1.5m and beam width 0.3m. Assume the modulus of concrete 29GPa and length of beam span 12m. The beam depth parameter was changed from a small value 0.2m and to a large value 0.5m. Table 4.16 and Fig. 4.30 give the beam deflection envelopes showing the effects of beam depth. The maximum deflection of the beam found at edge of beam. When beam depth increased the deflection of beam almost the same. So the beam depth has less influence than beam width, but higher influence the beam spacing and flange thickness on the deflection of the beam.

Table 4.16 beam deflection (m) vs X (m) at Different beam depth for soil swell.

x	h=0.2m	h=0.25m	h=0.3m	h=0.35m	h=0.4m	h=0.45m	h=0.5m
0	0.0042499	0.004438	0.0045162	0.0045483	0.004561	0.0045653	0.0045661
0.25	0.0041212	0.0042012	0.0042271	0.0042335	0.004234	0.0042319	0.0042302
0.5	0.0039642	0.0039469	0.0039265	0.0039107	0.0039	0.0038943	0.0038911
0.83	0.0036561	0.0035487	0.0034885	0.0034562	0.00344	0.0034335	0.0034319
1	0.0034296	0.0033015	0.0032352	0.0032029	0.003189	0.0031859	0.0031874
1.25	0.0030213	0.0028918	0.0028325	0.0028099	0.002806	0.0028109	0.0028198
1.5	0.0025488	0.0024433	0.0024053	0.0024004	0.002411	0.0024277	0.0024461
1.75	0.0020358	0.0019716	0.0019642	0.0019825	0.002011	0.002041	0.00207
2	0.0015023	0.0014899	0.0015186	0.0015627	0.00161	0.0016545	0.0016945
2.25	0.000965	0.0010094	0.0010761	0.0011468	0.001213	0.0012717	0.0013223
2.5	0.0004376	0.0005395	0.0006437	0.0007397	0.000824	0.0008955	0.0009557
2.75	-6.778E-05	8.834E-05	0.000227	0.0003458	0.000446	0.0005285	0.0005968
3	-0.0005406	-0.000337	-0.0001687	-3.117E-05	8.11E-05	0.0001726	0.0002472
3.25	-0.0005386	-0.000286	-8.812E-05	-6.796E-05	0.000192	0.000292	0.0003722
3.5	-0.000973	-0.000681	-0.0004599	-0.0002894	-0.000156	-5.022E-05	3.403E-05
3.75	-0.0013394	-0.001027	-0.0007946	-0.0006183	-0.000482	-0.0003746	-0.00029
4	-0.0016259	-0.001315	-0.0010871	-0.000915	-0.000783	-0.0006791	-0.000597
4.25	-0.0018248	-0.001542	-0.0013343	-0.0011776	-0.001057	-0.0009628	-0.000888
4.5	-0.0019482	-0.001715	-0.0015419	-0.0014102	-0.001308	-0.0012281	-0.001164
4.75	-0.0020119	-0.001845	-0.0017173	-0.0016183	-0.00154	-0.0014781	-0.001428
5	-0.0020299	-0.001941	-0.0018674	-0.0018069	-0.001757	-0.001716	-0.001682
5.17	-0.0020761	-0.002069	-0.0020533	-0.0020346	-0.002015	-0.001997	-0.001981
5.5	-0.0019261	-0.001992	-0.002024	-0.0020383	-0.002043	-0.0020433	-0.002041
5.75	-0.0019251	-0.00211	-0.0022269	-0.0023022	-0.002352	-0.0023851	-0.002408
6	-0.001867	-0.002148	-0.0023329	-0.0024564	-0.002541	-0.0026011	-0.002644

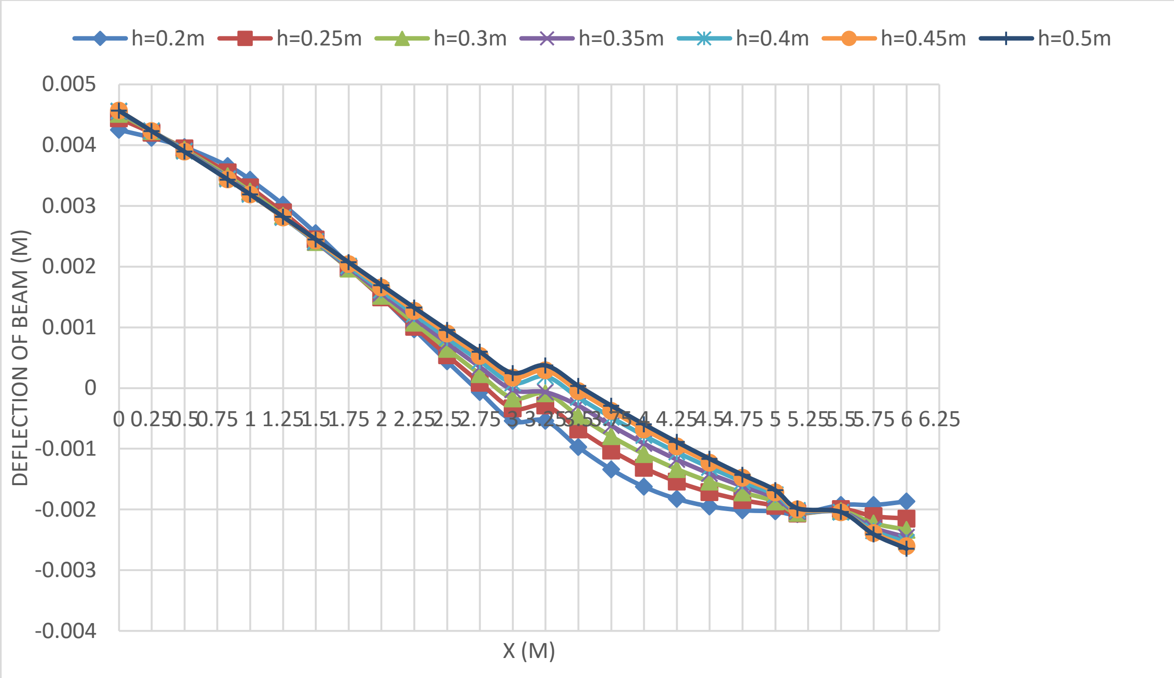


Fig. 4.30 Beam deflection (m) vs X(m) at Different beam depth for soil swell

Generally the maximum beam deflection of the beam are small for all beam parameters so swelling pressure is not problem to structure because the beam has small contact area with beam and heaved soil dissipated in to void under the slab

4.3 Design example

To design moment, shear and deflection of waffle mat foundation assume T-beam on elastic foundation.

Data

Used secondary data form (Building Foundation Characterization and Analysis: A Case Study in Ayat Area, Bole Sub-City, Addis Ababa, Central Ethiopia. Selamawit Taddese, 2017, AAIT) Thesis.

Thickness of soil layer (H) =3m

Cohesion(C) = 45kPa

Angle of internal friction (Ø) =15

Unit Weight = 19kN/m^3

Liquid limit (LL) = 63%

Plastic Index (PI) = 27%

%-2micron = 42%

Moisture content = 44%

Sieve 200 pass = 92%

Compute bearing capacity of a soil by using Terzaghi (1943).

N_c =12.9 N_q=4.4 N_γ =2.5

$$q_{ult(0.5m)} = CN_c + qN_q + 0.5B\gamma N_\gamma$$

$$q_{ult(0.5m)} = (45*12.9) + (0.5*19*4.4) + (0.5*19*1*2.5) = 645.05\text{KPa}$$

$$q_{allw} = \frac{q_{ult(0.5)}}{F.S} = \frac{645.05}{3} = 215.01\text{KPa}$$

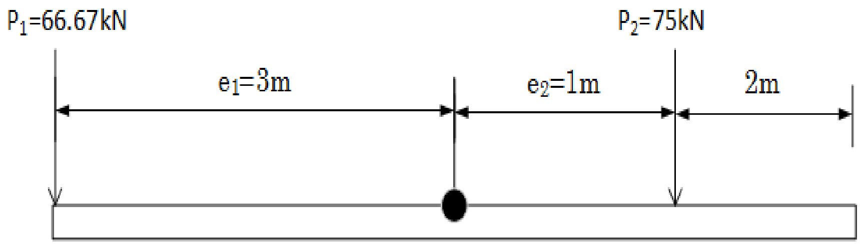


Fig. 4.31 Beam with two concentrated loads for design example

$\sum pe + p_2 * e_2 - p_1 * e_1 = 0 \rightarrow$ e = 0.8824m from the center of beam

$$\sigma = \frac{\sum p}{A}\left(1 + \frac{6e}{l}\right) + \frac{w}{b} = 148.15 + 25 = 173.15\text{KPa} < 215.01\text{KPa}$$

Therefor the bearing capacity of soil is enough to carrying the imposed load

Compute soil heave

First compute diffusion coefficient of soil

$$S = -20.29 + 0.1555(LL\%) - 0.117(PI\%) + 0.0684(\% - \#200) = -8.183$$

$$A_c = \frac{PI\%}{\left(\frac{\%-2micron}{\%-No200sieve}\right)*100} = 0.591$$

$$\text{CEC} = (LL\%)^{0.912} = 43.75$$

$$CEA_c = \frac{CEC}{\left(\frac{\%-2micron}{\%-No200sieve}\right)*100} = 0.98$$

By using Cation Exchange Activity (CEA_c) and Activity Ratio (A_C) determine suction compression index

Suction compression index number = lllB from figure (4.1)

The value of $\gamma_o = 0.096$ from table (4.1)

$$y_{h(swelling)} = y_h e^{\gamma h} = 0.1056$$

$$y_{h(shrinkage)} = y_h e^{-\gamma h} = 0.0872$$

$$\alpha = 0.0029 - 0.000162S - 0.0122\gamma_h$$

$$\alpha_{(swelling)} = 0.00302\ cm^2 / \min$$

$$\alpha_{(shrinkage)} = 0.00322\ \text{cm}^2/\text{min}$$

Second compute soil heave and edge moisture distance by using Abdulmalak (2007) proposed new mound shape equation.

Assume

$\Delta u = 1PF$ for shrinkage case $\quad \Delta u_{edge.shrinkage} = 0.5\Delta u_{shrinkage} = 0.5PF$
$\Delta \text{u} = 2.5PF$ for swelling case $\quad \Delta u_{edge.swelling} = 0.5\Delta u_{swelling} = 1.25PF$

$f = 1$ n = 1,3,5,7,9 L = 12m ω = 0.0000119 rad/min

$$\Delta H(x) = f\gamma_h H \Delta U_{edge} \left(\left[\frac{2\pi\left(n-\frac{1}{2}\right)(-1)^{n-1}\exp\left(-\sqrt{\frac{\omega H^2}{2\alpha_{field}}}+2\sqrt{\frac{\omega H^2}{2\alpha_{field}}}\right)}{\frac{\omega H^2}{2\alpha_{field}}+\pi^2\left(n-\frac{1}{2}\right)^2} \right] * \left[\frac{\cosh\left(\left(n-\frac{1}{2}\right)\pi\frac{x}{H}\right)(-1)^{n-1}}{\cosh\left(\left(n-\frac{1}{2}\right)\pi\frac{L}{2H}\right)\pi\left(n-\frac{1}{2}\right)} \right] \right)$$

Table 4.17 Soil swelling and shrinkage for design example

X(cm)	ΔH(Shrinkage) in(cm)	ΔH(swelling) in(cm)
0	0.109000152	0.639756307
50	0.112769982	0.661882637
100	0.124340337	0.729792729
150	0.14451206	0.848187036
200	0.174682397	1.02526663
250	0.21694554	1.273323704
300	0.27425197	1.609678236
350	0.350666507	2.058197891
400	0.451851766	2.652155829
450	0.586232797	3.441145889
500	0.768700304	4.513165937
550	1.036472538	6.089725753
600	1.566102568	9.234775737

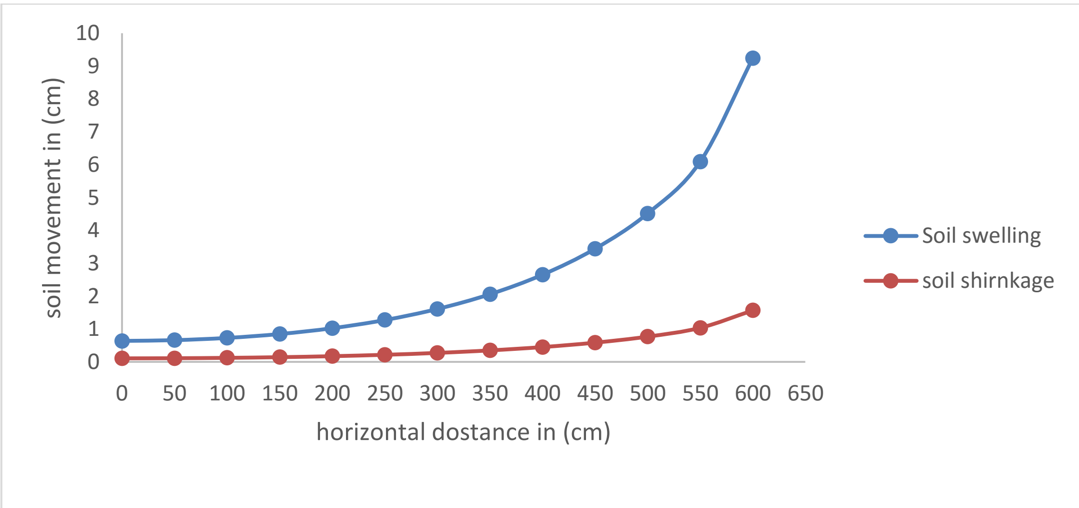


Fig. 4.32 Soil swelling and shrinkage for design example

Maximum shrinkage of soil = 0.0156m

Maximum swelling of soil = 0.0923m

Edge moisture distance for drying = 3m

Edge moisture distance for wetting = 3.5m

Now design interconnected T-beam and inverted L-beam

Beam Type 1

Slab thickness (h_f) = 0.1m Use C- 25 concrete f_{ck}= 20MPa

Rib depth = 0.5m f_{cd} = 11.33MPa

Web width of beam (b_w) = 0.3m S-300 steel f_{yk}= 300MPa

Flange width of beam (b_e) = 1.5m f_{yd} = 260.87MPa

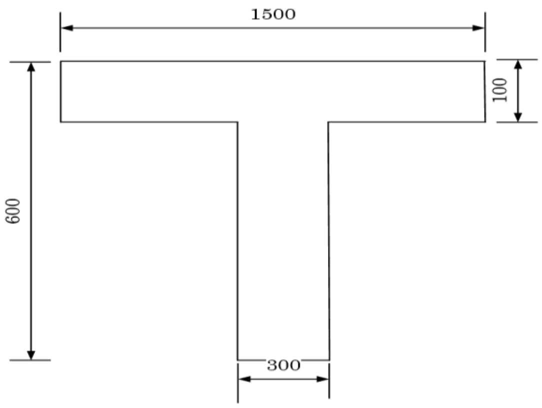


Fig. 4.33 Section view for design example beam type 1

- **Compute deflection of beam:**

Use equation A.1, A.4 and A.7 to determine maximum deflection of beam when the soil is shrink.

Deflection of beam for shrinkage = 0.007m < $\frac{L}{480} = \frac{6}{480} = 0.0125m$ OK.

Use equation (4.12), (4.15) and (4.18) to determine maximum deflection of the beam when the soil is swell.

Deflection of beam for swelling = 0.005m < 0.0125 OK.

If the deflection of beam greater than allowable deflection increase the beam section.

Maximum flange width (beam spacing) are least of:

e) $b_w + \frac{L}{5} = 300 + \frac{6000}{5} = 1500mm$

ii) Actual width of top slab extending between the center of adjacent span =1500mm

Therefore assume flange width (b_e) = 1500mm OK.

- **Design the beam for moment:**

Use equation (4.13), (4.16) and (4.19) to determine the maximum moment of the beam when the soil is swell.

Design for positive moment

$d = 600 - 50 - 8 - 10 = 532mm$

$M_u = 55.34KNm$

Assuming the N.A to fall in the flange

$$M_u = 0.8xb_e f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 6.5mm$$
$$x = 6.5mm < h_f = 100mm$$

This implies that the N.A falls within the flange

$$A_s = \frac{0.8xb_e f_{cd}}{f_{yd}} = 265.2mm^2$$

$$Number\ \text{of}\ \phi 20\text{mm rod} = \frac{265.2}{\frac{\pi * 20^2}{4}} = 1.66\ \text{bar}, \quad \text{provide}\ 2\phi 20\ \text{bars}$$

Use equation A.2, A.5 and A.8 to determine maximum moment of the beam when the soil is shrink.

Design for negative moment

$$d = 600 - 25 - 6 - 10 = 557mm$$

$$M_u = 233.75KNm$$

$$M_u = 0.8xb_w f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 175.91mm$$

$$A_s = \frac{0.8xb_w f_{cd}}{f_{yd}} = 1833.61\ mm^2$$

$$Number\ \text{of}\ \phi 20mm\ \text{rod} = \frac{1833.61}{\frac{\pi * 20^2}{4}} = 5.83, \quad \text{provide}\ 6\phi 20\ \ \text{bars}$$

- **Design the beam for shear**

$$v_d = 89.166KN$$

Compute ultimate shear resistance of concrete

$$v_{Rd} = 0.25 f_{cd} b_w d = 452.067KN > v_d = 89.166\text{KN} \quad \text{OK.}$$

Compute concrete shear carrying capacity

$$v_c = 0.25 f_{ctd} K_1 K_2 b_w d = 52.23KN < v_d = 89.166KN \qquad \text{K}_1 = 1.19 < 2, \quad \text{K}_2 = 1.068 > 1 \quad \text{OK.}$$

$$S = \frac{A_v f_{yd}}{0.4b_w} = 217.39mm \qquad\qquad S_{\max} \le \begin{Bmatrix} 0.5d = 266mm \\ 300mm \end{Bmatrix}$$

Use $\phi 8@210c/c$

$$v_s = \frac{A_v f_{yd} d}{S} = 66.087KN$$

$v_c + v_s = 118.317KN > v_d = 89.116KN$ OK.

Beam Type 2

Slab thickness (h_f) = 0.1m Use C- 25 concrete f_{ck}= 20MPa

Rib depth = 0.5m f_{cd} = 11.33MPa

Web width of beam (b_w) = 0.4m S-300 steel f_{yk}= 300MPa

Flange width of beam (b_e) = 1m f_{yd} = 260.87MPa

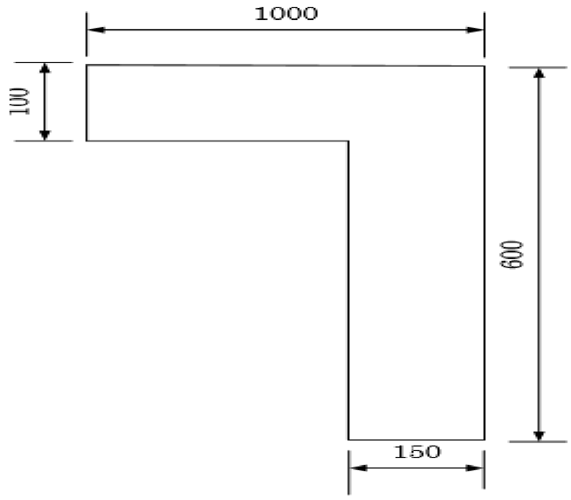


Fig. 4.34 Section view for design example beam type 2

- **Compute deflection of beam:**

Use equation A.1, A.4 and A.7 to determine maximum deflection of beam when the soil is shrink.

Deflection of beam for shrinkage = 0.007m < $\frac{L}{480} = \frac{6}{480} = 0.0125m$ OK.

Use equation (4.12), (4.15) and (4.18) to determine maximum deflection of the beam when the soil is swell.

Deflection of beam for swelling = 0.00364m < 0.0125 OK

Maximum flange width (beam spacing) are least of:

$i)\quad b_w + \frac{L}{10} = 400 + \frac{6000}{10} = 1000mm$

ii) Actual width of top slab extending between the center of adjacent span =1000mm

Therefore assume flange width (b_e) = 1000mm OK.

- **Design the beam for moment:**

Use equation (4.13), (4.16) and (4.19) to determine the maximum moment of the beam when the soil is swell.

Design for positive moment

$d = 600 - 50 - 8 - 10 = 532mm$

$M_u = 89.6KNm$

Assuming the N.A to fall in the flange

$$M_u = 0.8xb_e f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 18.77mm$$
$$x = 18.77mm < h_f = 100mm$$

This implies that the N.A falls within the flange

$$A_s = \frac{0.8xb_e f_{cd}}{f_{yd}} = 652.34mm^2$$

$$Number\ \text{of}\ \phi 20\text{mm rod} = \frac{652.34}{\frac{\pi * 20^2}{4}} = 2.074\ \text{bar}, \quad \text{provide } 3\phi 20 \text{ bars}$$

Use equation A.2, A.5 and A.8 to determine maximum moment of the beam when the soil is shrink.

Design for negative moment

$$d = 600 - 25 - 8 - 10 = 557mm$$

$$M_u = 333.75KNm$$

$$M_u = 0.8xb_w f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 190.7mm$$

$$A_s = \frac{0.8xb_w f_{cd}}{f_{yd}} = 2650.37\ mm^2$$

$$Number\ \text{of}\ \phi 20mm\ \text{rod} = \frac{2650.37}{\frac{\pi * 20^2}{4}} = 8.43, \quad \text{provide } 9\phi 20\ \text{ bars}$$

- **Design the beam for shear**

$$v_d = 122.727KN$$

Compute ultimate shear resistance of concrete

$$v_{Rd} = 0.25 f_{cd} b_w d = 602.75KN > v_d = 122.727\text{KN} \quad \text{OK.}$$

Compute concrete shear carrying capacity

$$v_c = 0.25 f_{ctd} K_1 K_2 b_w d = 67.124KN < v_d = 122.727KN \quad \text{K}_1 = 1.147 < 2, \quad \text{K}_2 = 1.068 > 1 \quad \text{OK.}$$

$$S = \frac{A_v f_{yd}}{0.4b_w} = 163.04mm \qquad S_{\max} \le \begin{Bmatrix} 0.5d = 266mm \\ 300mm \end{Bmatrix}$$

$$Use \quad \phi 8@160\text{c/c}$$

$$v_s = \frac{A_v f_{yd} d}{S} = 86.74KN$$

$v_c + v_s = 153.86KN > v_d = 122.727KN$ OK.

Beam Type 3

Slab thickness (h_f) = 0.1m Use C- 25 concrete f_{ck}= 20MPa

Rib depth = 0.5m f_{cd} = 11.33MPa

Web width of beam (b_w) = 0.15m S-300 steel f_{yk}= 300MPa

Flange width of beam (b_e) = 1.5m f_{yd} = 260.87MPa

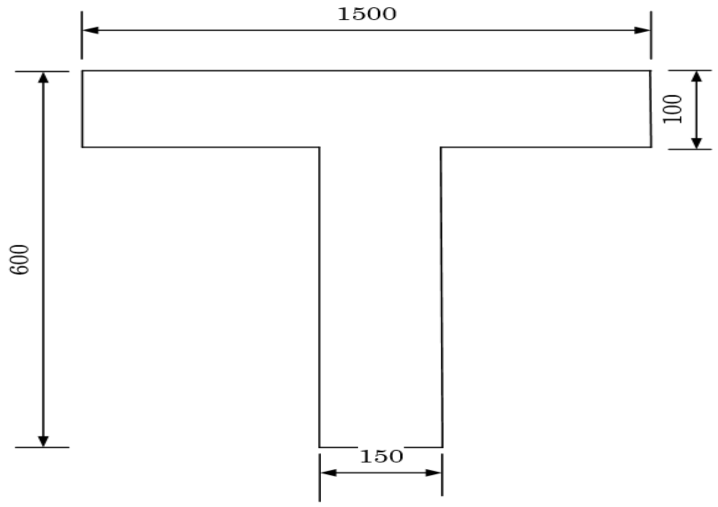


Fig. 4.35 Section view for design example beam type 3

- **Compute deflection of beam:**

Use equation A.1, A.4 and A.7 to determine maximum deflection of beam when the soil is shrink.

Deflection of beam for shrinkage = 0.00058m < $\frac{L}{480} = \frac{6}{480} = 0.0125m$ OK.

Use equation (4.12), (4.15) and (4.18) to determine maximum deflection of the beam when the soil is swell.

Deflection of beam for swelling = 0.009m < 0.0125 OK

Maximum flange width (beam spacing) are least of:

$$i)\ \ b_w + \frac{L}{10} = 150 + \frac{6000}{5} = 1500mm$$

ii) Actual width of top slab extending between the center of adjacent span =1500mm

Therefore assume flange width (b_e) = 1500mm OK.

- **Design the beam for moment:**

Use equation (4.13), (4.16) and (4.19) to determine the maximum moment of the beam when the soil is swell.

Design for positive moment

$$d = 600 - 50 - 8 - 7 = 535mm$$

$$M_u = 39.47KNm$$

Assuming the N.A to fall in the flange

$$M_u = 0.8xb_e f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 5.46mm$$
$$x = 5,46mm < h_f = 100mm$$

This implies that the N.A falls within the flange

$$A_s = \frac{0.8xb_e f_{cd}}{f_{yd}} = 284.56mm^2$$

$$Number\ \text{of}\ \phi 14\text{mm rod} = \frac{284.56}{\frac{\pi * 14^2}{4}} = 1.84\ \text{bar,} \quad \text{provide}\ 2\phi 14\text{bars}$$

Use equation A.2, A.5 and A.8 to determine maximum moment of the beam when the soil is shrink.

Design for negative moment

$d = 600 - 25 - 8 - 7 = 562mm$

$M_u = 33.75KNm$

$M_u = 0.8xb_w f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 40.115mm$

$$A_s = \frac{0.8xb_w f_{cd}}{f_{yd}} = 209.28\ mm^2$$

$$Number\ \text{of}\ \phi 14mm\ \text{rod} = \frac{209.28}{\frac{\pi * 14^2}{4}} = 1.36, \quad \text{provide } 2\phi 14 \text{ bars}$$

- **Design the beam for shear**

$v_d = 41.17KN$

Compute ultimate shear resistance of concrete

$v_{Rd} = 0.25 f_{cd} b_w d = 226.03KN > v_d = 41.17\text{KN} \quad \text{OK.}$

Compute concrete shear carrying capacity

$v_c = 0.25 f_{ctd} K_1 K_2 b_w d = 26.11KN < v_d = 41.17KN \quad \text{K}_1 = 1.19 < 2, \quad \text{K}_2 = 1.068 > 1 \quad \text{OK.}$

$$S = \frac{A_v f_{yd}}{0.4 b_w} = 434.18mm \qquad S_{\max} \leq \begin{Bmatrix} 0.5d = 266mm \\ 300mm \end{Bmatrix}$$

$Use \quad \phi 8@260\text{c/c}$

$$v_s = \frac{A_v f_{yd} d}{S} = 53.38KN$$

$v_c + v_s = 94.52 > v_d = 41.17KN \quad \text{OK.}$

Beam Type 4

Slab thickness (h_f) = 0.1m   Use C- 25 concrete f_{ck}= 20MPa

Rib depth = 0.5m f_{cd} = 11.33MPa

Web width of beam (b_w) = 0.1m   S-300 steel f_{yk}= 300MPa

Flange width of beam (b_e) = 1m f_{yd} = 260.87MPa

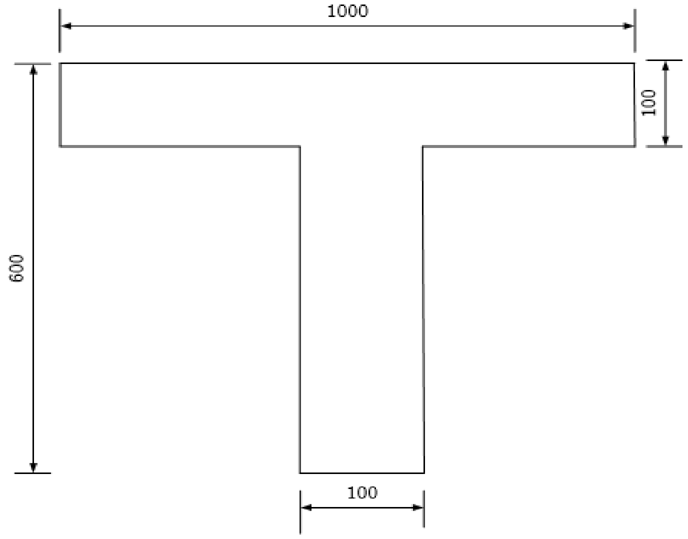


Fig. 4.36 Section view for design example beam type 4

- **Compute deflection of beam:**

Use equation A.1, A.4 and A.7 to determine maximum deflection of beam when the soil is shrink.

Deflection of beam for shrinkage = 0.00042m < $\frac{L}{480} = \frac{6}{480} = 0.0125m$ OK.

Use equation (4.12), (4.15) and (4.18) to determine maximum deflection of the beam when the soil is swell.

Deflection of beam for swelling = 0.009m < 0.0125   OK

Maximum flange width (beam spacing) are least of:

$$i)\quad b_w + \frac{L}{5} = 100 + \frac{6000}{5} = 1300mm$$

ii) Actual width of top slab extending between the center of adjacent span =1500mm

Therefore assume flange width (b_e) = 1000mm OK.

- **Design the beam for moment:**

Use equation (4.13), (4.16) and (4.19) to determine the maximum moment of the beam when the soil is swell.

Design for positive moment

$$d = 600 - 50 - 8 - 7 = 535mm$$

$$M_u = 31.04KNm$$

Assuming the N.A to fall in the flange

$$M_u = 0.8xb_e f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 6.47mm$$
$$x = 6.47mm < h_f = 100mm$$

This implies that the N.A falls within the flange

$$A_s = \frac{0.8xb_e f_{cd}}{f_{yd}} = 224.75mm^2$$

$$Number\ \text{of}\ \phi 14\text{mm rod} = \frac{224.75}{\frac{\pi * 14^2}{4}} = 1.4\ \text{bar}, \quad \text{provide } 2\phi 14 \text{ bars}$$

Use equation A.2, A.5 and A.8 to determine maximum moment of the beam when the soil is shrink.

Design for negative moment

$d = 600 - 25 - 8 - 7 = 562mm$

$M_u = 22.5KNm$

$M_u = 0.8xb_w f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 45.65mm$

$A_s = \dfrac{0.8xb_w f_{cd}}{f_{yd}} = 158.62\ mm^2$

$Number\ \text{of}\ \phi 14mm\ \text{rod} = \dfrac{158.62}{\dfrac{\pi * 14^2}{4}} = 1.03, \quad \text{provide}\ 2\phi 14\ \text{bars}$

- **Design the beam for shear**

$v_d = 37.07KN$

Compute ultimate shear resistance of concrete

$v_{Rd} = 0.25 f_{cd} b_w d = 226.03KN > v_d = 37.07\text{KN} \quad \text{OK.}$

Compute concrete shear carrying capacity

$v_c = 0.25 f_{ctd} K_1 K_2 b_w d = 17.41KN < v_d = 37.07KN \quad \text{K}_1 = 1.19 < 2, \quad \text{K}_2 = 1.068 > 1 \quad \text{OK.}$

$S = \dfrac{A_v f_{yd}}{0.4 b_w} = 652mm \qquad S_{\max} \leq \begin{Bmatrix} 0.5d = 266mm \\ 300mm \end{Bmatrix}$

$Use \quad \phi 8@260\text{c/c}$

$v_s = \dfrac{A_v f_{yd} d}{S} = 53.38KN$

$v_c + v_s = 70.79 > v_d = 37.07KN \quad \text{OK.}$

Beam Type 5

Slab thickness (h_f) = 0.1m Use C- 25 concrete f_{ck}= 20MPa

Rib depth = 0.5m f_{cd} = 11.33MPa

Web width of beam (b_w) = 0.1m S-300 steel f_{yk}= 300MPa

Flange width of beam (b_e) = 0.75m f_{yd} = 260.87MPa

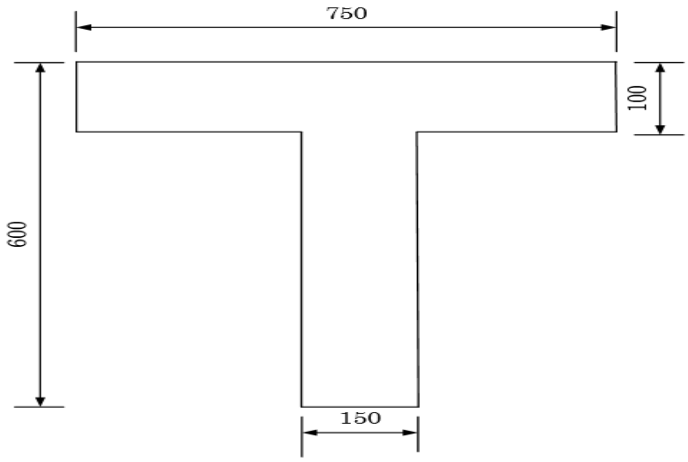


Fig. 4.37Section view for design example beam type 5

- **Compute deflection of beam:**

Use equation A.1, A.4 and A.7 to determine maximum deflection of beam when the soil is shrink.

Deflection of beam for shrinkage = 0.00034m < $\frac{L}{480} = \frac{6}{480} = 0.0125m$ OK.

Use equation (4.12), (4.15) and (4.18) to determine maximum deflection of the beam when the soil is swell.

Deflection of beam for swelling = 0.009m < 0.0125 OK

Maximum flange width (beam spacing) are least of:

$$i)\quad b_w + \frac{L}{5} = 100 + \frac{6000}{5} = 1300mm$$

ii) Actual width of top slab extending between the center of adjacent span =1500mm

Therefore assume flange width (b_e) = 750mm OK.

- **Design the beam for moment:**

Use equation (4.13), (4.16) and (4.19) to determine the maximum moment of the beam when the soil is swell.

Design for positive moment

$$d = 600 - 50 - 8 - 7 = 535mm$$

$$M_u = 26.83KNm$$

Assuming the N.A to fall in the flange

$$M_u = 0.8xb_e f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 7.5mm$$
$$x = 7.5mm < h_f = 100mm$$

This implies that the N.A falls within the flange

$$A_s = \frac{0.8xb_e f_{cd}}{f_{yd}} = 195.44mm^2$$

$$Number\ \text{of}\ \phi 14\text{mm rod} = \frac{195.44}{\frac{\pi * 14^2}{4}} = 1.26\ \text{bar}, \quad \text{provide } 2\phi 14 \text{ bars}$$

Use equation A.2, A.5 and A.8 to determine maximum moment of the beam when the soil is shrink.

Design for negative moment

$d = 600 - 25 - 8 - 7 = 562mm$

$M_u = 16.875KNm$

$M_u = 0.8xb_w f_{cd}(d - 0.4x) \quad \rightarrow \quad x = 33.95mm$

$A_s = \frac{0.8xb_w f_{cd}}{f_{yd}} = 117.96\ mm^2$

$Number\ \text{of}\ \phi 14mm\ \text{rod} = \frac{117.96}{\frac{\pi * 14^2}{4}} = 0.76, \quad \text{provide } 2\phi 14\ \text{bars}$

- **Design the beam for shear**

  $v_d = 35.02KN$

Compute ultimate shear resistance of concrete

$v_{Rd} = 0.25 f_{cd} b_w d = 150.7KN > v_d = 35.02\text{KN} \quad \text{OK.}$

Compute concrete shear carrying capacity

$v_c = 0.25 f_{ctd} K_1 K_2 b_w d = 17.41KN < v_d = 35.02KN \quad \text{K}_1 = 1.19 < 2, \quad \text{K}_2 = 1.068 > 1 \quad \text{OK.}$

$S = \frac{A_v f_{yd}}{0.4b_w} = 652mm \qquad S_{\max} \leq \begin{Bmatrix} 0.5d = 266mm \\ 300mm \end{Bmatrix}$

$Use \quad \phi 8@260\text{c/c}$

$v_s = \frac{A_v f_{yd} d}{S} = 53.38KN$

$v_c + v_s = 70.79 > v_d = 35.02KN \quad \text{OK.}$

Design isolated footing to compare with waffle mat foundation cost.

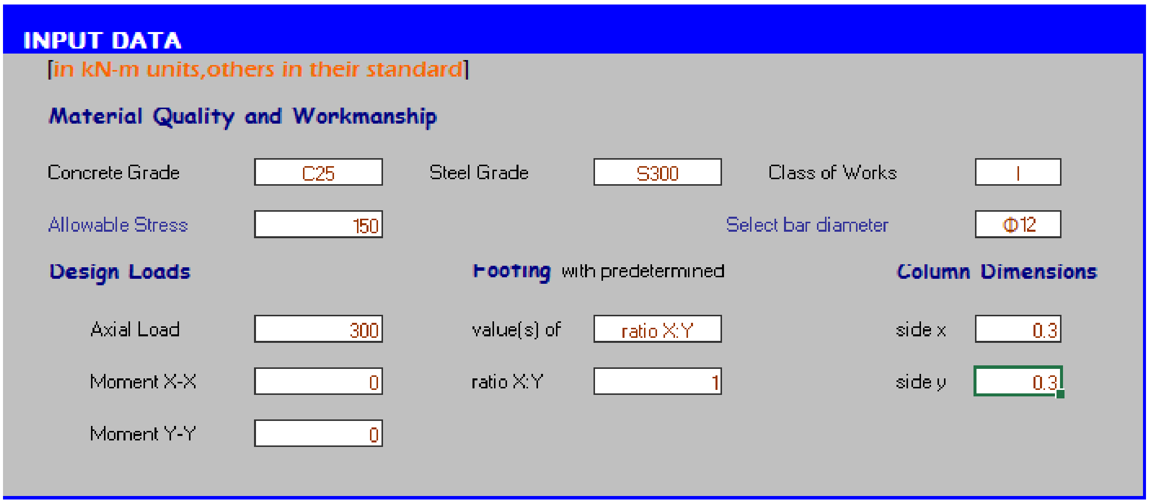
- Excavate 3m thickness expansive soil and replace with 1m thickness selected material.

  Assume foundation depth is 2m from natural ground surface.

- Assume backfill selected material soil allowable bearing capacity is 150KPa

Footing type 1

Use C-25 concrete S-300 stee ϕ12 bar Axial load = 300KN



OUTPUTS

[in kN-m units, unless it is indicated]

Partial safety factor and Design Constants

Concrete	γ_c	fck	fcd	fctd	**Design eccentricities**
	1.50	20.00	11.33	1.04	
Steel	γ_s	fyk	fyd	ρ_{min}	ex= 0.000
	1.15	300	260.87	0.002	ey= 0.000

PROPORTIONING

FOOTING DIMENSIONS

from calculation/given

side X	side Y
1.41	**1.41**

CONTACT EARTH PRESSURE DISTRIBUTION
(At the four corners)

σ_A	σ_B	σ_C	σ_D
150.00	150.00	150.00	150.00

STRUCTURAL DESIGN

EFFECTIVE DEPTH REQUIRED FOR:

(a) punching shear ,d_{eff}= 0.190

(b) wide beam shear ,d_{eff}= 0.193

Observation: **the critical d_{eff}= 0.193**

FOOTING THICKNESS (rounded to multipe of 50) = **300mm**

Hence, modified critical d_{eff} = 0.232

MAXIMUM MOMENTS and area of reinforcement

	M_{max}	As_{req}(mm²)	No. bar	As_{prov}(mm²)	spacing(c/c)
Mdy=	32.92	554.72	5Φ12	565.20	280
Mdx=	32.92	554.72	5Φ12	565.20	280

Result: provide Φ12 with spacing of 280mm(c/c) in the direction of X-axis
provide Φ12 with spacing of 280mm(c/c) in the direction of Y-axis.

Therefore take

Footing dimension =1.5 x 1.5m

Footing thickness = 0.35m

Provide 5ϕ12 with spacing of 280mm(c/c) in the direction of X-axis

Provide 5ϕ12 with spacing of 280mm(c/c) in the direction of Y-axis

Footing type 2

Axial load = 300KN

INPUT DATA

[in kN-m units,others in their standard]

Material Quality and Workmanship

Concrete Grade	C25	Steel Grade	S300	Class of Works	I
Allowable Stress	150			Select bar diameter	Φ12

Design Loads		**Footing** with predetermined		**Column Dimensions**	
Axial Load	200	value(s) of	ratio X:Y	side x	0.3
Moment X-X	0	ratio X:Y	1	side y	0.3
Moment Y-Y	0				

OUTPUTS

[in kN-m units, unless it is indicated]

Partial safety factor and Design Constants

Concrete	γ_c	fck	fcd	fctd	**Design**
	1.50	20.00	11.33	1.04	**eccentricities**
Steel	γ_s	fyk	fyd	ρ_{min}	ex= 0.000
	1.15	300	260.87	0.002	ey= 0.000

PROPORTIONING

FOOTING DIMENSIONS		**CONTACT EARTH PRESSURE DISTRIBUTION (At the four corners)**			
from calculation/given					
side X	side Y	σ_A	σ_B	σ_C	σ_D
1.15	1.15	150.00	150.00	150.00	150.00

STRUCTURAL DESIGN

EFFECTIVE DEPTH REQUIRED FOR:

(a) punching shear ,d_{eff}= 0.144

(b) wide beam shear ,d_{eff}= 0.148

Observation: **the critical d_{eff}= 0.148**

FOOTING THICKNESS (rounded to multipe of 50) = **250mm**

Hence, modified critical d_{eff} = 0.182

MAXIMUM MOMENTS and **area of reinforcement**

	M_{max}	As_{req}(mm²)	No. bar	As_{prov}(mm²)	spacing(c/c)
Mdy=	15.82	350.26	4Φ12	452.16	350
Mdx=	15.82	350.26	4Φ12	452.16	350

Result: provide Φ12 with spacing of 350mm(c/c) in the direction of X-axis **[Asmim is used]**
provide Φ12 with spacing of 350mm(c/c) in the direction of Y-axis. **[Asmim is used]**

Therefore take

Footing dimension =1.3 x 1.3m

Footing thickness = 0.3m

Provide 4ϕ12 with spacing of 350mm(c/c) in the direction of X-axis

Provide 4ϕ12 with spacing of 350mm(c/c) in the direction of Y-axis

4.4 Cost Analysis

As it can be seen from Tables 4.18 and 4.19, the total cost isolated footing is higher than waffle mat foundation. So the waffle mat foundation the cheapest because it does not require excavate the existing soil, fill selected material, masonry, foundation column, and basaltic hardcore works.

The unit rate for waffle mat foundation and isolated footing is taken from the Addis Ababa City Government Construction Bureau (2011, 3rd quarter construction material unit price) whereas wafflebox unit rate of the waffle mat foundation, due to local unavailability, is adopted from international market.

N.B.: All prices are expressed here after are in **ETB (Ethiopian Birr)** and are net prices which are subject to 15% VAT

Table (4.18) Waffle mat foundation bill of quantity

Item no	Description	Unit	Qty	Rate	Amount
1	C-25 concrete for waffle mat foundation	m3	37.875	2623.1	99,350.29
2	Deformable bar 20mm diameter	Kg	2370.37	49.82	118,091.83
3	Deformable bar 14mm diameter	Kg	696.89	50.91	35,478.67
4	Deformable bar 8mm diameter	Kg	771.39	54.67	42,171.89
5	waffle box	pc	389	100	38,900.00
	TOTAL......................................				**333,992.69**
	VAT (15%).................................				**50,098.90**
	GRAND TOTAL.........................				**384,091.59**

Table (4.19) Isolated footing bill of quantity

Item no	**Description**	**Unit**	Qty	**Rate**	**Amount**
1	C-25 concrete for isolated foundation	m3	20.106	2623.11	52,740.25
2	Deformable bar 12mm diametere	Kg	645.33	52.75	34,041.16
3	Deformable bar 8mm diametere	Kg	262.95	54.67	14,375.48
4	Bulk excavation of expansive soil to remove	m3	432	156.89	67,776.48
5	Back fill selected material to replace expansive soil	m3	367.49	377.65	138,782.60
7	25cm thick basaltic hard core(basaltic stone)	m2	123.21	245.05	30,192.61
	TOTAL......................................				**337,908.57**
	VAT (15%)..				**50,686.29**
	GRAND TOTAL................................				**388,594.86**

5 CONCLUSIONS AND RECCOMENDATIONS

5.1 General

In Ethiopia there is a general increase in building construction with ever population increase at a very faster rate and large country area is composed with expansive soil. So there is necessary to understand the characteristics of this soil type and alternative foundation type for buildings.

The characteristics of expansive soil depend on different factors such as origin, mineralogy type and amount of clay, weather and climate condition. In this thesis, parametric study is conducted for different parameters such that surface edge suction, drying period, active depth and diffusitive coefficient that have effect on suction distribution, mound of soil and edge moisture distance by using formula Remon I. Abdelmalak (2007), he depends on pioneer work Mitchell (1979). When heave case conduct parametric study for interconnected T-beam by using M.hetenyi (1946) formula, assume the beam as simply supported and considering subgrade reaction of the soil.

5.2 Conclusion

1. Diffusion coefficient has more than 100% increasing the depth of moisture distribution, edge moisture distance and soil heave for loose (cracked) soil compare with tight (uncracked) soil. So it has great sensitivity on expansive soil rather than other parameters. The diffusion coefficient is high when the soil is loos or cracked, then the moisture easily passes through the soil both vertically and horizontally. Heave and shrinkage of the soil is high. When the soil is tight or uncracked, the moisture is not easily passes through the soil.
2. Differential soil heave and edge moisture distance increase when slab dimension increase. When slab dimensions are small, moisture easily passes through underneath slab so differential soil heave are small at center and edge of slab and vice versa.

3. For dry case beam depth has great sensitivity compare with flange width, flange thickness and beam width on the deflection of the beam. The beam deflection decrease by 75.6% when the beam depth increase by double (0.2m to 0.4m). Flange thickness has secondary sensitivity to deflection.
4. For heave case beam width increased the beam deflection also increase. Because when increase the beam width also increase contact area and swelling pressure of the soil. So it has higher sensitivity on the deflection of the beam. However the beam deflection is small for soil heave this mean the effect of swelling pressure on the beam is small.
5. Waffle mat foundation is compatible with swelling and shrinkage characteristics of expansive soil. It has high stiffness, low cost and save construction time compare with other foundation type.

5.3 Recommendation for Further study

For further study, all the analysis in this study also was performed that waffle mat foundation as interconnected T-beam. So ignore the plate effect of the slab. Thus, to get more accurate in terms of design and cost the plate effect need to be considered

Besides that, all the analysis was performed in this study the static characteristics of waffle mat foundation, to use this foundation in factory the dynamic behavior need to be considered.

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APPENDIX A

Dry case

Waffle mat foundation structure analyses are assume as T- beams resting on expansive soil and they are fixed support at the middle point of the beam as show in Fig.(3.9). There are three load on beam p1 = 66.67kN at free end of beam, P2 = 75kN at 2m distance from fixed end of beam and distribution load on entire beam length its value depends on beam spacing and analyses by use superposition. However, assume distributed slab area load w=10kN/m², this load transfer to T-beam when consider waffle mat foundation as interconnected T-beam.

Case1

Evaluate deflection of concentrated load at free end of T-beam by following equation A.1.

$$y_1 = \frac{-p_1 x^2}{6EI}(3L - x) \qquad \text{A.1}$$

$$M_1 = -p_1(L - x) \qquad \text{A.2}$$

$$v = p_1 \qquad \text{A.3}$$

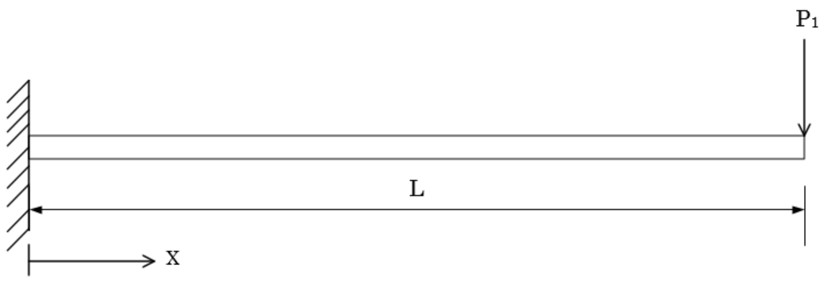

Fig. A.1 Cantilever beam with concentrated load at free end

Case 2

Evaluate deflection of concentrated load at 2m from fixed end of T-beam by following equation A.4.

$$y_2 = \frac{-p_2 x^2}{6EI}(3a - x) \qquad \text{for } x < a$$
$$y_2 = -\frac{p_2 a^2}{6EI}(3x - a) \qquad \text{for } a < x < L \qquad \text{A.4}$$

$$M_2 = -p_2(a - x) \qquad \text{for } x < a$$
$$M_2 = 0 \qquad \text{for } a < x < L \qquad \text{A.5}$$

$$v_2 = p_2 \qquad \text{for } x < a$$
$$v_2 = 0 \qquad \text{for } a < x < L \qquad \text{A.6}$$

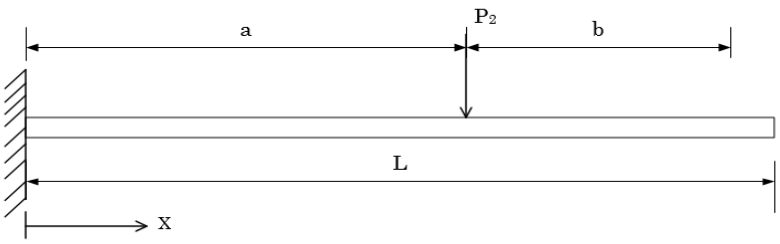

Fig. A.2 Cantilever beam with concentrated load at arbitrary point

Case 3

Evaluate deflection of uniformly distributed load on entire T-beam by following equation A.7

$$y_3 = -\frac{wx^2}{24EI}(6L^2 - 4Lx + x^2) \qquad \text{A.7}$$

$$M_3 = -\frac{w(L - x)^2}{2} \qquad \text{A.8}$$

$$v_3 = w(L - x) \qquad \text{A.9}$$

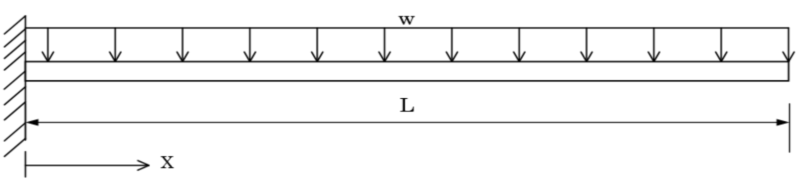

Fig. A.3 Cantilever beam with uniformly distribute load

A parametric study was undertaken to evaluate equation A.1, A.4 and A.7 by using superposition for predicting beam deflection profiles on expansive soil surface for dry case.

a) Flange width

The effects of the flange width parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant flange thickness 0.1m, beam depth 0.3m and beam width 0.3m. Assume the imposed load P1=66.67kN at free end, P2=75kN at 2m distance from center of beam and different uniformly distributed load it depends on flange width of the beam, modulus of concrete 29GPa and length of beam span 12m. The flange width parameter was changed from a small value 0.75m and to a large value 2m. Table A.1 and Fig. A.4 give the beam deflection envelopes showing the effects of flange width. The maximum deflection of the beam vary from 90mm to 77mm when the flange width increasing by double (0.75m to 1.5m).

Table A.1 beam deflection (m) vs X (m) at Different flange width for soil dry.

x	S=0.75m	S=1m	S=1.25m	S=1.5m	S=1.75m	S=2m
0	0	0	0	0	0	0
0.25	-0.00027	-0.00025	-0.00024	-0.00024	-0.00023	-0.00023
0.5	-0.00107	-0.00099	-0.00095	-0.00092	-0.00091	-0.0009
0.83	-0.00286	-0.00265	-0.00254	-0.00247	-0.00243	-0.00241
1	-0.00409	-0.00379	-0.00362	-0.00352	-0.00347	-0.00344
1.25	-0.00624	-0.00578	-0.00552	-0.00537	-0.00529	-0.00525
1.5	-0.00876	-0.00812	-0.00775	-0.00754	-0.00742	-0.00736
1.75	-0.01163	-0.01077	-0.01029	-0.01001	-0.00985	-0.00977
2	-0.0148	-0.0137	-0.01309	-0.01273	-0.01253	-0.01242
2.25	-0.01824	-0.01689	-0.01613	-0.01569	-0.01544	-0.01531
2.5	-0.02193	-0.02031	-0.01939	-0.01886	-0.01856	-0.0184
2.75	-0.02586	-0.02394	-0.02286	-0.02223	-0.02187	-0.02168
3	-0.03	-0.02777	-0.02652	-0.02578	-0.02536	-0.02514
3.25	-0.03434	-0.03178	-0.03034	-0.0295	-0.02901	-0.02876
3.5	-0.03885	-0.03596	-0.03432	-0.03336	-0.03281	-0.03252
3.75	-0.04353	-0.04028	-0.03844	-0.03736	-0.03674	-0.0364
4	-0.04836	-0.04474	-0.04269	-0.04148	-0.04078	-0.04041
4.25	-0.05331	-0.04931	-0.04704	-0.0457	-0.04493	-0.04451
4.5	-0.05837	-0.05399	-0.05149	-0.05002	-0.04916	-0.0487
4.75	-0.06353	-0.05874	-0.05602	-0.05441	-0.05347	-0.05295
5	-0.06876	-0.06357	-0.06062	-0.05886	-0.05783	-0.05727
5.17	-0.07236	-0.06689	-0.06377	-0.06192	-0.06083	-0.06024
5.5	-0.0794	-0.07339	-0.06995	-0.06791	-0.0667	-0.06604
5.75	-0.08478	-0.07834	-0.07466	-0.07247	-0.07118	-0.07046
6	-0.09017	-0.08331	-0.07939	-0.07705	-0.07566	-0.07489

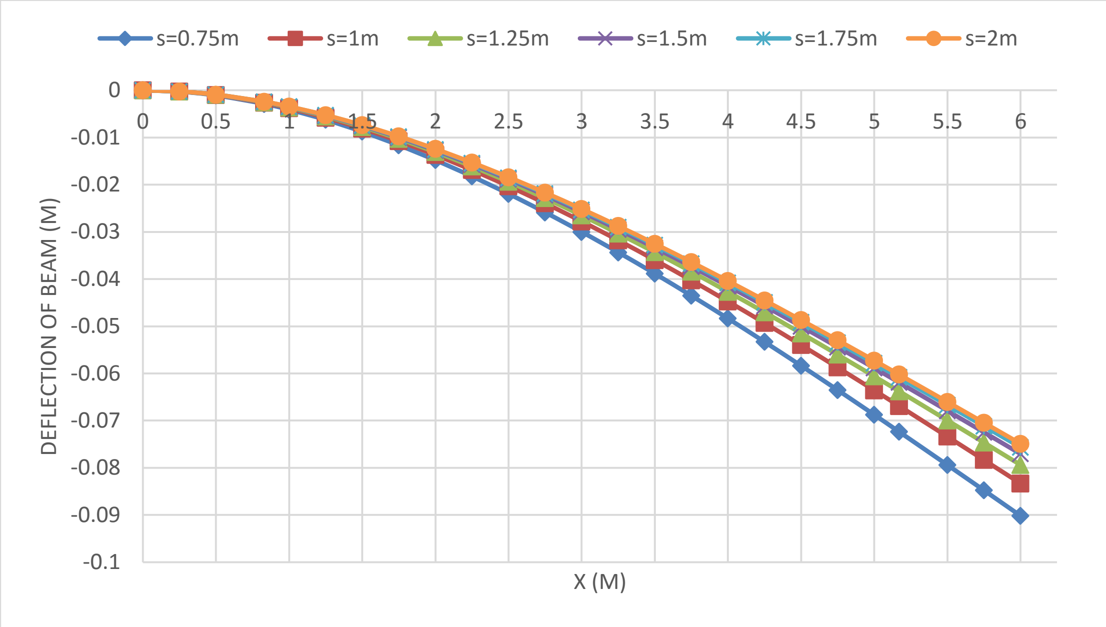

Fig. A.4 beam deflection (m) vs X (m) at Different flange width for soil dry

b) Flange thickness

Considering flange thickness parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant beam spacing1.5m, rib depth 0.3m and beam width 0.3m. Assume the imposed loads P1=66.67kN at free end, P2 = 75kN at 2m from center of beam and 7.5KN/m uniformly distributed load, modulus of concrete 29GPa and length of beam span 12m. The flange thickness parameter was changed from a small value 0.1m and to a large value 0.35m. Table A.2 and Fig. A.5 give the beam deflection envelopes showing the effects of flange thickness of T-beam. The maximum deflection vary from 77mm to 39mm when the flange thickness increasing by double (0.1m to 0.2m).

Table A.2 beam deflection (m) vs X (m) at Different flange thickness for soil dry.

x	t=0.1m	t=0.15m	t=0.2m	t=0.25m	t=0.3m	t=0.35m
0	0	0	0	0	0	0
0.25	-0.00024	-0.00017	-0.00012	-8.9E-05	-6.7E-05	-5.1E-05
0.5	-0.00092	-0.00065	-0.00047	-0.00035	-0.00026	-0.0002
0.83	-0.00247	-0.00173	-0.00125	-0.00093	-0.0007	-0.00053
1	-0.00352	-0.00247	-0.00179	-0.00132	-0.001	-0.00076
1.25	-0.00537	-0.00377	-0.00273	-0.00202	-0.00152	-0.00116
1.5	-0.00754	-0.00529	-0.00383	-0.00283	-0.00213	-0.00163
1.75	-0.01001	-0.00701	-0.00509	-0.00376	-0.00283	-0.00216
2	-0.01273	-0.00892	-0.00647	-0.00478	-0.0036	-0.00274
2.25	-0.01569	-0.011	-0.00797	-0.0059	-0.00443	-0.00338
2.5	-0.01886	-0.01322	-0.00958	-0.00709	-0.00533	-0.00407
2.75	-0.02223	-0.01558	-0.0113	-0.00835	-0.00628	-0.00479
3	-0.02578	-0.01807	-0.0131	-0.00969	-0.00728	-0.00556
3.25	-0.0295	-0.02067	-0.01499	-0.01108	-0.00833	-0.00636
3.5	-0.03336	-0.02338	-0.01695	-0.01254	-0.00942	-0.00719
3.75	-0.03736	-0.02619	-0.01899	-0.01404	-0.01055	-0.00805
4	-0.04148	-0.02907	-0.02108	-0.01559	-0.01171	-0.00894
4.25	-0.0457	-0.03204	-0.02323	-0.01717	-0.01291	-0.00985
4.5	-0.05002	-0.03506	-0.02542	-0.01879	-0.01413	-0.01078
4.75	-0.05441	-0.03814	-0.02765	-0.02044	-0.01537	-0.01173
5	-0.05886	-0.04126	-0.02991	-0.02212	-0.01662	-0.01269
5.17	-0.06192	-0.0434	-0.03147	-0.02327	-0.01749	-0.01335
5.5	-0.06791	-0.0476	-0.03451	-0.02552	-0.01918	-0.01464
5.75	-0.07247	-0.0508	-0.03683	-0.02723	-0.02047	-0.01562
6	-0.07705	-0.05401	-0.03915	-0.02895	-0.02176	-0.01661

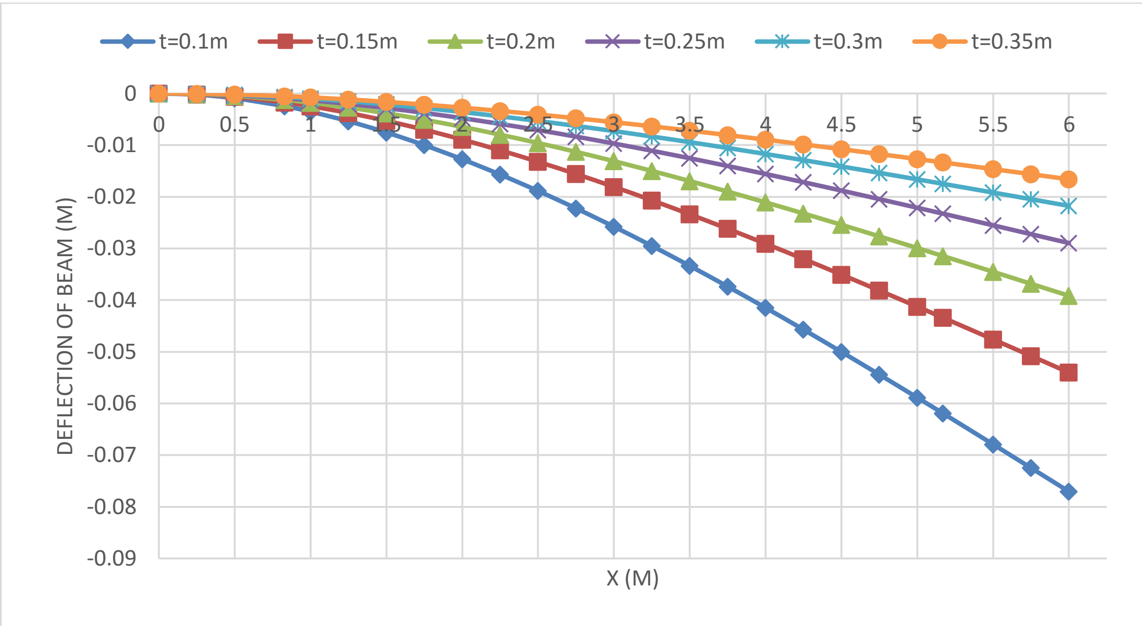

Fig. A.5 beam deflection (m) vs X(m) at Different flange thickness for soil dry

c) Beam width

Considering beam width parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant beam spacing1.5m, beam depth 0.3m and flange thickness 0.1m. Assume the imposed loads P1=66.67kN at free end , P2 = 75kN at 2m from center of beam and 7.5KN uniformly distributed load, modulus of concrete 29GPa and length of beam span 12m. The beam width parameter was changed from a small value 0.15m and to a large value 0.4m. Table A.3 and Fig. A.6 give the beam deflection envelopes showing the effects of beam width. The maximum deflection of the beam vary from 89mm to 54mm when the beam width increasing by double (0.15m to 0.3m).

Table A.3 beam deflection (m) vs X (m) at Different beam width for soil dry.

x	b=0.15m	b=0.2m	b=0.25m	b=0.3m	b=0.35m	b=0.4m
0	0	0	0	0	0	0
0.25	-0.00039	-0.00032	-0.00027	-0.00024	-0.00021	-0.0002
0.5	-0.00153	-0.00123	-0.00105	-0.00092	-0.00083	-0.00076
0.83	-0.00408	-0.00329	-0.0028	-0.00247	-0.00223	-0.00204
1	-0.00582	-0.0047	-0.004	-0.00352	-0.00318	-0.00291
1.25	-0.00887	-0.00716	-0.0061	-0.00537	-0.00484	-0.00444
1.5	-0.01246	-0.01005	-0.00856	-0.00754	-0.0068	-0.00623
1.75	-0.01652	-0.01333	-0.01136	-0.01001	-0.00902	-0.00827
2	-0.02102	-0.01696	-0.01445	-0.01273	-0.01148	-0.01052
2.25	-0.02591	-0.0209	-0.01781	-0.01569	-0.01414	-0.01296
2.5	-0.03114	-0.02513	-0.02141	-0.01886	-0.017	-0.01558
2.75	-0.03671	-0.02962	-0.02523	-0.02223	-0.02004	-0.01837
3	-0.04257	-0.03435	-0.02926	-0.02578	-0.02324	-0.0213
3.25	-0.0487	-0.0393	-0.03347	-0.0295	-0.02659	-0.02437
3.5	-0.05509	-0.04445	-0.03786	-0.03336	-0.03008	-0.02756
3.75	-0.06169	-0.04978	-0.0424	-0.03736	-0.03368	-0.03087
4	-0.06849	-0.05526	-0.04707	-0.04148	-0.0374	-0.03427
4.25	-0.07547	-0.06089	-0.05187	-0.0457	-0.0412	-0.03776
4.5	-0.08259	-0.06664	-0.05676	-0.05002	-0.04509	-0.04132
4.75	-0.08984	-0.07249	-0.06175	-0.05441	-0.04905	-0.04495
5	-0.0972	-0.07842	-0.0668	-0.05886	-0.05307	-0.04863
5.17	-0.10225	-0.0825	-0.07027	-0.06192	-0.05582	-0.05116
5.5	-0.11213	-0.09047	-0.07707	-0.06791	-0.06122	-0.0561
5.75	-0.11967	-0.09655	-0.08225	-0.07247	-0.06534	-0.05988
6	-0.12723	-0.10265	-0.08744	-0.07705	-0.06946	-0.06366

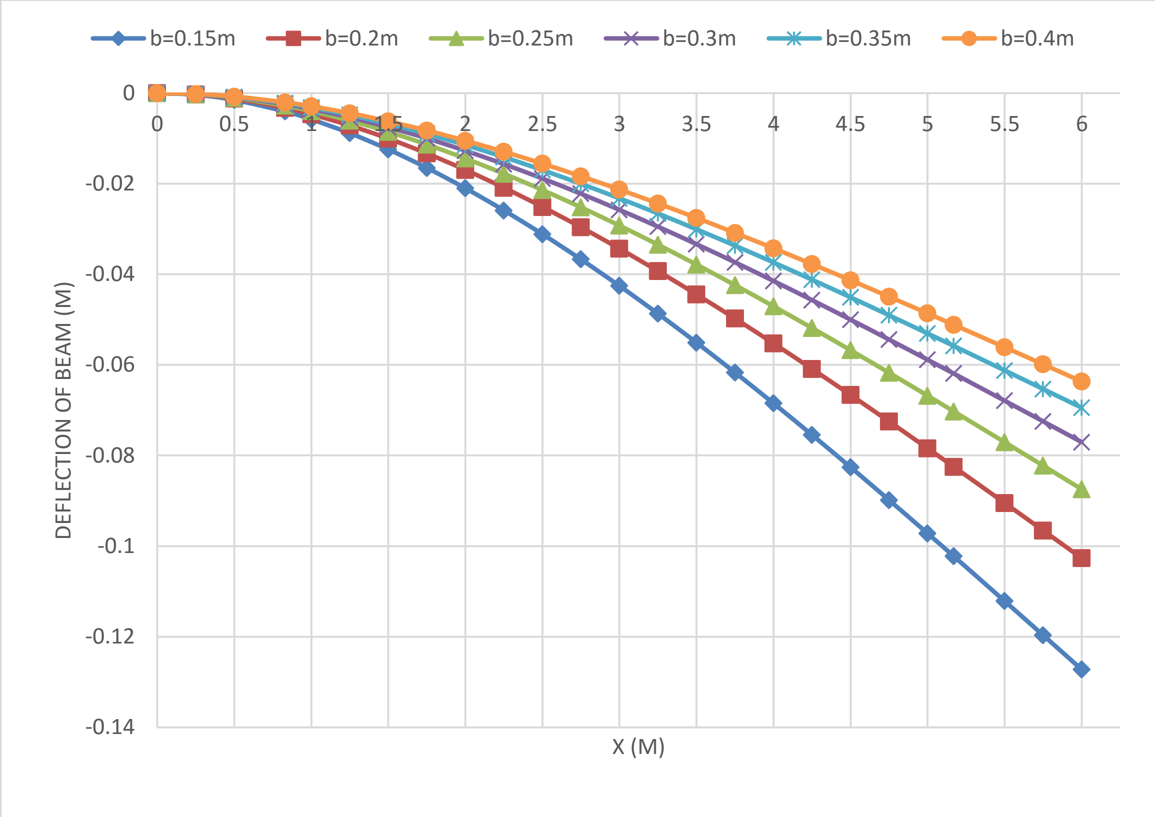

Fig A.6 beam deflection (m) vs X(m) at Different beam width for soil dry

d) Beam depth

The effects of the beam depth parameter in predicting the beam deflection using superposition of the above equations are undertaken for a constant flange thickness 0.1m, beam spacing 1.5m and beam width 0.3m. Assume the imposed loads P_1= 66.67kN at free end,P_2= 75kN at 2m from center of beam and 7.5KN/m uniformly distributed load, modulus of concrete 29GPa and length of beam span 12m. The beam depth parameter was changed from a small value 0.2m and to a large value 0.5m. Table A.4 and Fig.A.7 give the beam deflection envelopes showing the effects of beam depth. The maximum deflection of the beam vary from 182mm to 39mm when the depth increasing by double (0.2m to 0.4m).

Table A.4 beam deflection (m) vs X (m) at Different beam depth for soil dry.

x	D=0.2m	D=0.25m	D=0.3m	D=0.35m	D=0.4m	D=0.45m	D=0.5m
0	0	0	0	0	0	0	0
0.25	-0.00056	-0.00035	-0.00024	-0.00017	-0.00012	-9.3E-05	-7.2E-05
0.5	-0.00219	-0.00138	-0.00092	-0.00065	-0.00048	-0.00036	-0.00028
0.83	-0.00584	-0.00368	-0.00247	-0.00174	-0.00128	-0.00097	-0.00075
1	-0.00834	-0.00525	-0.00352	-0.00249	-0.00182	-0.00138	-0.00107
1.25	-0.01271	-0.008	-0.00537	-0.00379	-0.00278	-0.00211	-0.00164
1.5	-0.01784	-0.01124	-0.00754	-0.00532	-0.00391	-0.00296	-0.0023
1.75	-0.02367	-0.01491	-0.01001	-0.00706	-0.00518	-0.00392	-0.00305
2	-0.03012	-0.01897	-0.01273	-0.00898	-0.00659	-0.00499	-0.00388
2.25	-0.03712	-0.02338	-0.01569	-0.01107	-0.00812	-0.00615	-0.00479
2.5	-0.04462	-0.0281	-0.01886	-0.01331	-0.00976	-0.0074	-0.00575
2.75	-0.05259	-0.03313	-0.02223	-0.01568	-0.01151	-0.00872	-0.00678
3	-0.06099	-0.03842	-0.02578	-0.01819	-0.01335	-0.01011	-0.00786
3.25	-0.06978	-0.04395	-0.0295	-0.02081	-0.01527	-0.01157	-0.009
3.5	-0.07892	-0.04971	-0.03336	-0.02353	-0.01727	-0.01308	-0.01018
3.75	-0.08838	-0.05567	-0.03736	-0.02635	-0.01934	-0.01465	-0.0114
4	-0.09813	-0.06181	-0.04148	-0.02926	-0.02147	-0.01627	-0.01265
4.25	-0.10812	-0.0681	-0.0457	-0.03224	-0.02366	-0.01793	-0.01394
4.5	-0.11832	-0.07453	-0.05002	-0.03528	-0.02589	-0.01962	-0.01526
4.75	-0.12871	-0.08107	-0.05441	-0.03838	-0.02817	-0.02134	-0.01659
5	-0.13925	-0.08771	-0.05886	-0.04152	-0.03047	-0.02309	-0.01795
5.17	-0.14648	-0.09227	-0.06192	-0.04368	-0.03206	-0.02429	-0.01889
5.5	-0.16065	-0.10119	-0.06791	-0.0479	-0.03515	-0.02663	-0.02071
5.75	-0.17144	-0.10799	-0.07247	-0.05112	-0.03752	-0.02842	-0.0221
6	-0.18227	-0.11481	-0.07705	-0.05435	-0.03989	-0.03022	-0.0235

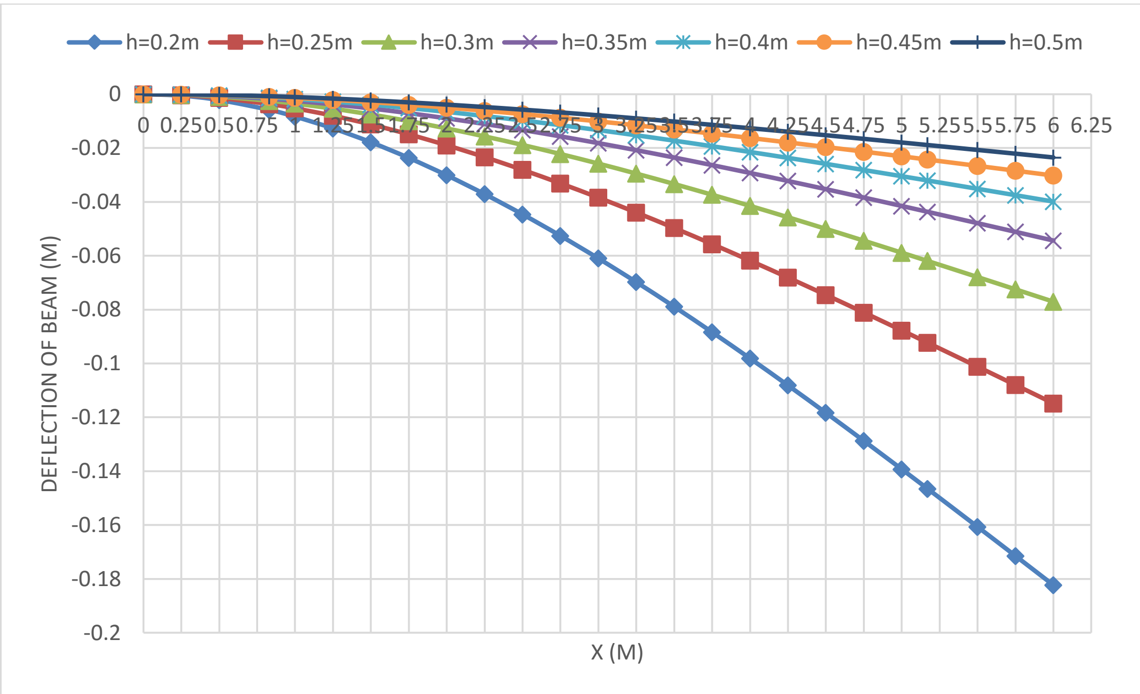

Fig. A.7 beam deflection (m) vs X(m) at Different beam depth for soil dry

Table A.5 Summary of maximum beam deflection due to different parameters

Parameters	Double Increase		Maximum deflection of the beam		
			Decreasing		Decrease in percent (%)
	From	To	From	To	
Flange width	0.75m	1.5m	90mm	77mm	14.40
Flange thickness	0.1m	0.2m	77mm	39mm	49.3
Beam width	0.15m	0.3m	89mm	54mm	39.3
Beam depth	0.2m	0.4m	182mm	39mm	75.6

The above table indicate that beam depth has more sensitivity to beam deflection than other beam parameters.

Concentrated load at one end of beam for soil heave

Table A.6. Beam deflection (m) vs X (m) at Different flange width for soil swell.

P=66.67KPA	E=29GPA	L=6m				
x	S=0.75m	S=1m	S=1.25m	S=1.5m	S=1.75m	S=2m
0	0.005163	0.005045	0.004966	0.004908	0.004864	0.004828
0.25	0.004582	0.004494	0.004435	0.004391	0.004358	0.004331
0.5	0.004015	0.003955	0.003915	0.003885	0.003862	0.003844
0.83	0.003304	0.003277	0.003259	0.003246	0.003236	0.003227
1	0.002959	0.002948	0.00294	0.002934	0.002929	0.002925
1.25	0.002482	0.002491	0.002496	0.0025	0.002503	0.002505
1.5	0.002046	0.00207	0.002086	0.002098	0.002106	0.002113
1.75	0.00165	0.001687	0.001711	0.001729	0.001742	0.001753
2	0.001296	0.001341	0.001371	0.001394	0.00141	0.001424
2.25	0.000982	0.001033	0.001067	0.001091	0.00111	0.001126
2.5	0.000707	0.000759	0.000795	0.000821	0.000841	0.000857
2.75	0.000467	0.000519	0.000555	0.000581	0.000601	0.000617
3	0.000261	0.00031	0.000343	0.000368	0.000387	0.000402
3.25	8.35E-05	0.000127	0.000158	0.00018	0.000197	0.000211
3.5	-6.8E-05	-3.1E-05	-4.7E-06	1.44E-05	2.93E-05	4.11E-05
3.75	-0.0002	-0.00017	-0.00015	-0.00013	-0.00012	-0.00011
4	-0.00031	-0.00029	-0.00027	-0.00026	-0.00025	-0.00025
4.25	-0.0004	-0.00039	-0.00038	-0.00038	-0.00037	-0.00037
4.5	-0.00048	-0.00049	-0.00049	-0.00049	-0.00049	-0.00049
4.75	-0.00056	-0.00057	-0.00058	-0.00058	-0.00059	-0.00059
5	-0.00063	-0.00065	-0.00066	-0.00068	-0.00068	-0.00069
5.17	-0.00067	-0.0007	-0.00072	-0.00074	-0.00075	-0.00076
5.5	-0.00075	-0.00079	-0.00083	-0.00085	-0.00087	-0.00088
5.75	-0.00081	-0.00087	-0.00091	-0.00094	-0.00096	-0.00098
6	-0.00087	-0.00094	-0.00098	-0.00102	-0.00105	-0.00107

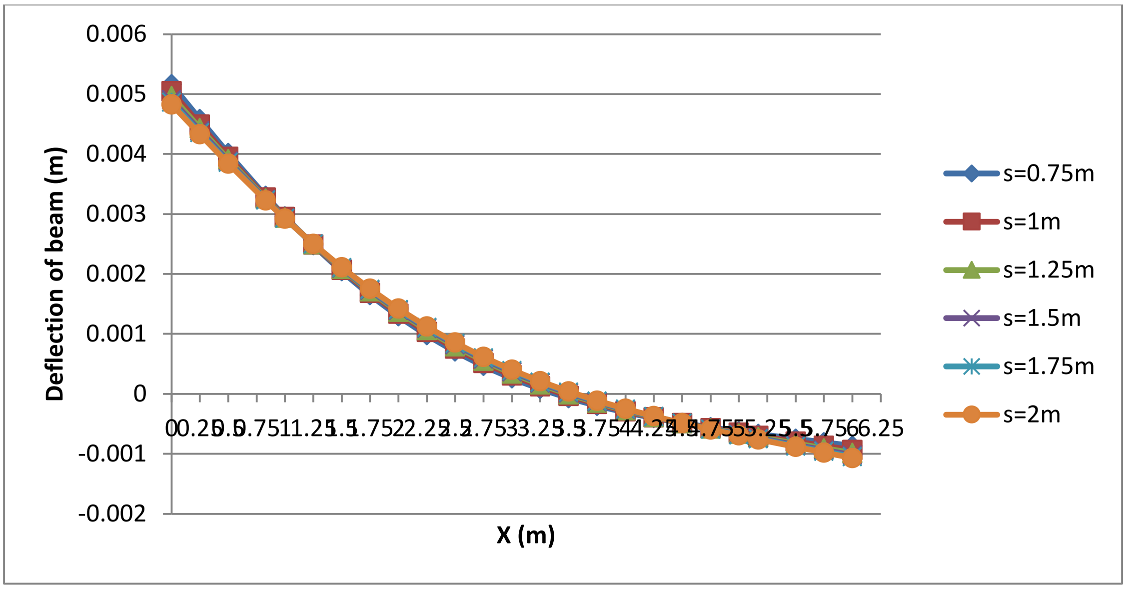

Fig. A.8. Beam deflection (m) vs X (m) at Different flange width for soil swell.

Table A.7. Beam deflection (m) vs X (m) at Different flange depth for soil swell

P=66.67KPA	E=29GPA	L=6m				
x	t=0.1m	t=0.15m	t=0.2m	t=0.25m	t=0.3m	t=0.35m
0	0.004908	0.004606	0.004389	0.004228	0.004108	0.004018
0.25	0.004391	0.004163	0.003998	0.003875	0.003783	0.003714
0.5	0.003885	0.003728	0.003613	0.003526	0.003461	0.003413
0.83	0.003246	0.003174	0.003119	0.003077	0.003045	0.003021
1	0.002934	0.0029	0.002874	0.002853	0.002836	0.002824
1.25	0.0025	0.002517	0.002526	0.002532	0.002536	0.002539
1.5	0.002098	0.002156	0.002196	0.002225	0.002246	0.002262
1.75	0.001729	0.00182	0.001884	0.001932	0.001967	0.001993
2	0.001394	0.001509	0.001591	0.001653	0.001698	0.001733
2.25	0.001091	0.001222	0.001317	0.001388	0.001441	0.001481
2.5	0.000821	0.000959	0.00106	0.001137	0.001194	0.001237
2.75	0.000581	0.000719	0.000822	0.000899	0.000957	0.001001
3	0.000368	0.0005	0.000599	0.000674	0.000731	0.000773
3.25	0.00018	0.000301	0.000391	0.00046	0.000513	0.000552
3.5	1.44E-05	0.000118	0.000197	0.000257	0.000303	0.000338
3.75	-0.00013	-4.9E-05	1.39E-05	6.28E-05	0.0001	0.000129
4	-0.00026	-0.0002	-0.00016	-0.00012	-9.6E-05	-7.5E-05
4.25	-0.00038	-0.00035	-0.00032	-0.0003	-0.00029	-0.00028
4.5	-0.00049	-0.00048	-0.00048	-0.00048	-0.00047	-0.00047
4.75	-0.00058	-0.00061	-0.00063	-0.00065	-0.00066	-0.00067
5	-0.00068	-0.00074	-0.00078	-0.00081	-0.00084	-0.00086
5.17	-0.00074	-0.00082	-0.00088	-0.00093	-0.00096	-0.00099
5.5	-0.00085	-0.00098	-0.00107	-0.00114	-0.0012	-0.00124
5.75	-0.00094	-0.0011	-0.00122	-0.00131	-0.00138	-0.00143
6	-0.00102	-0.00121	-0.00136	-0.00147	-0.00156	-0.00162

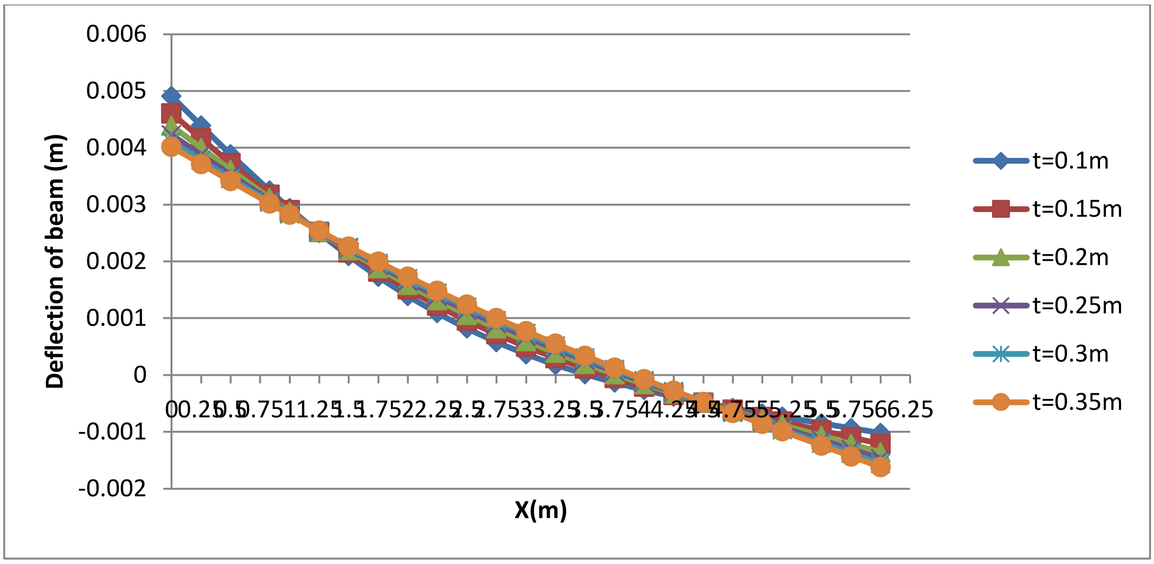

Fig. A.9. Beam deflection (m) vs X (m) at Different flange depth for soil swell.

Table A.8. Beam deflection (m) vs X (m) at Different beam width for soil swell.

P=66.67KPA	E=29GPA	L=6m					
x	b=0.15m	b=0.2m	b=0.25m	b=0.3m	b=0.35m	b=0.4m	b=0.45m
0	0.009473	0.007199	0.005827	0.004908	0.004248	0.003752	0.003363
0.25	0.008524	0.006464	0.005222	0.004391	0.003795	0.003346	0.002996
0.5	0.007592	0.005743	0.00463	0.003885	0.003351	0.00295	0.002637
0.83	0.006411	0.004831	0.00388	0.003246	0.002792	0.002451	0.002185
1	0.005831	0.004384	0.003514	0.002934	0.002519	0.002208	0.001965
1.25	0.00502	0.003759	0.003004	0.0025	0.00214	0.00187	0.001661
1.5	0.004262	0.003178	0.002529	0.002098	0.00179	0.001559	0.00138
1.75	0.003561	0.002642	0.002093	0.001729	0.001469	0.001275	0.001124
2	0.002918	0.002152	0.001696	0.001394	0.001179	0.001018	0.000894
2.25	0.002331	0.001707	0.001337	0.001091	0.000918	0.000788	0.000688
2.5	0.001798	0.001306	0.001014	0.000821	0.000685	0.000584	0.000506
2.75	0.001318	0.000945	0.000725	0.000581	0.000479	0.000404	0.000346
3	0.000885	0.000622	0.000468	0.000368	0.000298	0.000246	0.000206
3.25	0.000495	0.000334	0.000241	0.00018	0.000138	0.000108	8.5E-05
3.5	0.000145	7.64E-05	3.82E-05	1.44E-05	-1.4E-06	-1.2E-05	-2E-05
3.75	-0.00017	-0.00015	-0.00014	-0.00013	-0.00012	-0.00012	-0.00011
4	-0.00046	-0.00036	-0.0003	-0.00026	-0.00023	-0.00021	-0.00019
4.25	-0.00072	-0.00055	-0.00045	-0.00038	-0.00033	-0.00029	-0.00026
4.5	-0.00097	-0.00073	-0.00058	-0.00049	-0.00042	-0.00036	-0.00032
4.75	-0.0012	-0.00089	-0.00071	-0.00058	-0.0005	-0.00043	-0.00038
5	-0.00142	-0.00105	-0.00082	-0.00068	-0.00057	-0.00049	-0.00043
5.17	-0.00157	-0.00115	-0.0009	-0.00074	-0.00062	-0.00053	-0.00047
5.5	-0.00184	-0.00134	-0.00105	-0.00085	-0.00071	-0.00061	-0.00053
5.75	-0.00205	-0.00149	-0.00115	-0.00094	-0.00078	-0.00067	-0.00058
6	-0.00226	-0.00163	-0.00126	-0.00102	-0.00085	-0.00072	-0.00062

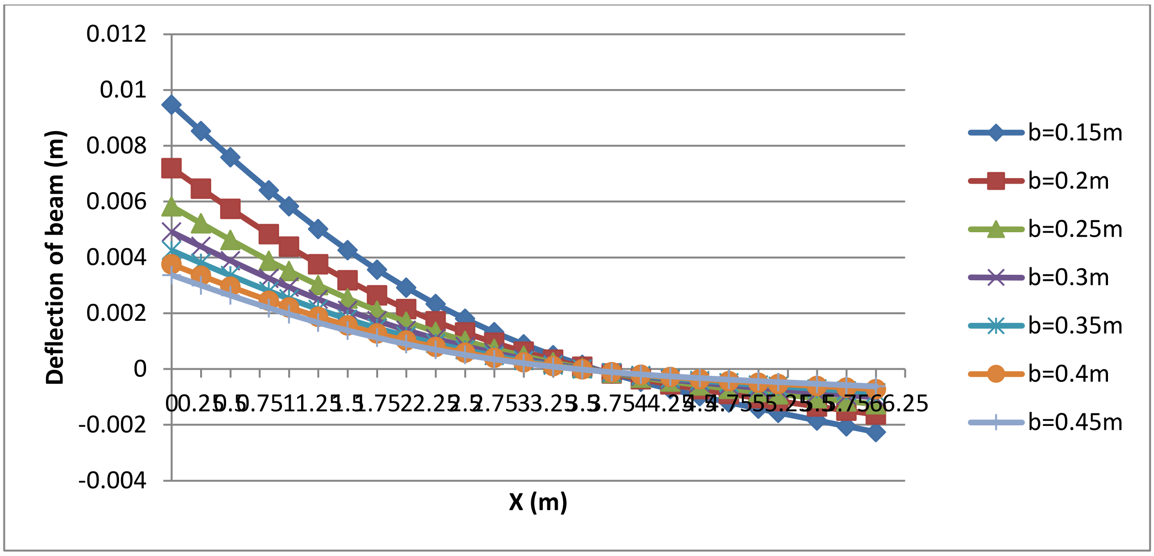

Fig. A.10. Beam deflection (m) vs X (m) at Different beam width for soil swell.

Table A.9. Beam deflection (m) vs X (m) at Different beam depth for soil swell.

P=66.67KPA	E=29GPA	L=6m					
x	h=0.2m	h=0.25m	h=0.3m	h=0.35m	h=0.4m	h=0.45m	h=0.5m
0	0.005932	0.00533	0.004908	0.004611	0.004401	0.004249	0.004137
0.25	0.005149	0.004707	0.004391	0.004167	0.004007	0.003891	0.003806
0.5	0.00439	0.004099	0.003885	0.00373	0.003619	0.003537	0.003477
0.83	0.003455	0.00334	0.003246	0.003175	0.003122	0.003083	0.003053
1	0.00301	0.002973	0.002934	0.002901	0.002875	0.002855	0.00284
1.25	0.002409	0.002469	0.0025	0.002516	0.002526	0.002532	0.002535
1.5	0.001874	0.002011	0.002098	0.002155	0.002194	0.002221	0.002241
1.75	0.001406	0.001598	0.001729	0.001818	0.001881	0.001926	0.001958
2	0.001003	0.001232	0.001394	0.001507	0.001587	0.001645	0.001687
2.25	0.000663	0.000912	0.001091	0.00122	0.001312	0.001378	0.001428
2.5	0.00038	0.000633	0.000821	0.000957	0.001055	0.001127	0.00118
2.75	0.000151	0.000395	0.000581	0.000717	0.000816	0.000889	0.000943
3	-3.2E-05	0.000193	0.000368	0.000498	0.000594	0.000664	0.000716
3.25	-0.00017	2.28E-05	0.00018	0.000299	0.000386	0.000451	0.0005
3.5	-0.00028	-0.00012	1.44E-05	0.000116	0.000193	0.000249	0.000291
3.75	-0.00036	-0.00024	-0.00013	-5.1E-05	1.06E-05	5.65E-05	9.09E-05
4	-0.00041	-0.00033	-0.00026	-0.0002	-0.00016	-0.00013	-0.0001
4.25	-0.00045	-0.00042	-0.00038	-0.00035	-0.00032	-0.00031	-0.00029
4.5	-0.00047	-0.00048	-0.00049	-0.00048	-0.00048	-0.00048	-0.00048
4.75	-0.00048	-0.00054	-0.00058	-0.00061	-0.00063	-0.00065	-0.00066
5	-0.00048	-0.00059	-0.00068	-0.00074	-0.00078	-0.00081	-0.00083
5.17	-0.00048	-0.00063	-0.00074	-0.00082	-0.00088	-0.00092	-0.00095
5.5	-0.00047	-0.00068	-0.00085	-0.00097	-0.00107	-0.00113	-0.00119
5.75	-0.00047	-0.00073	-0.00094	-0.00109	-0.00121	-0.0013	-0.00136
6	-0.00046	-0.00077	-0.00102	-0.00121	-0.00135	-0.00146	-0.00153

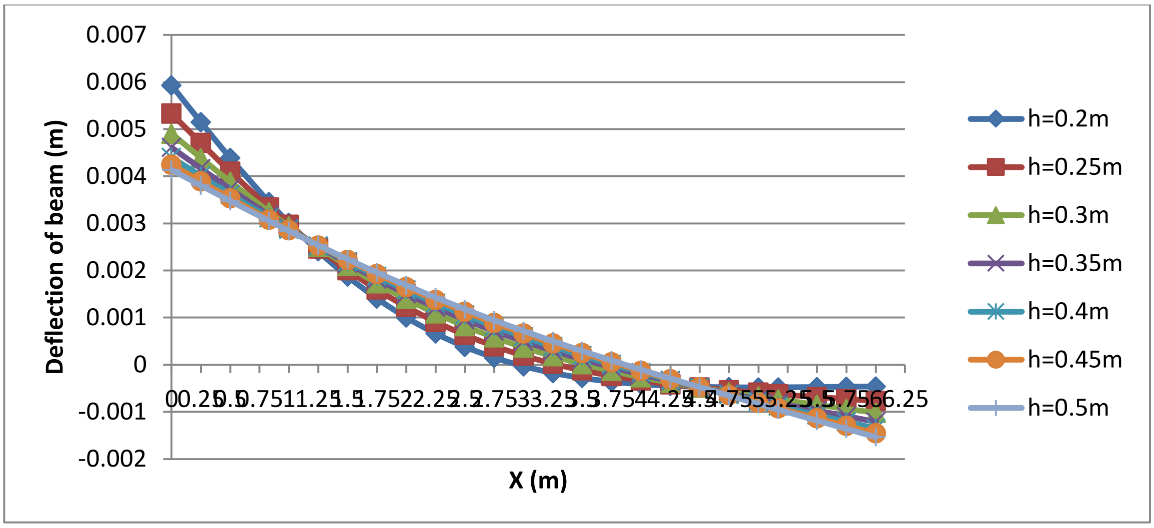

Fig. A.11. Beam deflection (m) vs X (m) at Different beam depth for soil swell.

Concentrated load at 4m from one end of beam for soil heave

Table A.10. Beam deflection (m) vs X (m) at Different flange width for soil swell.

x	S=0.75m	S=1m	S=1.25m	S=1.5m	S=1.75m	S=2m
0	-0.00035	-0.00032	-0.00031	-0.0003	-0.00029	-0.00028
0.25	-0.00021	-0.00019	-0.00017	-0.00016	-0.00016	-0.00015
0.5	-6.9E-05	-5.2E-05	-4E-05	-3.2E-05	-2.5E-05	-2E-05
0.83	0.000114	0.000127	0.000136	0.000143	0.000148	0.000152
1	0.000209	0.00022	0.000227	0.000233	0.000237	0.00024
1.25	0.000349	0.000357	0.000362	0.000366	0.000368	0.000371
1.5	0.00049	0.000494	0.000497	0.000499	0.0005	0.000501
1.75	0.000632	0.000633	0.000633	0.000633	0.000632	0.000632
2	0.000774	0.000771	0.000768	0.000766	0.000764	0.000763
2.25	0.000916	0.000908	0.000903	0.000898	0.000895	0.000892
2.5	0.001056	0.001044	0.001035	0.001029	0.001024	0.00102
2.75	0.001191	0.001175	0.001163	0.001155	0.001148	0.001143
3	0.00132	0.0013	0.001286	0.001275	0.001267	0.001261
3.25	0.001438	0.001415	0.001398	0.001386	0.001377	0.00137
3.5	0.001542	0.001516	0.001498	0.001485	0.001475	0.001467
3.75	0.001625	0.001598	0.00158	0.001567	0.001557	0.001549
4	0.001681	0.001656	0.001639	0.001627	0.001618	0.001611
4.25	0.001705	0.001685	0.001672	0.001662	0.001655	0.001649
4.5	0.001703	0.00169	0.001682	0.001676	0.001671	0.001667
4.75	0.001682	0.001678	0.001676	0.001674	0.001672	0.001671
5	0.001648	0.001654	0.001658	0.001661	0.001664	0.001666
5.17	0.00162	0.001633	0.001642	0.001648	0.001654	0.001658
5.5	0.001558	0.001586	0.001605	0.001618	0.001629	0.001637
5.75	0.001508	0.001548	0.001574	0.001593	0.001608	0.00162
6	0.001458	0.001509	0.001543	0.001568	0.001587	0.001602

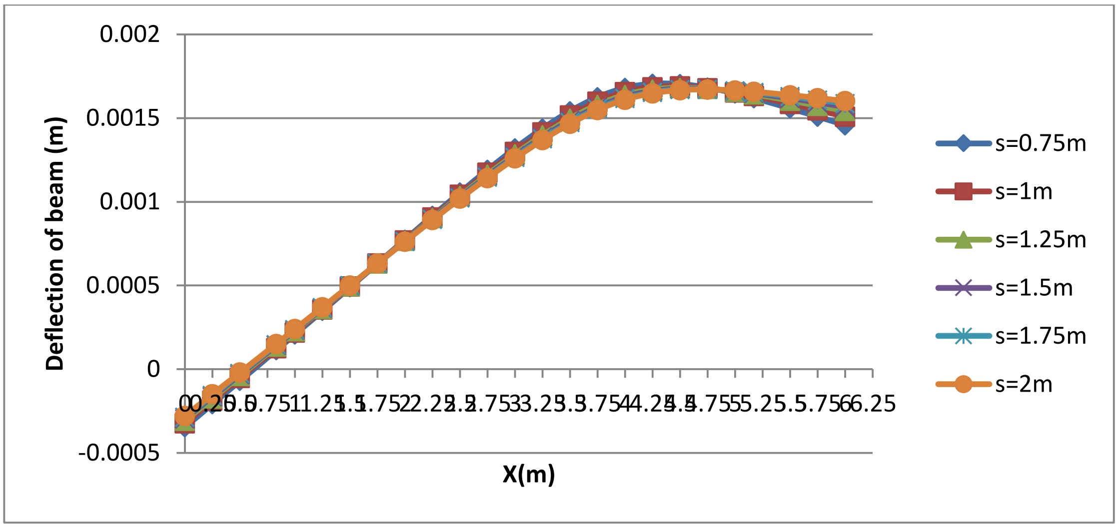

Fig. A.12. Beam deflection (m) vs X (m) at Different flange width for soil swell.

Table A.11. Beam deflection (m) vs X (m) at Different flange thickness for soil swell.

x	t=0.1m	t=0.15m	t=0.2m	t=0.25m	t=0.3m	t=0.35m
0	-0.0003	-0.00023	-0.00018	-0.00014	-0.00011	-8.5E-05
0.25	-0.00016	-0.00011	-6.4E-05	-3E-05	-4.1E-06	1.56E-05
0.5	-3.2E-05	1.56E-05	5.14E-05	7.9E-05	0.0001	0.000116
0.83	0.000143	0.000178	0.000203	0.000223	0.000238	0.000249
1	0.000233	0.000261	0.000282	0.000297	0.000308	0.000317
1.25	0.000366	0.000384	0.000397	0.000406	0.000413	0.000417
1.5	0.000499	0.000507	0.000512	0.000515	0.000517	0.000518
1.75	0.000633	0.00063	0.000627	0.000623	0.00062	0.000618
2	0.000766	0.000753	0.000741	0.000731	0.000724	0.000717
2.25	0.000898	0.000874	0.000854	0.000838	0.000826	0.000816
2.5	0.001029	0.000993	0.000965	0.000944	0.000927	0.000914
2.75	0.001155	0.001109	0.001074	0.001047	0.001027	0.001011
3	0.001275	0.00122	0.001178	0.001147	0.001123	0.001105
3.25	0.001386	0.001324	0.001278	0.001243	0.001217	0.001197
3.5	0.001485	0.001418	0.001369	0.001333	0.001306	0.001286
3.75	0.001567	0.0015	0.001452	0.001416	0.00139	0.00137
4	0.001627	0.001566	0.001522	0.00149	0.001467	0.001449
4.25	0.001662	0.001613	0.001579	0.001554	0.001535	0.001522
4.5	0.001676	0.001645	0.001624	0.001609	0.001598	0.00159
4.75	0.001674	0.001665	0.00166	0.001657	0.001655	0.001654
5	0.001661	0.001678	0.001691	0.001701	0.001709	0.001715
5.17	0.001648	0.001684	0.00171	0.001729	0.001744	0.001756
5.5	0.001618	0.001691	0.001743	0.001782	0.001811	0.001833
5.75	0.001593	0.001694	0.001767	0.001821	0.001861	0.001891
6	0.001568	0.001697	0.00179	0.001859	0.001911	0.001949

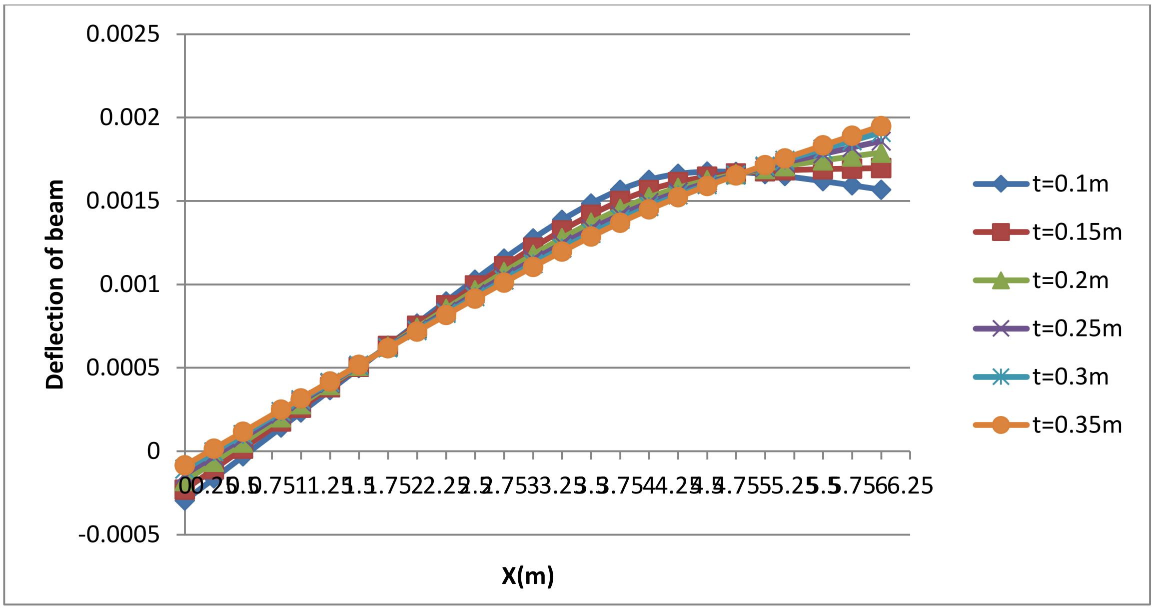

Fig. A.13. Beam deflection (m) vs X (m) at Different flange thickness for soil swell.

Table A.12. Beam deflection (m) vs X (m) at Different flange thickness for soil swell.

x	b=0.15m	b=0.2m	b=0.25m	b=0.3m	b=0.35m	b=0.4m	b=0.45m
0	-0.00052	-0.00041	-0.00034	-0.0003	-0.00026	-0.00024	-0.00022
0.25	-0.00026	-0.00022	-0.00018	-0.00016	-0.00015	-0.00014	-0.00013
0.5	-1E-05	-2.3E-05	-2.8E-05	-3.2E-05	-3.3E-05	-3.4E-05	-3.5E-05
0.83	0.000325	0.000233	0.000178	0.000143	0.000118	9.9E-05	8.48E-05
1	0.000498	0.000364	0.000285	0.000233	0.000196	0.000168	0.000147
1.25	0.000752	0.000559	0.000443	0.000366	0.000311	0.00027	0.000238
1.5	0.001008	0.000753	0.000601	0.000499	0.000426	0.000372	0.00033
1.75	0.001263	0.000948	0.000759	0.000633	0.000542	0.000475	0.000422
2	0.001518	0.001142	0.000917	0.000766	0.000658	0.000577	0.000514
2.25	0.00177	0.001335	0.001073	0.000898	0.000773	0.000679	0.000606
2.5	0.002018	0.001524	0.001227	0.001029	0.000886	0.000779	0.000696
2.75	0.002258	0.001708	0.001377	0.001155	0.000996	0.000877	0.000783
3	0.002488	0.001883	0.001519	0.001275	0.0011	0.000969	0.000866
3.25	0.002702	0.002046	0.001651	0.001386	0.001197	0.001054	0.000943
3.5	0.002894	0.002192	0.001768	0.001485	0.001282	0.00113	0.00101
3.75	0.003058	0.002314	0.001867	0.001567	0.001353	0.001191	0.001065
4	0.003184	0.002407	0.00194	0.001627	0.001403	0.001235	0.001104
4.25	0.003268	0.002466	0.001984	0.001662	0.001432	0.001258	0.001123
4.5	0.003316	0.002497	0.002004	0.001676	0.001441	0.001264	0.001127
4.75	0.003338	0.002506	0.002007	0.001674	0.001436	0.001258	0.001119
5	0.003341	0.002501	0.001997	0.001661	0.001422	0.001242	0.001103
5.17	0.003337	0.002492	0.001985	0.001648	0.001408	0.001228	0.001089
5.5	0.003319	0.002466	0.001957	0.001618	0.001377	0.001197	0.001057
5.75	0.003301	0.002444	0.001933	0.001593	0.001352	0.001171	0.001032
6	0.003282	0.002421	0.001908	0.001568	0.001326	0.001145	0.001006

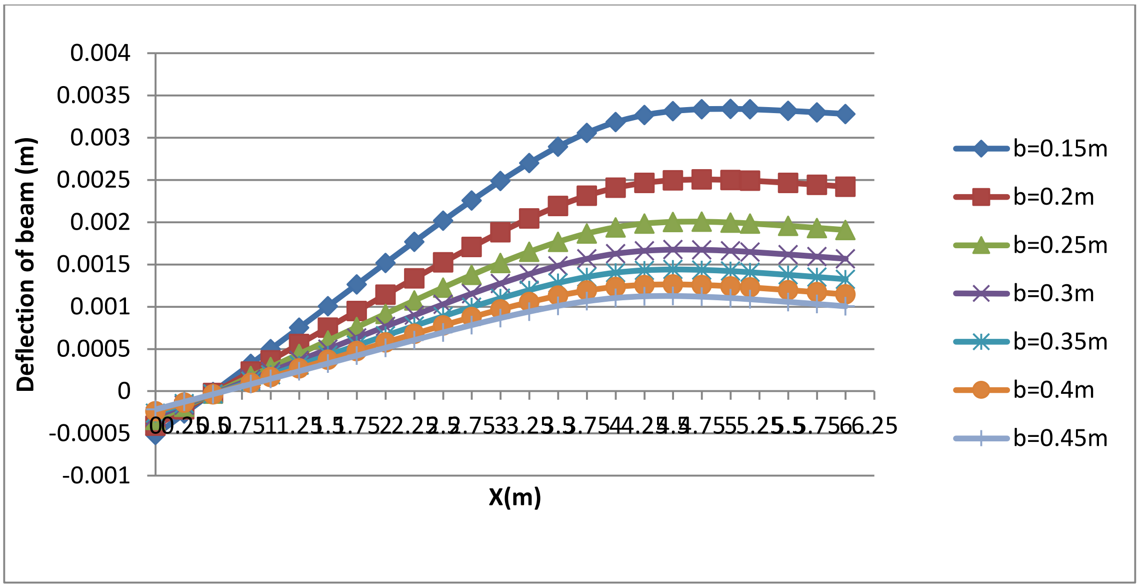

Fig. A.14. Beam deflection (m) vs X (m) at Different beam width for soil swell.

Table A.13. Beam deflection (m) vs X (m) at Different beam depth for soil swell

x	h=0.2m	h=0.25m	h=0.3m	h=0.35m	h=0.4m	h=0.45m	h=0.5m
0	-0.00047	-0.00038	-0.0003	-0.00023	-0.00018	-0.00014	-0.00012
0.25	-0.00032	-0.00023	-0.00016	-0.00011	-6.6E-05	-3.4E-05	-1.1E-05
0.5	-0.00016	-9.2E-05	-3.2E-05	1.48E-05	4.96E-05	7.54E-05	9.48E-05
0.83	3.49E-05	9.6E-05	0.000143	0.000177	0.000202	0.00022	0.000234
1	0.000139	0.000194	0.000233	0.000261	0.000281	0.000295	0.000306
1.25	0.000295	0.000338	0.000366	0.000384	0.000396	0.000405	0.000411
1.5	0.000454	0.000484	0.000499	0.000507	0.000512	0.000514	0.000516
1.75	0.000616	0.000631	0.000633	0.00063	0.000627	0.000624	0.000621
2	0.000782	0.000778	0.000766	0.000753	0.000742	0.000733	0.000726
2.25	0.00095	0.000926	0.000898	0.000874	0.000855	0.00084	0.000829
2.5	0.001119	0.001072	0.001029	0.000994	0.000967	0.000947	0.000931
2.75	0.001285	0.001214	0.001155	0.00111	0.001076	0.001051	0.001032
3	0.001443	0.001349	0.001275	0.001221	0.001181	0.001151	0.001129
3.25	0.001588	0.001472	0.001386	0.001325	0.00128	0.001247	0.001223
3.5	0.001711	0.001579	0.001485	0.001419	0.001372	0.001338	0.001313
3.75	0.001802	0.001663	0.001567	0.001501	0.001454	0.001421	0.001396
4	0.00185	0.001717	0.001627	0.001567	0.001524	0.001494	0.001472
4.25	0.001845	0.001734	0.001662	0.001614	0.00158	0.001557	0.00154
4.5	0.001796	0.001722	0.001676	0.001645	0.001625	0.001611	0.0016
4.75	0.001716	0.001688	0.001674	0.001666	0.001661	0.001658	0.001656
5	0.001615	0.00164	0.001661	0.001678	0.001691	0.0017	0.001707
5.17	0.001538	0.001601	0.001648	0.001683	0.001708	0.001727	0.001741
5.5	0.001379	0.001518	0.001618	0.001689	0.00174	0.001777	0.001804
5.75	0.001254	0.001453	0.001593	0.001692	0.001763	0.001814	0.001851
6	0.001129	0.001386	0.001568	0.001695	0.001785	0.00185	0.001898

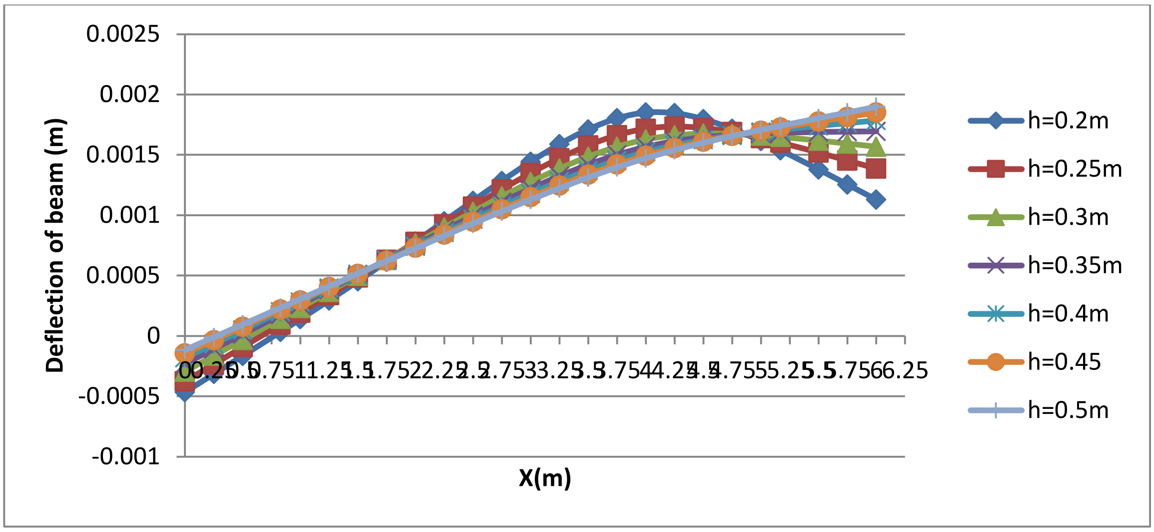

Fig. A.15. Beam deflection (m) vs X (m) at Different beam depth for soil swell.

Uniformly distributed load on the beam for soil swell.

Table A.14. Beam deflection (m) vs X (m) at Different flange width for soil swell

x	S=0.75m	S=1m	S=1.25m	S=1.5m	S=1.75m	S=2m
0	0	0	0	0	0	0
0.25	3.79E-05	4.87E-05	5.92E-05	6.94E-05	7.95E-05	8.95E-05
0.5	7.49E-05	9.62E-05	0.000117	0.000137	0.000157	0.000177
0.83	0.000121	0.000155	0.000189	0.000222	0.000254	0.000286
1	0.000143	0.000184	0.000223	0.000262	0.0003	0.000338
1.25	0.000173	0.000222	0.00027	0.000317	0.000363	0.000409
1.5	0.000199	0.000256	0.000311	0.000366	0.000419	0.000472
1.75	0.000222	0.000285	0.000347	0.000408	0.000468	0.000527
2	0.000241	0.00031	0.000377	0.000443	0.000508	0.000573
2.25	0.000255	0.000329	0.000401	0.000471	0.00054	0.000608
2.5	0.000266	0.000343	0.000417	0.000491	0.000563	0.000634
2.75	0.000272	0.000351	0.000428	0.000503	0.000577	0.00065
3	0.000275	0.000354	0.000431	0.000507	0.000581	0.000655
3.25	0.000272	0.000351	0.000428	0.000503	0.000577	0.00065
3.5	0.000266	0.000343	0.000417	0.000491	0.000563	0.000634
3.75	0.000255	0.000329	0.000401	0.000471	0.00054	0.000608
4	0.000241	0.00031	0.000377	0.000443	0.000508	0.000573
4.25	0.000222	0.000285	0.000347	0.000408	0.000468	0.000527
4.5	0.000199	0.000256	0.000311	0.000366	0.000419	0.000472
4.75	0.000173	0.000222	0.00027	0.000317	0.000363	0.000409
5	0.000143	0.000184	0.000223	0.000262	0.0003	0.000338
5.17	0.000121	0.000155	0.000189	0.000222	0.000254	0.000286
5.5	7.49E-05	9.62E-05	0.000117	0.000137	0.000157	0.000177
5.75	3.79E-05	4.87E-05	5.92E-05	6.94E-05	7.95E-05	8.95E-05
6	0	0	0	0	0	0

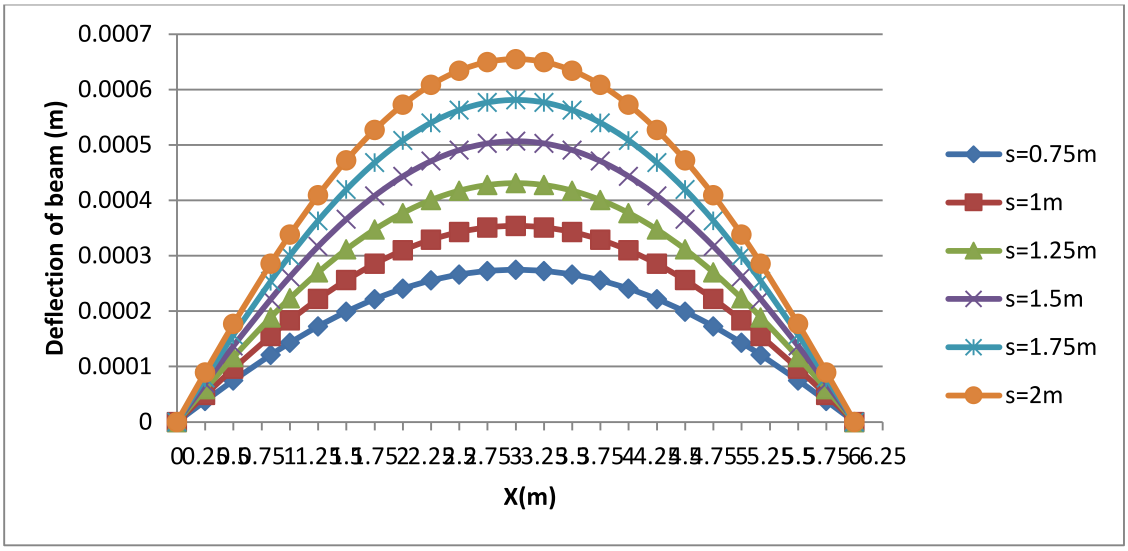

Fig. A.16. Beam deflection (m) vs X (m) at Different flange width for soil swell.

Table A.15. Beam deflection (m) vs X (m) at Different flange thickness for soil swell

x	t=0.1m	t=0.15m	t=0.2m	t=0.25m	t=0.3m	t=0.35m
0	0	0	0	0	0	0
0.25	6.94E-05	5.98E-05	5.1E-05	4.3E-05	3.58E-05	2.97E-05
0.5	0.000137	0.000118	0.000101	8.5E-05	7.09E-05	5.87E-05
0.83	0.000222	0.000191	0.000163	0.000138	0.000115	9.51E-05
1	0.000262	0.000226	0.000193	0.000163	0.000136	0.000113
1.25	0.000317	0.000274	0.000234	0.000198	0.000165	0.000137
1.5	0.000366	0.000317	0.000271	0.000229	0.000191	0.000158
1.75	0.000408	0.000354	0.000303	0.000256	0.000214	0.000177
2	0.000443	0.000385	0.00033	0.000278	0.000233	0.000193
2.25	0.000471	0.000409	0.000351	0.000296	0.000248	0.000205
2.5	0.000491	0.000427	0.000366	0.000309	0.000258	0.000214
2.75	0.000503	0.000437	0.000375	0.000317	0.000265	0.00022
3	0.000507	0.000441	0.000378	0.00032	0.000267	0.000222
3.25	0.000503	0.000437	0.000375	0.000317	0.000265	0.00022
3.5	0.000491	0.000427	0.000366	0.000309	0.000258	0.000214
3.75	0.000471	0.000409	0.000351	0.000296	0.000248	0.000205
4	0.000443	0.000385	0.00033	0.000278	0.000233	0.000193
4.25	0.000408	0.000354	0.000303	0.000256	0.000214	0.000177
4.5	0.000366	0.000317	0.000271	0.000229	0.000191	0.000158
4.75	0.000317	0.000274	0.000234	0.000198	0.000165	0.000137
5	0.000262	0.000226	0.000193	0.000163	0.000136	0.000113
5.17	0.000222	0.000191	0.000163	0.000138	0.000115	9.51E-05
5.5	0.000137	0.000118	0.000101	8.5E-05	7.09E-05	5.87E-05
5.75	6.94E-05	5.98E-05	5.1E-05	4.3E-05	3.58E-05	2.97E-05
6	0	0	0	0	0	0

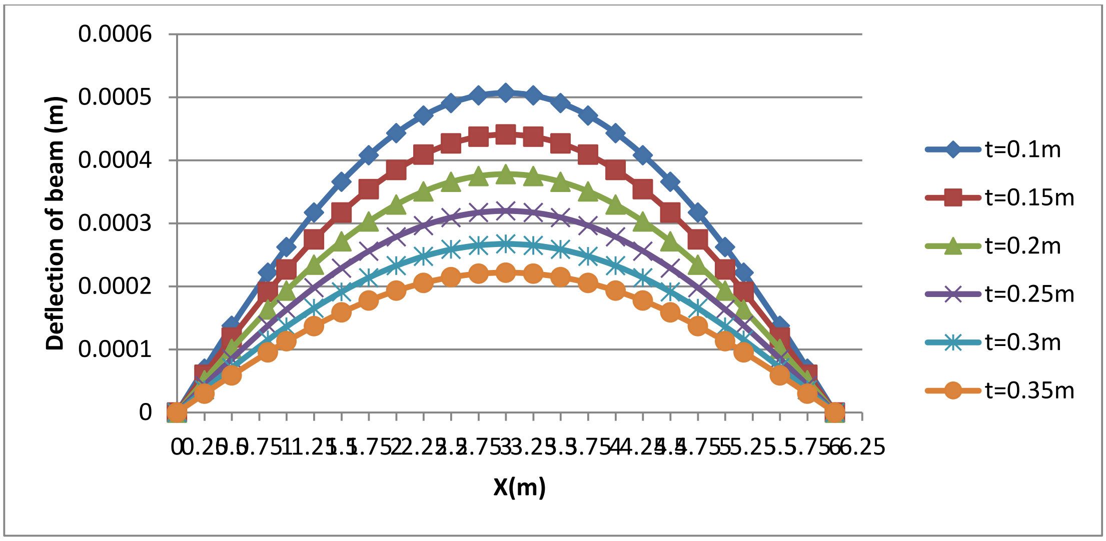

Fig. A.17. Beam deflection (m) vs X (m) at Different flange thickness for soil swell.

Table A.16. Beam deflection (m) vs X (m) at Different beam width for soil swell

x	b=0.15m	b=0.2m	b=0.25m	b=0.3m	b=0.35m	b=0.4m	b=0.45m
0	0	0	0	0	0	0	0
0.25	0.000129	9.94E-05	8.16E-05	6.94E-05	6.07E-05	5.4E-05	4.87E-05
0.5	0.000254	0.000196	0.000161	0.000137	0.00012	0.000107	9.62E-05
0.83	0.000411	0.000317	0.00026	0.000222	0.000193	0.000172	0.000155
1	0.000486	0.000376	0.000308	0.000262	0.000229	0.000203	0.000184
1.25	0.000588	0.000454	0.000372	0.000317	0.000277	0.000246	0.000222
1.5	0.00068	0.000525	0.00043	0.000366	0.000319	0.000284	0.000256
1.75	0.000759	0.000586	0.00048	0.000408	0.000356	0.000317	0.000285
2	0.000824	0.000636	0.000521	0.000443	0.000387	0.000344	0.00031
2.25	0.000876	0.000676	0.000554	0.000471	0.000411	0.000365	0.000329
2.5	0.000913	0.000705	0.000577	0.000491	0.000428	0.00038	0.000343
2.75	0.000936	0.000722	0.000591	0.000503	0.000438	0.00039	0.000351
3	0.000944	0.000728	0.000596	0.000507	0.000442	0.000393	0.000354
3.25	0.000936	0.000722	0.000591	0.000503	0.000438	0.00039	0.000351
3.5	0.000913	0.000705	0.000577	0.000491	0.000428	0.00038	0.000343
3.75	0.000876	0.000676	0.000554	0.000471	0.000411	0.000365	0.000329
4	0.000824	0.000636	0.000521	0.000443	0.000387	0.000344	0.00031
4.25	0.000759	0.000586	0.00048	0.000408	0.000356	0.000317	0.000285
4.5	0.00068	0.000525	0.00043	0.000366	0.000319	0.000284	0.000256
4.75	0.000588	0.000454	0.000372	0.000317	0.000277	0.000246	0.000222
5	0.000486	0.000376	0.000308	0.000262	0.000229	0.000203	0.000184
5.17	0.000411	0.000317	0.00026	0.000222	0.000193	0.000172	0.000155
5.5	0.000254	0.000196	0.000161	0.000137	0.00012	0.000107	9.62E-05
5.75	0.000129	9.94E-05	8.16E-05	6.94E-05	6.07E-05	5.4E-05	4.87E-05
6	0	0	0	0	0	0	0

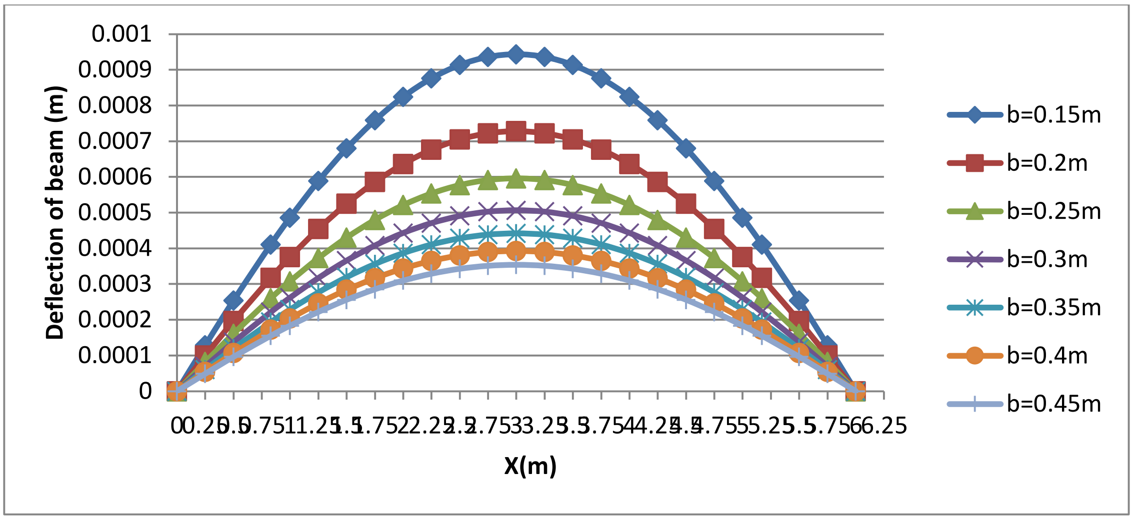

Fig. A.18. Beam deflection (m) vs X (m) at Different beam width for soil swell.

Table A17. Beam deflection (m) vs X (m) at Different beam depth for soil swell

x	h=0.2m	h=0.25m	h=0.3m	h=0.35m	h=0.4m	h=0.45m	h=0.5m
0	0	0	0	0	0	0	0
0.25	9.02E-05	7.96E-05	6.94E-05	6E-05	5.15E-05	4.41E-05	3.77E-05
0.5	0.000178	0.000157	0.000137	0.000119	0.000102	8.72E-05	7.46E-05
0.83	0.000285	0.000253	0.000222	0.000192	0.000165	0.000141	0.000121
1	0.000336	0.000299	0.000262	0.000227	0.000195	0.000167	0.000143
1.25	0.000404	0.000361	0.000317	0.000275	0.000237	0.000203	0.000174
1.5	0.000464	0.000416	0.000366	0.000318	0.000274	0.000235	0.000201
1.75	0.000516	0.000463	0.000408	0.000355	0.000306	0.000262	0.000225
2	0.000558	0.000502	0.000443	0.000386	0.000333	0.000286	0.000245
2.25	0.00059	0.000533	0.000471	0.00041	0.000354	0.000304	0.00026
2.5	0.000614	0.000554	0.000491	0.000428	0.000369	0.000317	0.000272
2.75	0.000628	0.000568	0.000503	0.000438	0.000378	0.000325	0.000279
3	0.000632	0.000572	0.000507	0.000442	0.000382	0.000328	0.000281
3.25	0.000628	0.000568	0.000503	0.000438	0.000378	0.000325	0.000279
3.5	0.000614	0.000554	0.000491	0.000428	0.000369	0.000317	0.000272
3.75	0.00059	0.000533	0.000471	0.00041	0.000354	0.000304	0.00026
4	0.000558	0.000502	0.000443	0.000386	0.000333	0.000286	0.000245
4.25	0.000516	0.000463	0.000408	0.000355	0.000306	0.000262	0.000225
4.5	0.000464	0.000416	0.000366	0.000318	0.000274	0.000235	0.000201
4.75	0.000404	0.000361	0.000317	0.000275	0.000237	0.000203	0.000174
5	0.000336	0.000299	0.000262	0.000227	0.000195	0.000167	0.000143
5.17	0.000285	0.000253	0.000222	0.000192	0.000165	0.000141	0.000121
5.5	0.000178	0.000157	0.000137	0.000119	0.000102	8.72E-05	7.46E-05
5.75	9.02E-05	7.96E-05	6.94E-05	6E-05	5.15E-05	4.41E-05	3.77E-05
6	0	0	0	0	0	0	0

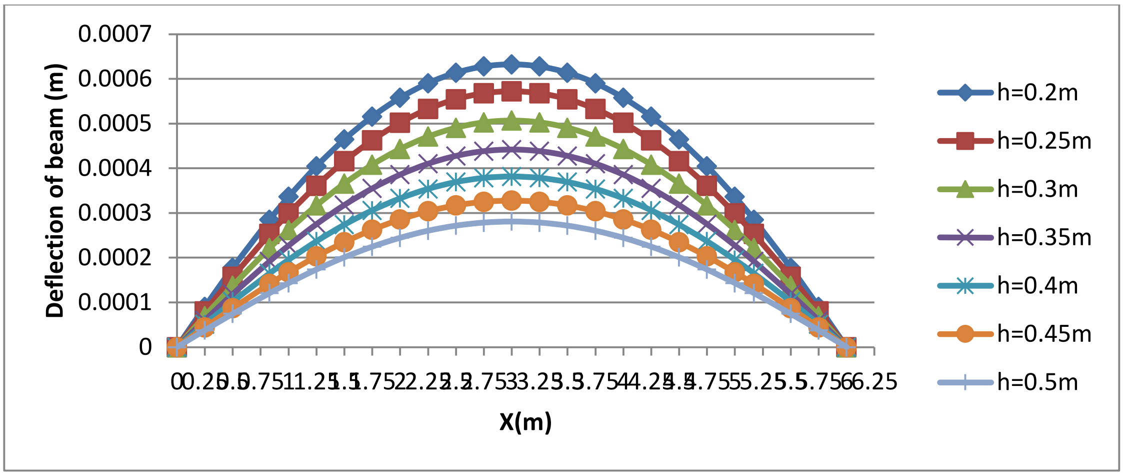

Fig. A.19. Beam deflection (m) vs X (m) at Different beam width for soil swell.

Heave load on beam for soil swell.

Table A.18. Beam deflection (m) vs X (m) at Different flange width for soil swell.

x	S=0.75m	S=1m	S=1.25m	S=1.5m	S=1.75m	S=2m
0	-0.00929	-0.00922	-0.00917	-0.00913	-0.0091	-0.00908
0.25	-0.00866	-0.0086	-0.00856	-0.00852	-0.0085	-0.00848
0.5	-0.00804	-0.00798	-0.00794	-0.00792	-0.0079	-0.00788
0.83	-0.00718	-0.00714	-0.00712	-0.0071	-0.00708	-0.00707
1	-0.00673	-0.0067	-0.00668	-0.00666	-0.00665	-0.00664
1.25	-0.00604	-0.00603	-0.00602	-0.00601	-0.00601	-0.00601
1.5	-0.00536	-0.00536	-0.00537	-0.00537	-0.00537	-0.00537
1.75	-0.00469	-0.00471	-0.00472	-0.00473	-0.00474	-0.00475
2	-0.00405	-0.00408	-0.0041	-0.00412	-0.00413	-0.00414
2.25	-0.00344	-0.00348	-0.00351	-0.00354	-0.00355	-0.00357
2.5	-0.00287	-0.00292	-0.00296	-0.00298	-0.003	-0.00302
2.75	-0.00234	-0.0024	-0.00244	-0.00247	-0.00249	-0.0025
3	-0.00186	-0.00191	-0.00195	-0.00198	-0.002	-0.00202
3.25	-0.00141	-0.00147	-0.0015	-0.00153	-0.00155	-0.00157
3.5	-0.001	-0.00105	-0.00109	-0.00111	-0.00113	-0.00115
3.75	-0.00063	-0.00067	-0.0007	-0.00072	-0.00074	-0.00075
4	-0.00029	-0.00032	-0.00034	-0.00036	-0.00037	-0.00038
4.25	3.13E-05	1.05E-05	-3.7E-06	-1.4E-05	-2.2E-05	-2.9E-05
4.5	0.000327	0.000319	0.000314	0.00031	0.000307	0.000305
4.75	0.000605	0.000612	0.000617	0.00062	0.000623	0.000625
5	0.000871	0.000893	0.000908	0.000919	0.000928	0.000935
5.17	0.001047	0.00108	0.001102	0.001119	0.001131	0.001142
5.5	0.001381	0.001435	0.001472	0.001499	0.00152	0.001537
5.75	0.00163	0.001701	0.00175	0.001785	0.001813	0.001835
6	0.00188	0.001967	0.002027	0.00207	0.002104	0.002131

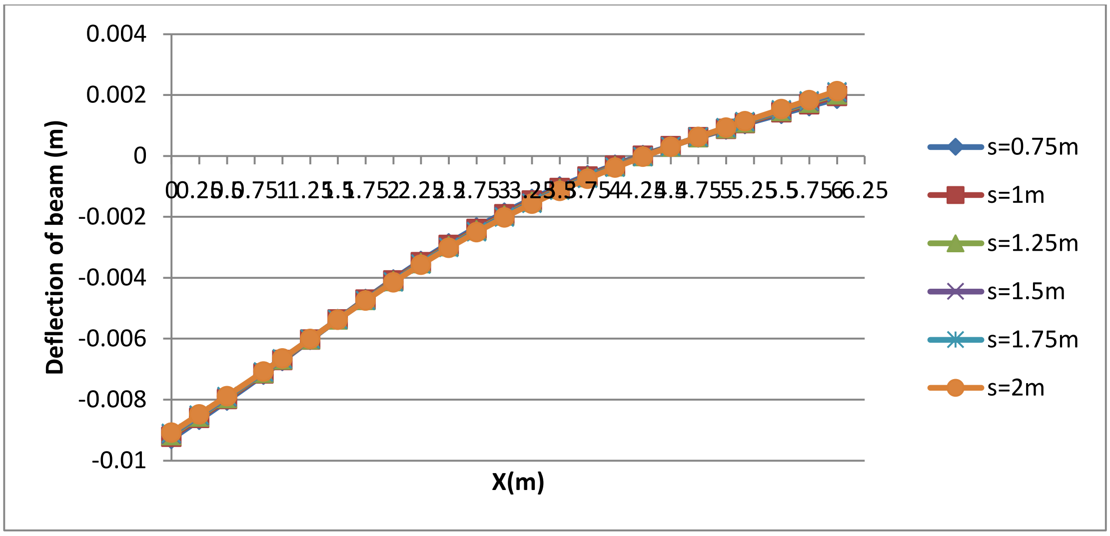

Fig. A.20. Beam deflection (m) vs X (m) at Different flange width for soil swell.

Table A.19. Beam deflection (m) vs X (m) at Different flange thickness for soil swell.

x	t=0.1m	t=0.15m	t=0.2m	t=0.25m	t=0.3m	t=0.35m
0	-0.00913	-0.00893	-0.00877	-0.00865	-0.00857	-0.0085
0.25	-0.00852	-0.00835	-0.00822	-0.00812	-0.00804	-0.00799
0.5	-0.00792	-0.00777	-0.00766	-0.00758	-0.00752	-0.00748
0.83	-0.0071	-0.007	-0.00693	-0.00687	-0.00683	-0.0068
1	-0.00666	-0.00659	-0.00654	-0.0065	-0.00647	-0.00645
1.25	-0.00601	-0.00598	-0.00596	-0.00595	-0.00594	-0.00593
1.5	-0.00537	-0.00538	-0.00539	-0.0054	-0.00541	-0.00541
1.75	-0.00473	-0.00479	-0.00483	-0.00486	-0.00488	-0.0049
2	-0.00412	-0.00421	-0.00427	-0.00432	-0.00436	-0.00439
2.25	-0.00354	-0.00365	-0.00374	-0.0038	-0.00385	-0.00389
2.5	-0.00298	-0.00312	-0.00322	-0.0033	-0.00335	-0.0034
2.75	-0.00247	-0.00261	-0.00272	-0.0028	-0.00287	-0.00291
3	-0.00198	-0.00213	-0.00224	-0.00233	-0.00239	-0.00244
3.25	-0.00153	-0.00168	-0.00178	-0.00186	-0.00193	-0.00197
3.5	-0.00111	-0.00124	-0.00134	-0.00142	-0.00147	-0.00151
3.75	-0.00072	-0.00083	-0.00092	-0.00098	-0.00103	-0.00106
4	-0.00036	-0.00044	-0.00051	-0.00056	-0.00059	-0.00062
4.25	-1.4E-05	-7E-05	-0.00011	-0.00014	-0.00017	-0.00019
4.5	0.00031	0.000289	0.000274	0.000262	0.000253	0.000246
4.75	0.00062	0.000638	0.000651	0.000661	0.000668	0.000673
5	0.000919	0.000978	0.001022	0.001054	0.001079	0.001098
5.17	0.001119	0.001207	0.001271	0.00132	0.001357	0.001385
5.5	0.001499	0.001645	0.001752	0.001833	0.001895	0.001941
5.75	0.001785	0.001975	0.002115	0.002221	0.002301	0.002362

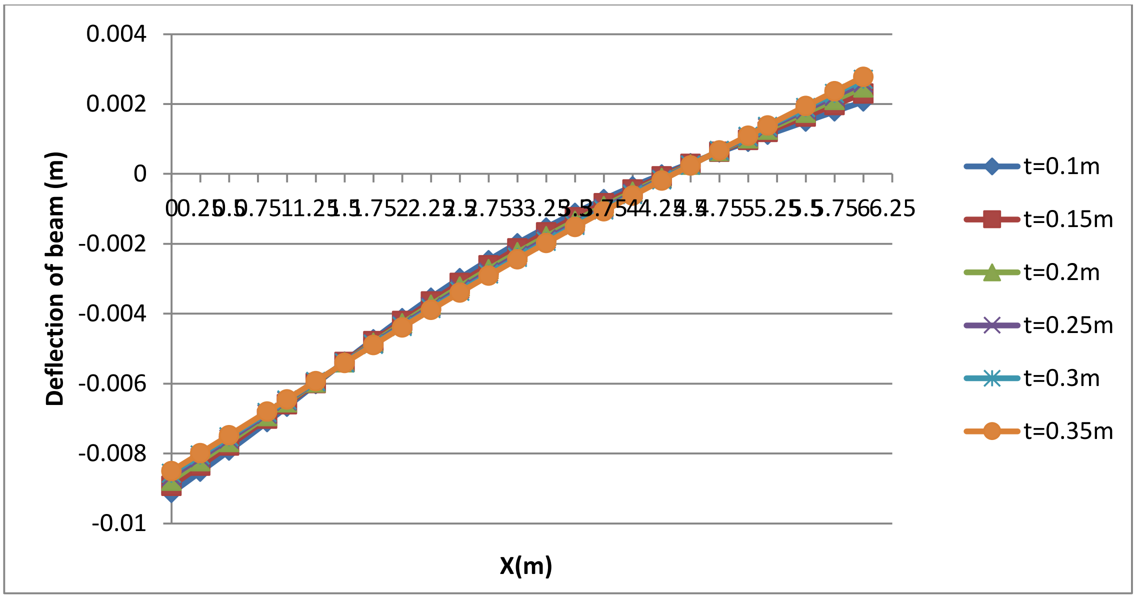

Fig. A.21. Beam deflection (m) vs X (m) at Different flange thickness for soil swell.

Table A.20. Beam deflection (m) vs X (m) at Different beam width for soil swell.

x	b=0.15m	b=0.2m	b=0.25m	b=0.3m	b=0.35m	b=0.4m	b=0.45m
0	-0.00901	-0.00906	-0.00909	-0.00913	-0.00916	-0.00919	-0.00922
0.25	-0.00843	-0.00846	-0.00849	-0.00852	-0.00855	-0.00858	-0.0086
0.5	-0.00784	-0.00787	-0.00789	-0.00792	-0.00794	-0.00796	-0.00798
0.83	-0.00704	-0.00706	-0.00708	-0.0071	-0.00711	-0.00713	-0.00714
1	-0.00662	-0.00664	-0.00665	-0.00666	-0.00668	-0.00669	-0.0067
1.25	-0.006	-0.006	-0.00601	-0.00601	-0.00602	-0.00602	-0.00603
1.5	-0.00537	-0.00537	-0.00537	-0.00537	-0.00537	-0.00536	-0.00536
1.75	-0.00476	-0.00475	-0.00474	-0.00473	-0.00473	-0.00472	-0.00471
2	-0.00417	-0.00415	-0.00414	-0.00412	-0.00411	-0.00409	-0.00408
2.25	-0.0036	-0.00358	-0.00356	-0.00354	-0.00352	-0.0035	-0.00348
2.5	-0.00306	-0.00303	-0.00301	-0.00298	-0.00296	-0.00294	-0.00292
2.75	-0.00255	-0.00252	-0.00249	-0.00247	-0.00244	-0.00242	-0.0024
3	-0.00207	-0.00203	-0.00201	-0.00198	-0.00196	-0.00193	-0.00191
3.25	-0.00161	-0.00158	-0.00156	-0.00153	-0.00151	-0.00149	-0.00147
3.5	-0.00119	-0.00116	-0.00113	-0.00111	-0.00109	-0.00107	-0.00105
3.75	-0.00078	-0.00076	-0.00074	-0.00072	-0.0007	-0.00069	-0.00067
4	-0.00041	-0.00039	-0.00037	-0.00036	-0.00034	-0.00033	-0.00032
4.25	-4.6E-05	-3.4E-05	-2.4E-05	-1.4E-05	-5.4E-06	2.79E-06	1.05E-05
4.5	0.000298	0.000303	0.000307	0.00031	0.000313	0.000316	0.000319
4.75	0.00063	0.000626	0.000623	0.00062	0.000617	0.000614	0.000612
5	0.000953	0.00094	0.000929	0.000919	0.00091	0.000901	0.000893
5.17	0.001168	0.00115	0.001134	0.001119	0.001105	0.001092	0.00108
5.5	0.001581	0.001551	0.001524	0.001499	0.001477	0.001455	0.001435
5.75	0.001892	0.001852	0.001817	0.001785	0.001755	0.001727	0.001701
6	0.002202	0.002153	0.00211	0.00207	0.002034	0.001999	0.001967

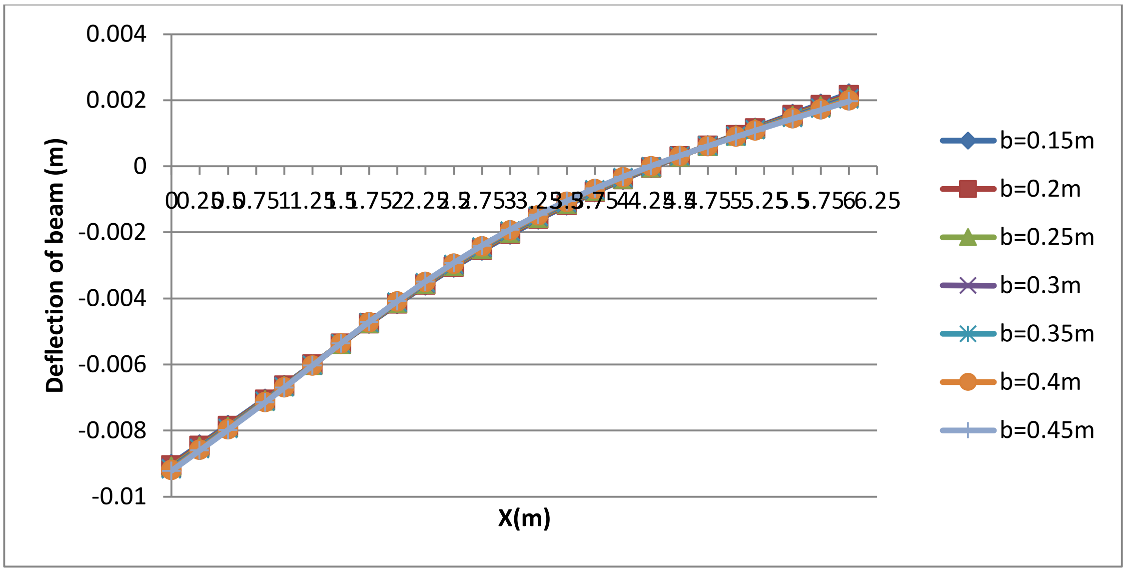

Fig. A.22. Beam deflection (m) vs X (m) at Different beam width for soil swell.

Table A.21. Beam deflection (m) vs X (m) at Different beam depth for soil swell.

x	h=0.2m	h=0.25m	h=0.3m	h=0.35m	h=0.4m	h=0.45m	h=0.5m
0	-0.00972	-0.00939	-0.00913	-0.00893	-0.00878	-0.00867	-0.00859
0.25	-0.00905	-0.00875	-0.00852	-0.00835	-0.00823	-0.00813	-0.00806
0.5	-0.00837	-0.00811	-0.00792	-0.00777	-0.00767	-0.00759	-0.00754
0.83	-0.00743	-0.00724	-0.0071	-0.007	-0.00693	-0.00688	-0.00684
1	-0.00692	-0.00677	-0.00666	-0.00659	-0.00654	-0.0065	-0.00648
1.25	-0.00613	-0.00606	-0.00601	-0.00599	-0.00596	-0.00595	-0.00594
1.5	-0.00534	-0.00535	-0.00537	-0.00538	-0.00539	-0.0054	-0.0054
1.75	-0.00457	-0.00466	-0.00473	-0.00479	-0.00482	-0.00485	-0.00487
2	-0.00385	-0.004	-0.00412	-0.00421	-0.00427	-0.00432	-0.00435
2.25	-0.00317	-0.00338	-0.00354	-0.00365	-0.00373	-0.00379	-0.00384
2.5	-0.00255	-0.0028	-0.00298	-0.00312	-0.00322	-0.00329	-0.00334
2.75	-0.002	-0.00227	-0.00247	-0.00261	-0.00272	-0.00279	-0.00285
3	-0.0015	-0.00178	-0.00198	-0.00213	-0.00224	-0.00232	-0.00237
3.25	-0.00107	-0.00133	-0.00153	-0.00167	-0.00178	-0.00185	-0.00191
3.5	-0.00069	-0.00093	-0.00111	-0.00124	-0.00134	-0.00141	-0.00146
3.75	-0.00037	-0.00057	-0.00072	-0.00083	-0.00091	-0.00097	-0.00102
4	-8.7E-05	-0.00024	-0.00036	-0.00044	-0.0005	-0.00055	-0.00058
4.25	0.000157	6.02E-05	-1.4E-05	-6.9E-05	-0.00011	-0.00014	-0.00016
4.5	0.00037	0.000337	0.00031	0.00029	0.000275	0.000263	0.000255
4.75	0.00056	0.000595	0.00062	0.000638	0.00065	0.000659	0.000666
5	0.000732	0.00084	0.000919	0.000977	0.001019	0.00105	0.001073
5.17	0.000843	0.001	0.001119	0.001205	0.001268	0.001314	0.001348
5.5	0.001048	0.001305	0.001499	0.001642	0.001747	0.001823	0.00188
5.75	0.001199	0.001532	0.001785	0.001972	0.002108	0.002207	0.002281
6	0.001349	0.001758	0.00207	0.0023	0.002468	0.002591	0.002683

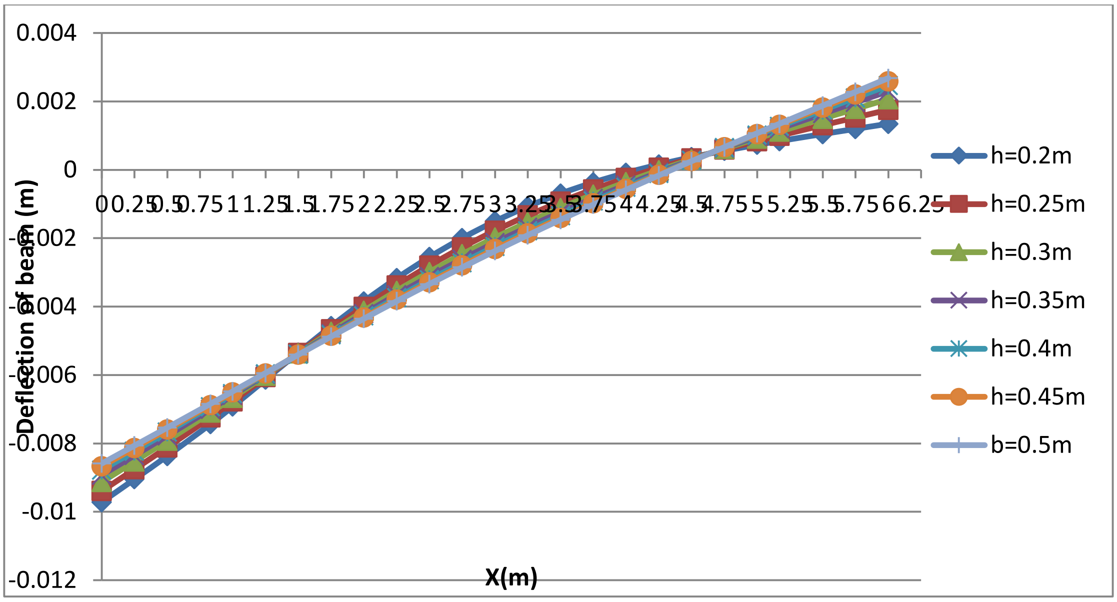

Fig. A.23. Beam deflection (m) vs X (m) at Different beam depth for soil swell.

Table A.22. Soil heave with different beam depth for soil swell

ΔU = 2.5PF		α= 0.00402		f=1		γh = 0.096		T = 1 year		H = 3m			
Slab Dimension 8x8m		Slab Dimension 10x10m		Slab Dimension 12x12m		Slab Dimension 14x14m		Slab Dimension 16x16m		Slab Dimension 18x18m		Slab Dimension 20x20m	
X (cm)	ΔH (cm)	X (cm)	ΔH (cm)	X (cm)	ΔH (cm)	X (cm)	ΔH (cm)	X (cm)	ΔH (cm)	X (cm)	ΔH (cm)	X (cm)	ΔH (cm)
0	1.88638	0	1.12729	0	0.66947	0	0.3967	0	0.23489	0	0.13904	0	0.08229
30	1.9098	40	1.15219	50	0.69262	60	0.41651	70	0.2509	80	0.15146	80	0.08965
60	1.98066	80	1.22801	100	0.76369	120	0.4779	140	0.30111	160	0.19094	160	0.11301
90	2.10074	120	1.35809	150	0.88758	180	0.58702	210	0.39237	240	0.26453	240	0.15657
120	2.27316	160	1.54823	200	1.07288	240	0.75477	280	0.53712	320	0.38539	320	0.22811
150	2.50241	200	1.80694	250	1.33245	300	0.99789	350	0.75509	400	0.5751	400	0.34039
180	2.79472	240	2.14596	300	1.6844	360	1.34076	420	1.07603	480	0.86758	480	0.51349
210	3.15852	280	2.58117	350	2.15366	420	1.81793	490	1.54388	560	1.31517	560	0.77835
240	3.60545	320	3.13439	400	2.77486	480	2.47871	560	2.22356	640	1.99862	640	1.18233
280	4.36141	360	3.83768	450	3.5992	540	3.3971	630	3.21509	720	3.04645	720	1.79807
320	5.37064	400	4.74498	500	4.71584	600	4.70558	700	4.70199	800	4.70073	800	2.73937
360	6.82038	450	6.35605	550	6.34197	650	6.33701	750	6.33527	850	6.33466	900	4.70028
400	9.46343	500	9.46343	600	9.46343	700	9.46343	800	9.46343	900	9.46343	1000	9.46343

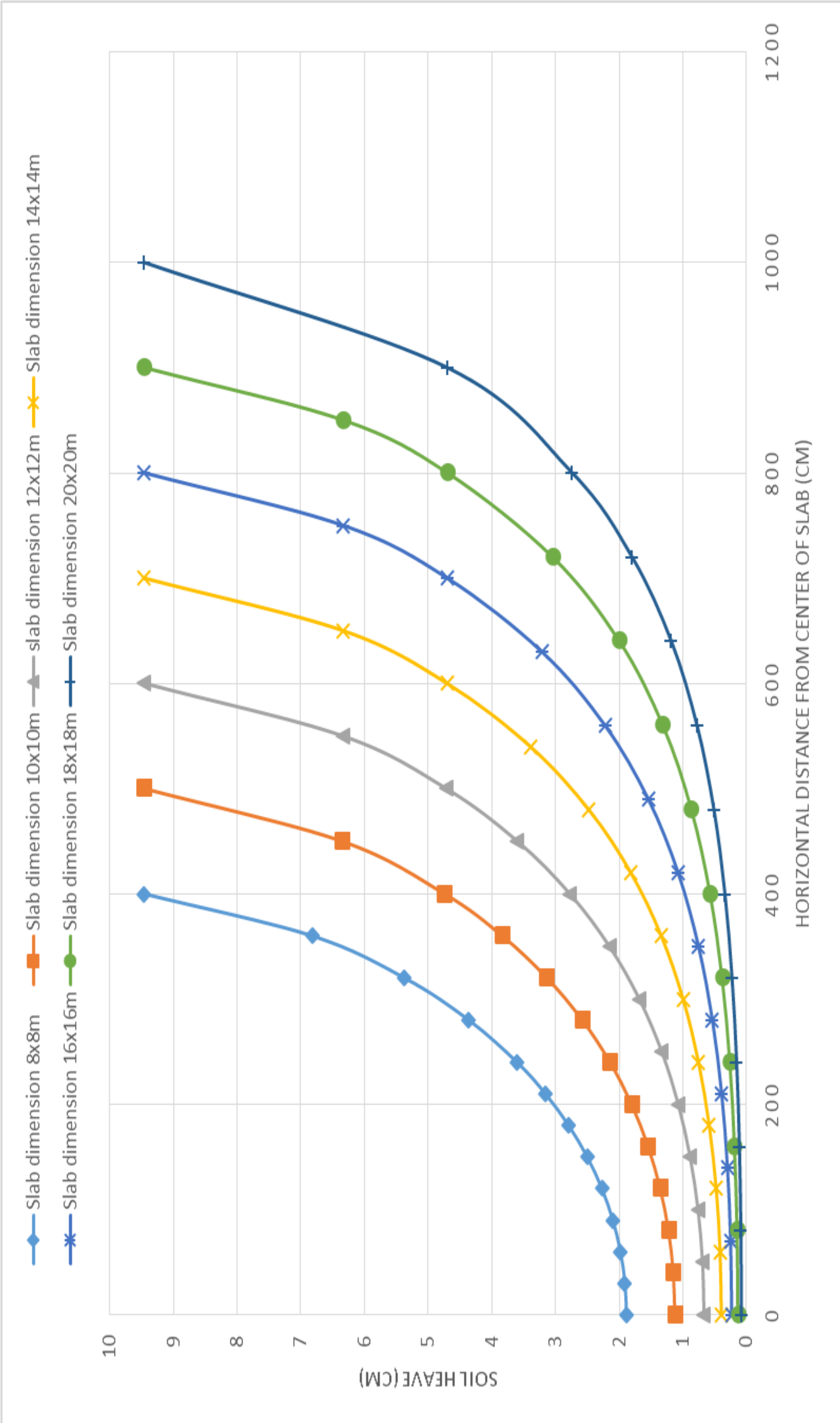

Fig. A.24. Soil heave with different slab dimension for soil swell

APPENDIX B

Break down for cost analysis

Waffle mat foundation

Concrete volume

Slab = 12*12*0.1 =14.4 m^3

Beam type 1

Beam depth in x direction = (0.3*11.2*0.5)*2 =3.36 m^3

Beam depth in y direction = (0.3*9.85*0.5)*2 = 2.95 m^3

Total = 3.36+2.95 = 6.31 m^3

Beam type 2

Beam depth in x direction = (0.4*12*0.5)*2 = 4.8 m^3

Beam depth in y direction = (0.4*11.2*0.5)*2 = 4.48 m^3

Total = 4.8+4.48 = 9.28 m^3

Beam type 3

Beam depth in x direction = (0.15*0.5*11.2)*3 =2.52 m^3

Beam depth in y direction = (0.15*9.85*0.5)*3 = 2.21m^3

Total = 2.52+2.21 = 4.73 m^3

Beam type 4

Beam depth in x direction = 0.1*11.2*0.5 = 0.56 m^3

Beam depth in y direction = 0.1*9.85*0.5 = 0.49 m_3

Total = 0.56+0.49 = 1.05 m^3

Beam type 5

Beam depth 47 x direction = (0.1*11.2*0.5)*2 = 1.12 m^3

Beam depth in y direction = (0.1*9.85*0.5)*2 = 0.98 m^3

Total = 1.12+0.98 = 2.05 m^3

Grand total = 14.4+6.31+9.28+4.73+1.05+2.105 = 37.875 m^3

Reinforcement

Beam type 1 = (8*12)*4 = 384m Dia. 20mm

Beam type 2 = (12*12)*4 = 576m Dia. 20mm

Beam type 3 = (4*12)*6 = 288m Dia. 14mm

Beam type 4 = (4*12)*2 = 96m Dia. 14mm

Beam type 5 = (4*12)*4 = 192m Dia. 14mm

Total reinforcement

Dia.20 = 384+576 = 960m Kg =$D^2L/162$

Dia.20 = 2370 Kg

Dia. 14 = 288+96+192 = 576m

Dia. 14 = 696.89 Kg

Total = 3067.26 Kg

Stirrup

Beam type 1

Number = 12000/210 = 58 length = 1,9m

= 58*1.9*4 =440 m Dia, 8mm

Beam type 2

Number = 12000/160 = 76 length = 2.1m

= 76*2.1*4 = 638.4 m Dia, 8mm

Beam type 3

Number = 12000/260 = 47 length = 1.6m

= 47*1.6*6 = 451.2m Dia, 8mm

Beam type 4

Number = 12000/260 = 47 length = 1.5m

= 47*1.5*2 = 141m Dia, 8mm

Beam type 5

Number = 12000/260 = 47 length = 1.5m

= 47*1.5*4 = 282 m Dia, 8mm

Total = 1952m Dia, 8mm

= 771.39 Kg Dia, 8mm

Isolated footing

Concrete volume

Beam type 1

Footing pad = (0.35*1.5*1.5)*4 = 3.15 m^3

Foundation column = (0.3*0.3*2)*4 = 0.72 m^3

Beam type 2

Footing pad = (0.3*1.3*1.3)*12 = 6.084 m^3

Foundation column = (0.3*0.3*2)*12 = 2.16 m^3

Grade beam in x direction = (0.3*0.3*12)*4 = 4.32 m^3

Grade beam in y direction = (0.3*0.3*10.8)*4 = 3.672 m^3

Total = 20.106 m^3

Reinforcement

Beam type 1 footing pad =10*1.75*4= 70m Dia. 12mm

Beam type 2 footing pad =8*1.5*12 = 144m Dia. 12mm

Foundation column =4*2*16 = 128m Dia. 12mm

Grade beam =4*12*8 = 384m Dia. 12mm

Total = 726m = 645.33 Kg

Excavation = 12*12*3 = 432m^3

Back fill = (12*12*3) – 20.106 – 44.4 =367.494 m^3

Masonry = (3.7*1*0.5)*24 = 44,4 m^3

Hardcore = 3.7*3.7*9 = 123.21m^2