AN INVESTIGATION OF EFFECT OF MODULUS OF SUBGRADE REACTION ON MAT FOUNDATION DESIGN

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ABSTRACT

AN INVESTIGATION OF EFFECT OF MODULUS OF SUBGRADE REACTION ON MAT FOUNDATION DESIGN

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A major problem is to estimate the numerical value of modulus of subgrade reaction, k_s in mat foundation design. Accuracy of analysis of the elastic foundation depends on our ability to define the modulus of subgrade reaction of the soil, which depends on the width and the shape of the loaded area, the depth of the loaded area below the ground surface, the position on the mat and time. In this study, the effects of modulus of subgrade reaction on mat foundation design is investigated. A mat foundation model is analyzed by SAFE with using different design parameters. The results of the analysis are examined and results are illustrated by figures. The relations between the subgrade modulus and load, thickness of slab and spans of columns are investigated.

Keywords: Modulus of subgrade reaction, subgrade models, mat foundations, elastic foundations.

ÖZ

ZEMİN YATAK KATSAYISININ RADYE TEMELLER ÜZERİNDEKİ ETKİSİNİN İNCELENMESİ

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Zemin yatak katsayısının belirlenmesi radye temel dizaynında büyük bir problem oluşturmaktadır. Elastik temellerin doğru sonuç vermesi bizim zemin yatak katsayısını belirleme yeteneğimize ve yüklenmeye maruz kalmış alanın enine, şekline, derinliğine, radye temelin pozisyonuna ve zamana bağlıdır. Bu çalışmada zemin yatak katsayısının radye temel dizaynı üzerindeki etkisi incelenmiştir. Bir radye temel modeli farklı dizayn parametreleri kullanılarak SAFE ile analiz edilmiştir. Analiz sonuçları incelenmiş ve sonuçlar grafiklerle gösterilerek açıklanmıştır. Zemin yatak katsayısı ile yük, temel tabakası kalınlığı ve kolonlar arası açıklık arasındaki ilişkiler irdelenmiştir.

Anahtar kelimeler: zemin yatak katsayısı, zemin modelleri, radye temeller, elastik temeller.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Foundations are part of a structure, which transmits the loads to the underlying soil. The most common types of foundation are spread foundations, continuous foundations and mat or raft foundations.

All soils, which are placed under the foundations that compress and this event cause to the settlement of the structure. If the soil has low bearing capacity and the column loads are great, the mat foundation may be used. Furthermore, soil is erratic or prone the differential settlements may cause to use mat foundation. Lateral loads are not uniformly distributed through the structure and thus may cause differential horizontal movements in spread footings or pile caps. Mat foundation prevent these movements. A particular advantage for basements at or below the ground water table is to provide a water basement.

Mat foundations require different approaches, depending on whether they can be classified as rigid or flexible. The column spacing in a strip is important to define the foundation type. If the spacing of the columns in a strip is less than 1.75/ β, foundation is considered as rigid or larger than 1.75/ β, foundation is considered as flexible (ACI, 1988).

Rigid method considers the mat far more rigid than the surrounding soils, so flexure of the mat does not affect distribution of bearing pressure.The design of conventional rigid method, magnitude and distribution of bearing pressure only depend on the applied loads and the weight of the mat. The soil pressure distribution is either uniform or varies linearly across the mat depending on eccentricity or moment.

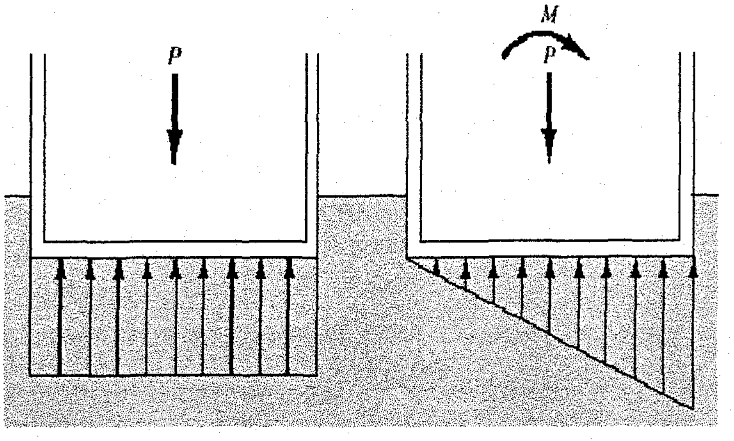

Figure 1.1 Soil pressure distribution according to rigid method

Flexible method considers the flexibility of the mat relative to the soil. In the design of approximate flexible method, the soil is assumed to be equivalent to an infinite number of elastic springs. This is generally referred to as the Winkler foundation. The elastic constant of these assumed springs is referred to as the coefficient of subgrade reaction, k (Coduto,1994).

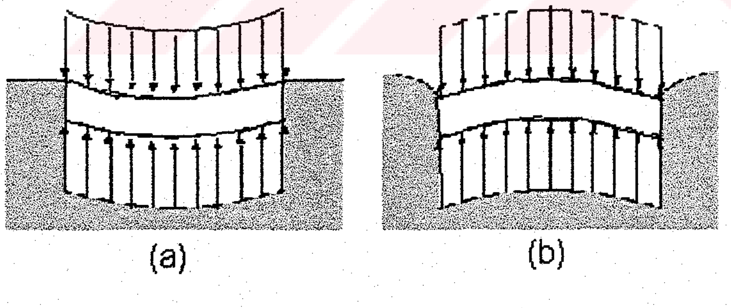

Figure 1.2 (a) Flexible foundation on clay, (b) Flexible foundation on sand

In the literature, Winkler's hypothesis does not represent the actual soil or subgrade model because real soil springs can be visualized as being linked together, not independent as Winkler's hypothesis implies (Horvath, 1983a). Many researchers have been attempted to idealize the actual subgrade model for the analysis of slabs. Such as the single parameter methods, the multiple parameter methods and simplified continuum models.

1.2 Definitions

The simplest and most popular model for the foundation base is the one proposed by Winkler (1867), consisting closely spaced independent springs. The model was developed for the analysis of railroad tracks, which could be conceived as infinitely long beam on an elastic support. In Winkler model, the coefficient of subgrade reaction is a relation between the footing contact pressure and the amount of the footing settlement under the pressure. It is defined as

$$k = \frac{q}{w} \qquad (1.1)$$

where k is the coefficient of subgrade reaction with units of force per unit volume and w is surface deflection. It is possible to interpret this model as a system of closely spaced springs with spring constant k as shown in figure 1.3 and figure 1.4.

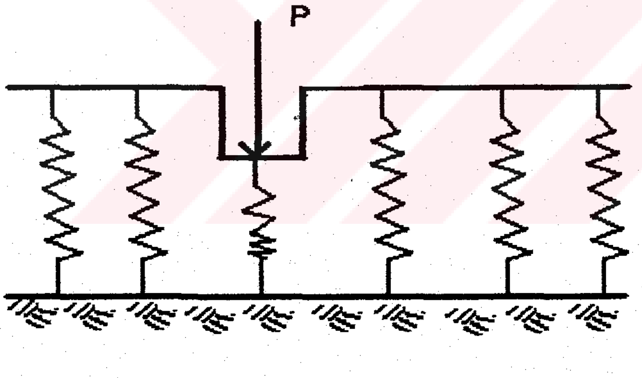

Figure 1.3 Displacements of the Winkler foundation model under concentrated load

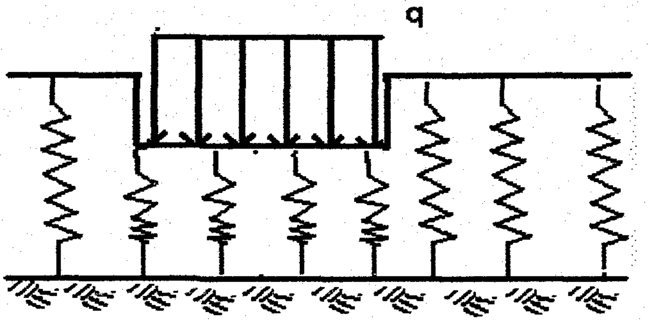

Figure 1.4 Displacement of the Winkler foundation model under uniform load

The main disadvantage of the Winkler method is that it predicts uniform settlement and zero bending moment for a uniformly loaded plate (Selvaduri, 1979).

In the literatures, two main types of soil subgrade model is acceptable that are mechanical models and simplified continuum models. In one category of these models (mechanical models) are divided by two sub-titles as a single parameter and a multiple parameter method. Single parameter method contains Winkler method and Pseudo-Coupled method types. Multiple parameter method contains Filonenko-Borodich model, Hetenyi model, and Pasternak model. Also, simplified continuum model contains Vlasov model and Reissner model.

The composition of the mechanical models in table 1.1, in order of their accuracy as a subgrade model.

Table 1.1 Composition of mechanical models

Subgrade Model	The physical elements of subgrade model
Winkler model	Springs
Pseudo-Coupled model	Coupled springs
Filonenko-Borodich model	Pre-tensioned membrane + spring
Pasternak model	Shear layer + spring
Hetenyi model	Spring + plate + spring

Contents of this table are mentioned in the following chapters.

1.3 Object and Scope

A literature survey on development of theories for analyzing the mat foundations include the choice of a subgrade model. When the chosen model closely represents the actual situation, the resulting analyses are useful tools for design engineers. The analysis of mat foundations on elastic soil can be performed by the concept of modulus of subgrade reaction, defined as stress per unit displacement. In such an analysis, subgrade modulus represents the stiffness of the Winkler springs. However, the determination of modulus of subgrade reaction presents difficulties. Because the value of modulus of subgrade reaction is not a constant for a particular soil. It depends on the depth of strata, the shape of the loaded area, relative stiffness of the foundation and soil (Terzaghi, 1955).

The objective of this study is to investigate the effects of ks on mat foundation, such as displacements, moments and soil pressures for different load patterns, thicknesses and span lengths for analyzing the mat foundations.

1.4 Organization of Thesis

This thesis is consist of seven chapters as mentioned below,

In first chapter, general knowledge about mat foundation design methods and subgrade models are explained.

Chapter 2 is separated for literature survey about analyzing mat foundations with various methods.

In Chapter 3, subgrade modulus and its determination methods are defined.

Determination of the design method of mat foundations is investigated in Chapter 4.

Subgrade methods are explained in chapter 5.

In Chapter 6, SAFE v7.01 is described. Furthermore, analysis of mat foundation and related graphics are presented with various situations.

In Chapter 7, conclusions and recommendations for future work are presented.

CHAPTER 2

LITERATURE SURVEY

As cited by Selvaduri (1979), Winkler (1867) had introduced the first simplified subgrade model. Winkler model is the most popular model because of its simplicity. This model was first used to analyze the deflections and stresses in rail road trucks. Nowadays, researchers are still accept the Winkler model as a origin of elastic foundations.

An extensive collection of closed form solutions for the differential equation of beams on Winkler foundation is presented. Hetenyi was developed a model, which is influenced an elastic plate between the springs under surface loads (Hetenyi, 1946).

As cited by Selvaduri (1979), Vlasov and Leontev assuming that the horizantal displacements in the soil are zero. Also the advantage of this model, subgrade modulus and shear modulus are determined directly from the modulus of elasticity and Poisson's ratio of the soil. Selvaduri has an extensive amount of work on elastic foundation models. Each model whether single parameter or multiple parameter is explained in this publication.

Horvath (1983a) based on Reissner's simplified foundation model presents result of his calculations comparing the performance of Winkler, Reissner models as well as the conventional rigid method in predicting soil settlement, contact stress distribution and bending moments in mat foundations. He concludes that Reissner's simplified model is superior and adequately represents the soil continuum.

Eisenberger and Clastornic (1987) derived element matrices for beams on variable two parameter elastic foundation, based on both exact and cubic shape functions and compared the results from the two models.

Landva et al. (1988) measured the contact stresses and settlements under a nine storey building which was constructed on a raft resting on a 30 meter thick layer of clayey silt. Comparative plots of measured and computed, based on Bowles' recommendation of doubling edge springs, values of contact stresses and settlements are given in the paper. They conclude that the edge spring doubling gives satisfactory results.

ACI committee (1988) addresses the design of shallow foundations. The report presents guidelines for the design of combined footings and mats, and the problem of determination of contact stresses and the estimation of modulus of subgrade reaction. The committee considered that a uniformly loaded plate should deflect more at the center, recommends that the continuity in the soil adequately be represented by doubling of edge springs which they call spring coupling. The reasoning behind this spring coupling procedure, according to the committee is given in the committee closure of discussion of the report.

Yankelevsky et al. (1989) formulated an iterative numerical procedure using exact solutions for a finite beam on a linear elastic Winkler foundation. They concluded that under high intensity of loading significant moments occur in the beam, pointing out that elastic analysis gives non-conservative results.

Vallabhan and Das (1988) pointed out inconsistencies in the use of the modulus of subgrade reaction and proposed the utilization of more elegant and consistent model by Vlasov. They extended the Vlasov model to cover elastic soil with Young's modulus linearly varying with depth and established a relationship between the G-parameter and the displacement of the beam or the slab on the top by using variational calculus.

Liou and Lai (1996) describes a method in lumping the Winkler springs under the mat foundations with grid floor beams as stiffeners. Based on yield line theory of slabs, the springs are lumped under each floor beam. Numerical comparisons are presented with more sophisticated finite element models.

Daloglu (2000) suggested a parameter, which is a characteristic length of the slab is defined as r. The coordinate parameters and lateral displacement are divided by r parameter and all of them non-dimensionalized by this way. Daloglu has used Vlasov model with these non-dimensional parameters to analyze the slabs on an elastic foundation and the value of the nondimensional modulus of subgrade reaction, K_{nw}, is computed by Vlasov equation which is modified by Daloglu (Daloglu,2000).

As may be observed from the continuing researchers in this area, the problem of beams and plates on elastic foundations still has practical importance.

CHAPTER 3

SUBGRADE MODULUS

3.1 Coefficient of Subgrade Reaction

The deformation characteristics of the soil are quantified in the coefficient of subgrade reaction, or subgrade modulus. The coefficient of subgrade reaction forms the basis for a "bed of springs" analogy as shown in figure 3.1 to model the soil-structure interaction in mat foundations.

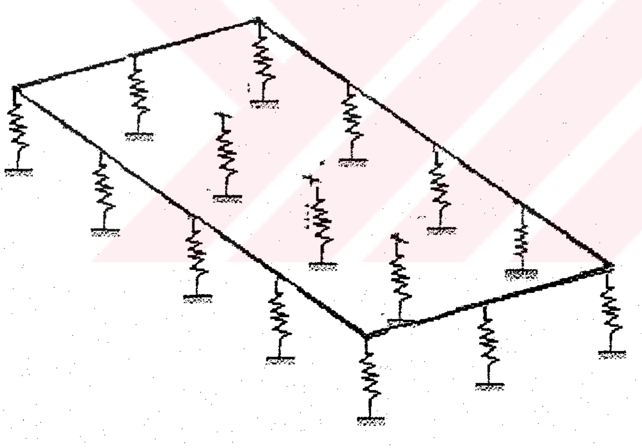

Figure 3.1 Application of coefficient of subgrade reaction to larger mats

The coefficient of subgrade reaction is a relation between the footing contact pressure and the amount of the footing settlement under the pressure. It is defined as

$$k_s = \frac{q}{w} \qquad (3.1)$$

where k_s is the coefficient of subgrade reaction, q is the load per unit area (or contact pressure) applied to the mat and w is the settlement of the mat foundation. The relationship of q and w is not linear. That is because determining k_s is difficult. The coefficient of subgrade reaction is not just a soil parameter, it also effected by the structural stiffness. It depends on several factors, such as the length and width of the foundation, the depth of the embedment of the foundation, the type of structure, and the type of soil beneath the foundation. It can also be time dependent because of the settlement of mats on deep compressible soils is due to consolidation (Coduto, 1994).

3.2 Determination of The Coefficient of Subgrade Reaction

The coefficient of subgrade reaction may be determined by field test as defined plate load test and standard penetration test, and empirical methods such as Biot (1937), Vesic (1961), Bowles (1996) and Daloglu (2000).

3.2.1 Determination of the coefficient of subgrade reaction by plate load test

The literature survey on determining subgrade reaction, as cited by Ordemir (1984), Abbett had explained that ks may or may not vary along depth depending on the nature of soil. It is generally observed that in case of sandy soil, ks increases with depth. For clayey soil however the value of ks is more or less constant with depth (Ordemir, 1984). However, Terzaghi ignored the depth factor for determining ks on sandy soil (Terzaghi, 1955).

According to Abbett, two load tests on a bearing plates of different sizes are necessary for determining ks on partially cohesive soils. The relationship of width and depth of plate (or footing), and k coefficient may be expressed as

$$k_s = \frac{q}{w} = c_1(1 + \frac{2d}{b}) + \frac{c_2}{b} \qquad (3.2)$$

where c_1 and c_2 are constants for the given soil, d is the depth of the ground surface of the bearing plate and b is the width of the bearing plate.

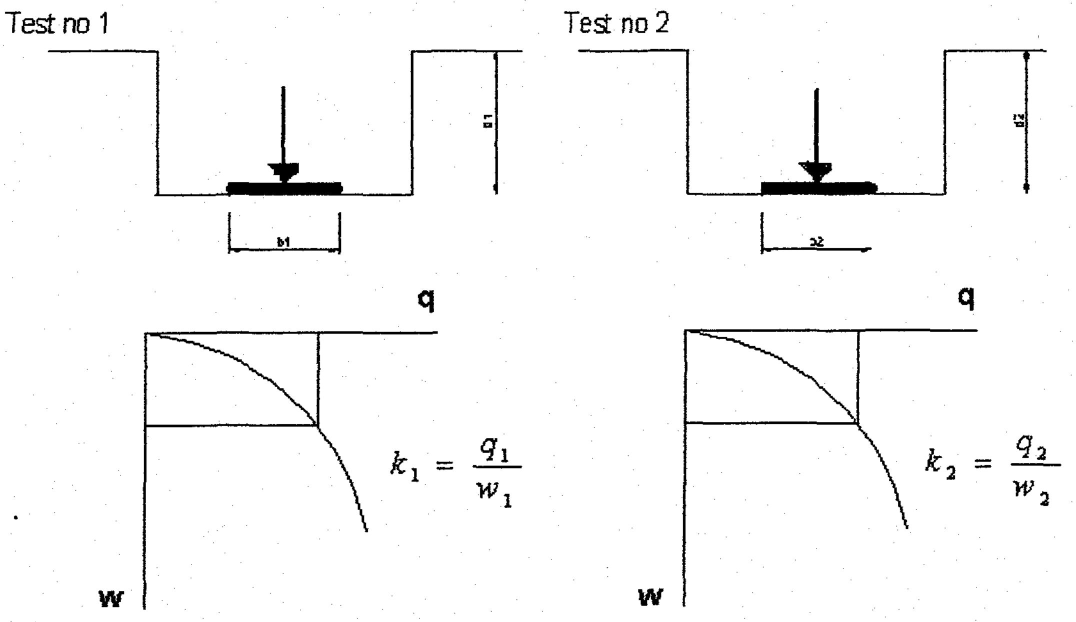

Figure 3.2 Test pits and load-settlement curves

$$k_1 = \frac{q_1}{w_1} \qquad (3.3)$$

$$k_2 = \frac{q_2}{w_2} \qquad (3.4)$$

where k_1 and k_2 are determined from load-settlement curves. Then equation (3.5) and (3.6) can be expressed relatively, according to test no 1 and test no 2.

$$k_1 = c_1(1+\frac{2d_1}{b_1})+\frac{c_2}{b_1} \qquad (3.5)$$

$$k_2 = c_1(1+\frac{2d_2}{b_2})+\frac{c_2}{b_2} \qquad (3.6)$$

The soil constants c_1 and c_2 can be calculated from the equations (3.3), (3.4), (3.5) and (3.6). Therefore, the coefficient of subgrade reaction of actual footing having a width, B and depth, D can be determined as

$$k_s = c_1(1+\frac{2D}{B})+\frac{c_2}{B} \quad \text{..........(3.7)}$$

The coefficient of subgrade reaction in cohesive soils can be determined by a single load test. Firstly, k_{test} is calculated from recorded q and w values. It is defined as

$$\frac{q}{w} = k_{test} \quad \text{..........(3.8)}$$

c_2 is calculated from below equations

$$k_{test} = \frac{c_2}{b} \quad \text{..........(3.9)}$$

$$c_2 = bk_{test} \quad \text{..........(3.10)}$$

The coefficient of subgrade reaction of actual footing in cohesive soil is defined as

$$k_s = \frac{c_2}{B} \quad \text{..........(3.11)}$$

In cohessionless soils, k_s can be determined by a single load test. q and w values is recorded to determine the k_{test}. It is defined as

$$\frac{q}{w} = k_{test} \quad \text{..........(3.12)}$$

Determined k_{test} value is used to calculate c_1 and defined as

$$k_{test} = c_1(1+\frac{2d}{b}) \quad \text{..........(3.13)}$$

In cohessionless soils, k_s of actual footing is defined as

$$k_s = c_1(1+\frac{2D}{B}) \qquad (3.14)$$

Terzaghi proposed that k_s for full-sized footing could be obtained from plate load tests using the following equations respectively, for footings on clay and for footings on sandy soils.

$$k_s = k_1\frac{B_1}{B} \qquad (3.15)$$

$$k_s = k_1(\frac{B+B_1}{2B})^2 \qquad (3.16)$$

where B_1 is the dimension of test plate, k_1 is the determined value is obtained from the plate load test, B is the actual footing width.

Scott (1981) proposed that the subgrade modulus can be determined by standard penetration test on sandy soils. It is defined as

$$k = 1.8N \qquad (3.17)$$

The ks value may be determined by empirical methods such as that proposed by Biot (1937) defined as

$$k = \frac{0.95E_s}{(1-\vartheta_s^2)}[\frac{B^4E_s}{(1-\vartheta_s^2)EI}]^{0.108} \qquad (3.18)$$

where E_s is the modulus of elasticity of the soil, ϑ_s is the Poisson's ratio of the soil, B is the width of the beam, EI is the bending rigidity of the beam. As cited by Daloglu, Biot solved the problem for an infinite beam with a concentrated load resting on a 3D elastic soil continuum and found a correlation of the continuum elastic theory and the

Winkler model where the maximum moments in the beam are equated (Daloglu, 2000).

Based on similar reasoning, Vesic tried to develop a value for k, except in stead of matching bending moments, he matched the maximum displacements of the beam in models. He obtained the equation for k for use in the Winkler model as

$$k = \frac{E_s}{B(1-\vartheta_s^2)} 0.65.\sqrt[12]{\frac{B^4 E_s}{EI}} \quad \text{.........(3.19)}$$

for practical uses Vesic's equation reduces to

$$k = \frac{E_s}{B(1-\vartheta_s^2)} \quad \text{.........(3.20)}$$

where E_s is the modulus of elasticity of the soil, ϑ_s is the Poisson's ratio of the soil, B is the width of the beam, EI is the bending rigidity of the beam (Horvath,1983a).

Bowles has suggested the coefficient of subgrade reaction from the allowable bearing capacity, q_a furnished by the geotechnical consultant

$$k_s = 40(SF)q_a \quad \text{.........(3.21)}$$

where ks is furnished in kN/m^3, and q_a is furnished in kPa.
For the ultimate soil pressure cause a settlement ΔH . For ΔH =6, 12, 20 mm, etc., the factor 40 can be adjusted to 160, 83, 50, etc., (Bowles,1996).

Daloglu suggested a parameter, which is a characteristic length of the slab is defined as

$$r = \sqrt[4]{\frac{DH}{E_s}} \quad \text{.........(3.22)}$$

where D is the flexural rigidity of the slab and H is depth of soil layer. The coordinate parameters and lateral displacement are divided by r parameter and all of them non-dimensionalized by this way. Daloglu has used Vlasov model with these non-dimensional parameters to analyze the slabs on an elastic foundation and the value of the nondimensional modulus of subgrade reaction, K_{nw}, is computed by Vlasov equation which is modified by Daloglu (Daloglu,2000). The modulus of subgrade reaction can be calculated (with dimensions of F/L^3) as

$$k = \frac{K_{nw} D}{r^4} \qquad (3.23)$$

In published sources there are very different values of k_s have defined. One of these is shown below in table 3.1 (Celep, Kumbasar, 2001).

Table 3.1 Range of modulus of subgrade reaction

Soil type	***k_o(kN/m^3)***	***k_o(t/m^3)***
Mud	*<2000*	*<200*
Plastic clay	*5000~10000*	*500~1000*
Semi-stiff clay	*10000~15000*	*1000~1500*
Stiff clay	*15000~30000*	*1500~3000*
Filled soil	*10000~20000*	*1000~2000*
Loose sand	*10000~20000*	*1000~2000*
Medium dense sand	*20000~50000*	*2000~5000*
Dense sand	*50000~100000*	*5000~10000*
Dense sand and Gravel	*100000~150000*	*10000~15000*
Strong schist	*>500000*	*>50000*
Rock	*>2000000*	*>200000*

CHAPTER 4

STRUCTURAL DESIGN OF MAT FOUNDATIONS

4.1 Structural Design of Mat Foundations

The structural design of mat foundations can be carried out by two methods, the conventional rigid method and the approximate flexible method.

4.1.1 Determination of the design method

One of the most commonly asked questions in geotechnical engineering is how to decide the design method. β is a very important parameter in the determination of whether a mat foundation should be designed by rigid method or flexible method.

$$\beta = \sqrt[4]{\frac{B * k}{4E_f I_f}} \quad \text{.......(4.1)}$$

where B is the foundation width, k is the coefficient of subgrade reaction, E_f is the young's modulus of the foundation, I_f is the moment of inertia of the foundation.

$$I_f = \frac{1}{12} B * h^3 \quad \text{.......(4.2)}$$

where h is the thickness of slab (Bowles, 1996).

According to the American Concrete Institute Committee 336 (1988), if the spacing of the columns in a strip is less than 1.75/ β, rigid method should be done or larger than 1.75/ β, flexible method should be done.

4.2 Conventional rigid method

The conventional rigid method is the simplest and, for decades, was the most commonly used analytical method for representing subgrade behavior in soil-foundation analysis, especially for mat foundations. The mat is assumed to be a rigid so that it does not deform but only displaces as a rigid body and thus will never produce differential settlements. The rigid method assumes there are no flexural deflections in the mat, so the distribution of soil bearing pressure is simple to define. However, these deflections are important because they influence the bearing pressure distribution. It assumes that the soil is linear elastic material (Coduto, 1994).

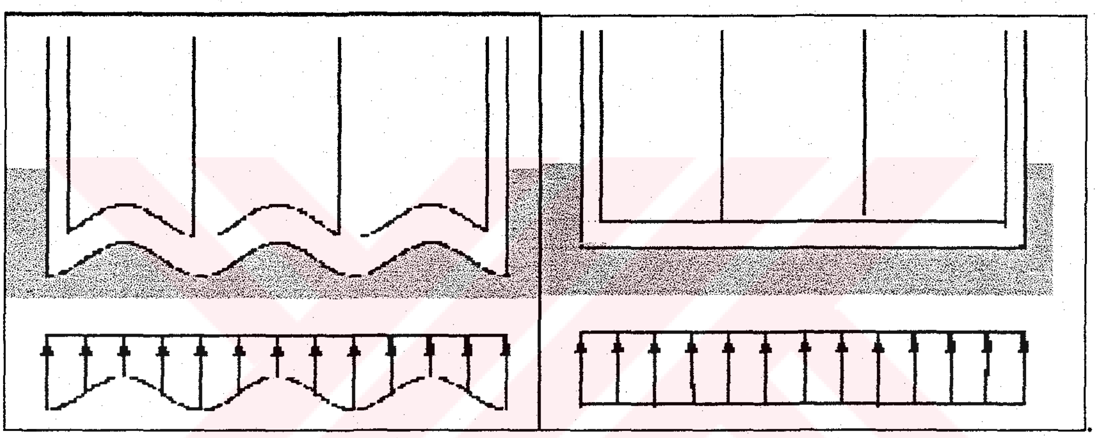

Figure 4.1 Respectively, flexible mat and rigid mat illustrated

In conclusion, the conventional rigid method is unacceptable for soil-foundation analysis because it does not and cannot provide estimates of total settlement. Furthermore, the conventional rigid method always produces incorrect results on the unconservative side for the two primary results required for mat design; differential settlements and bending moments (Horvath, 1983b).

4.3 Approximately Flexible Method

Flexible methods consider the deformation of the mat and their influence of bearing pressure distribution. These methods produce more accurate values of mat deformations and stresses. Furthermore, these methods are more difficult to implement than rigid methods because of soil-structure interaction.

Flexible methods must take into account that both the soil and the foundation have deformation characteristics. The deformation characteristics of the soil are quantified in the coefficient of subgrade reaction, or subgrade modulus, which is similar to the modulus of elasticity for unidirectional deformation.

4.3.1 In usual beam theory

Figure 4.3 shows the general case of simple beam under the spread loading.

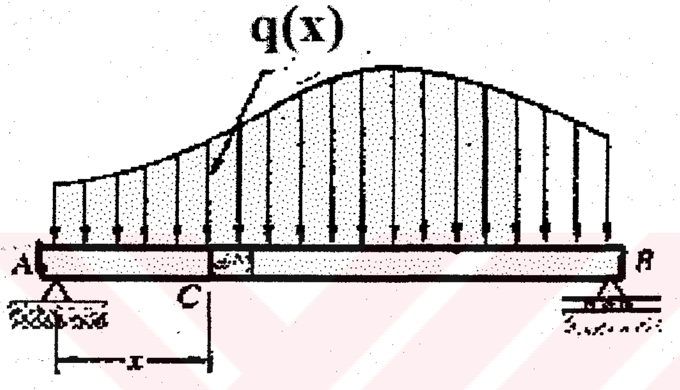

Figure 4.3 Simple beam under the arbitrary loading

Derivations of usual beam theory are defined as;

$$\frac{dV}{dx} = -q \qquad (4.3)$$

$$\frac{dM}{dx} = V \qquad (4.4)$$

$$\frac{d^2M}{dx^2} = -q \qquad (4.5)$$

According to equations (4.2), (4.3) and (4.4), moment is defined as;

$$M = -EI\frac{d^2w}{dx^2} \qquad (4.6)$$

Finally,

$$EI\frac{d^4w}{dx^4}=q \qquad (4.7)$$

4.3.2 Beam theory on Winkler foundation

Figure 4.4 shows the general case of a simple beam supported on a subgrade.

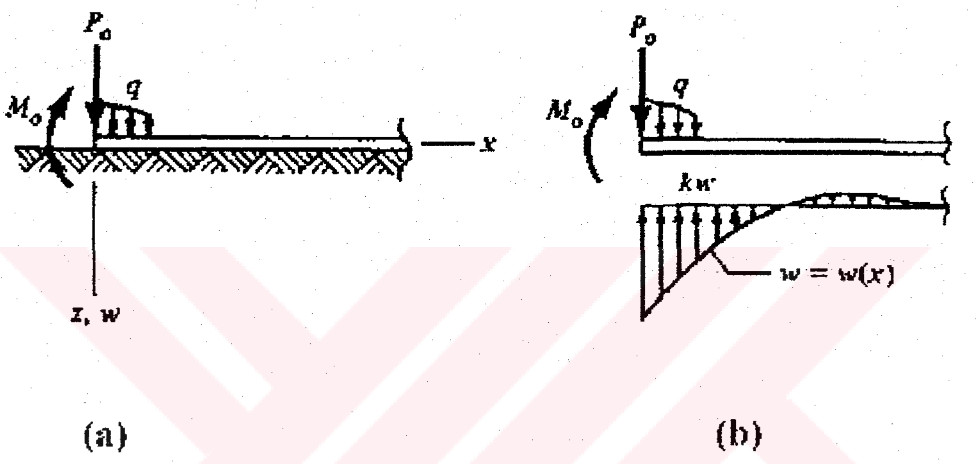

Figure 4.4 (a) Arbitrary loading on an elastically supported beam. (b) Reaction "kw" of a Winkler foundation

The curve w=w(x) in figure 4.4 (b), is deflected shape of the beam.

The differential equations for a simple beam for this situation are defined as;

$$\frac{dV}{dx}=-q+kw \qquad (4.8)$$

$$\frac{dM}{dx}=V \qquad (4.9)$$

$$\frac{d^2M}{dx^2}=kw-q \qquad (4.10)$$

According to equations (4.8), (4.9) and (4.10), moment is defined as;

$$M = -EI\frac{d^2w}{dx^2} \qquad (4.11)$$

Finally, the flexural behavior of this beam is defined as;

$$EI\frac{d^4w}{dx^4} + kw = q \qquad (4.12)$$

CHAPTER 5

SUBGRADE MODELS

5.1 Preliminary Remarks

A literature survey on development of theories for analyzing foundations include the choice of a subgrade model. When the chosen model closely represents the actual situation, the resulting analyses are useful tools for design engineers.

Subgrade models represent a unique type of constitutive model in geotechnical engineering because they are never intended to be general or complete material models. Rather, the intent of a subgrade model is to strike a balance between theoretical accuracy and ease of use in routine geotechnical engineering practice. An acceptable subgrade model for soil-foundation problems is one that produces reasonably accurate estimates of total settlement at the soil-foundation interface and bending moments within the foundation two parameters alone.

A detailed presentation of how various subgrade models can be applied to a particular problem involving a mat foundation. The primary goal of this is to illustrate at least qualitatively how calculated results can be sensitive to the subgrade model chosen as well as other details involving structural engineering aspects of the problem.

5.2 Mechanical Models

5.2.1 Single parameter methods

The term one parameter signifies that the model is defined by only one elastic constant as called coefficient of subgrade reaction, k.

5.2.1.1 Winkler's hypothesis

The earliest use of "springs" to represent the interaction between soil and foundation was done by Winkler in 1867; the model is thus referred to as the Winkler method. The one-dimensional representation of this is a "beam on elastic foundation," thus sometimes it is called the "beam on elastic foundation" method. Mat foundations represent a two-dimensional application of the Winkler method.

Winkler's Hypothesis is also referred to as the coefficient of subgrade reaction or beam on elastic foundation method. It is based on the use of the Winkler subgrade model, which is the most widely known and used subgrade model on foundation analysis. In this model, the relationship between the applied surface pressure and vertical surface deflection at every point can be expressed as;

$$q_i = kw_i \qquad (5.1)$$

where q_i is the applied vertical surface pressure at point i, w_i is the vertical surface deflection of point i, and k is the coefficient of subgrade reaction with units of stress per unit volume (Selvaduri, 1976).

The mechanical model for Winkler method, is described by an assemblage of an infinite number of linear springs, in which case k is equivalent to the spring constant per unit volume.

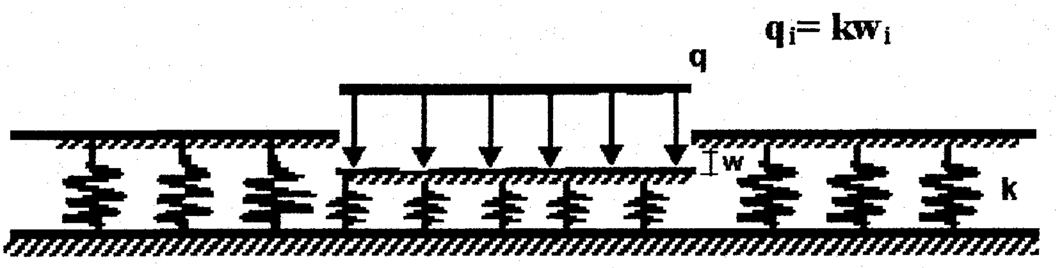

Figure 5.1 Winkler subgrade model

Winkler modeled the subgrade as a system of closely spaced springs with spring constant k. It is important to note that the length or spacing of the soil springs which is illustrated in figure 5.1 has no physical meaning. The spring analogy is only a concept employed to help visualize the behavior of subgrade model. It is also important a note that the spring analogy is a discrete model because the soil is a continuum and represented by an assemblage of independent mechanical elements -springs-. The response at one spring is not dependent on the response at any other spring except loaded flexible structure element is placed on the top of the springs.

Winkler model assumes that the plate is supported by discrete springs under the plate. Furthermore, the Winkler model disregards the continuity in the soil and the basic disadvantage of this assumption is that the plate makes uniform settlement under uniform loads leading to zero bending moments in the plate (Kerr, 1964).

5.2.1.1.1 Deficiencies of Winkler's hypothesis

The deficiencies of Winkler's hypothesis as a subgrade model, it assumes that a uniformly loaded mat underlain by a perfectly uniform soil will uniformly settle into the soil.

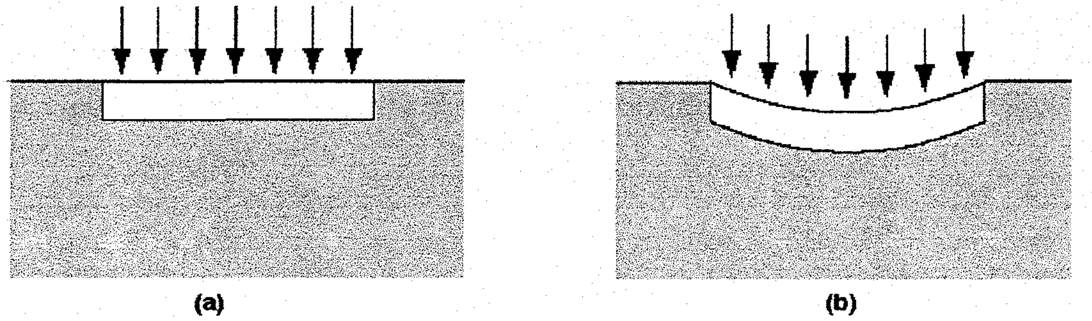

Figure 5.2 (a) Winkler subgrade model, (b) Actual subgrade model

Actual data show that such a mat-soil interaction will deflect in the center more than the edges. It is illustrated in figure 5.2.

Load-settlement curves are not really linear; we must make a linear approximation to use the Winkler model. It is shown in figure 5.3.

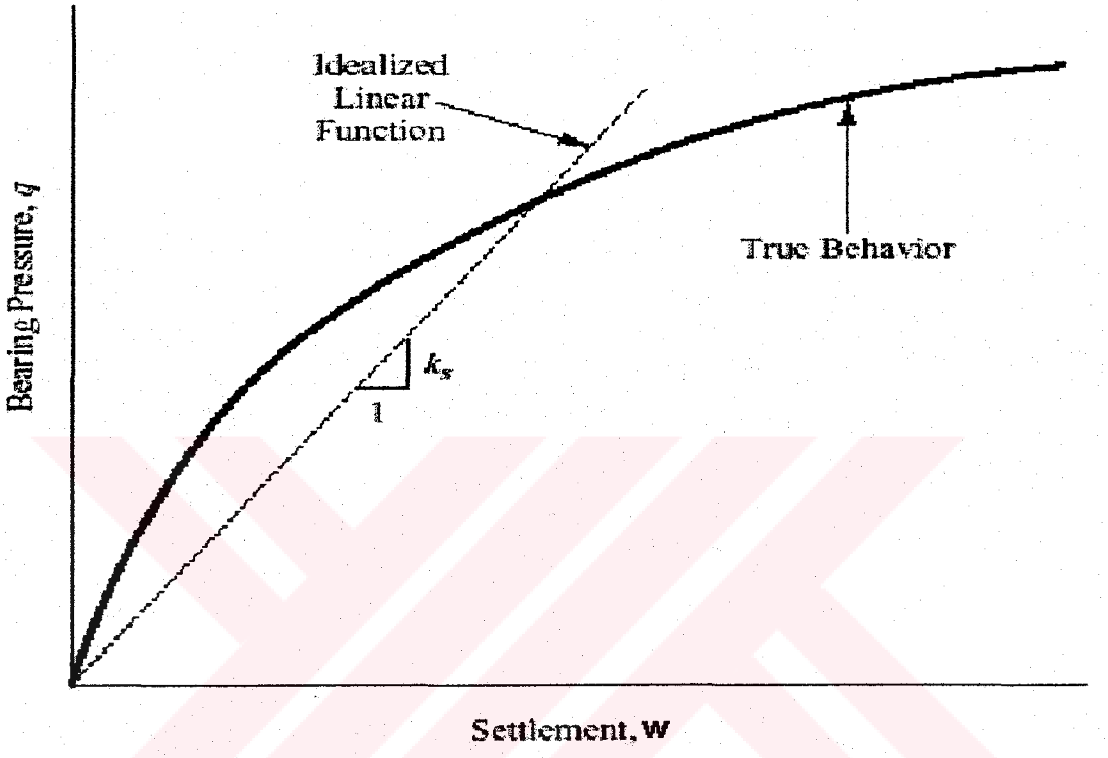

Figure 5.3 Non-linear characteristics of soil deformation

No single value of ks truly represents the interaction between the soil and the mat. The independent spring problem is in reality the largest problem with the Winkler model. In other words, the absence of "spring coupling" in Winkler model is its single most significant shortcoming as a subgrade model. Because of this, the Pseudo-Coupled Model is, in reality, nothing fundamentally new but is simply a repackaged version of the single – parameter Winkler hypothesis.

5.2.1.2 Pseudo-Coupled model - general concept

The Pseudo-Coupled model is simply a return to the general form of Winkler's hypothesis. The Pseudo-Coupled model provides the analytical improvement over the traditional use of Winkler model with a constant coefficient of subgrade reaction without increasing the analytical complexity. The Pseudo- Coupled model uses the additional springs which are described as a spring coupling shown in figure 5.4.

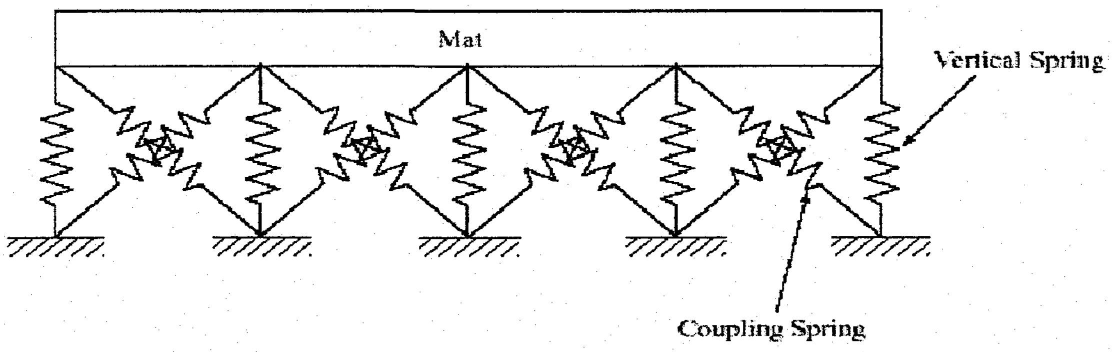

Figure 5.4 The Pseudo-Coupled subgrade model – general concept

Winkler's hypothesis is a poor subgrade model because it neglects the shearing effects that occurs within actual subgrade.

The problem with the Pseudo-Coupled model comes in selecting the values of the coefficient of subgrade reaction for the coupling springs because each application is a unique combination of structural and geotechnical components. As a result, the magnitude and distribution of the coefficient of subgrade reaction beneath a mat is a unique to a given application.

5.2.1.3 Pseudo-Coupled model – the discrete area method

There is a variation of the Pseudo-coupled model called the discrete area method. In the discrete–area method version of the Pseudo-Coupled model, both the magnitude and variation of k(x,y) are estimated on a project-specific basis. Distributed springs act independently like Winkler springs, but have different ks values depending upon their location on the mat.

The coefficient of subgrade reaction, ks progressively increases from the innermost zone to the outermost zone. Figure 5.5 shows plainly these zones. The shears, moments and deformations are evaluated by using the Winkler method.

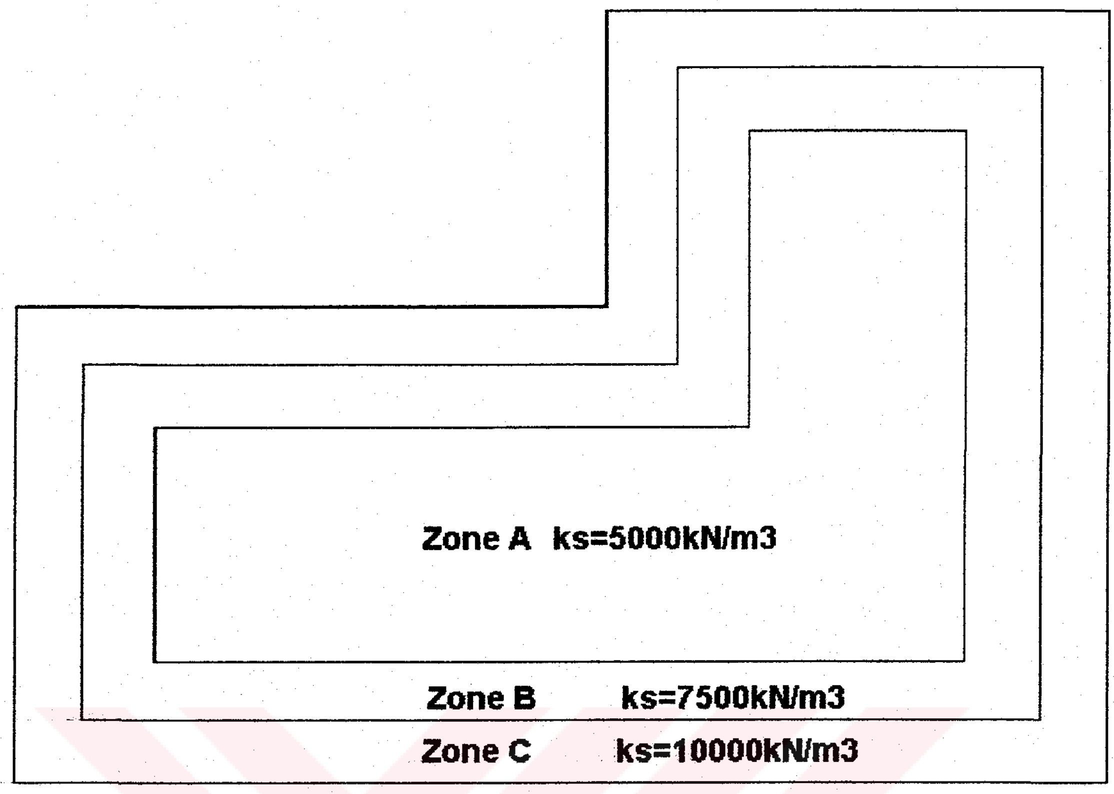

Figure 5.5 A typical mat and subgrade divided into zones for a Pseudo-Coupled analysis

The structural plate representing the mat in the structural analysis and the foundation level of the subgrade in the geotechnical analysis are each divided into the same pattern of "discrete areas" with the number, shape and size of areas used arbitrary. In the structural analysis, each area on the mat is supported by an independent spring but a different spring stiffness is allowed beneath each discrete area. Thus from a structural perspective the general form of Winkler's Hypothesis is used for the subgrade model. In the geotechnical analysis, each discrete area is subjected to a uniformly distributed load but a different load magnitude is allowed for each discrete area. Thus from a structural perspective the general form of Winkler's Hypothesis is used for the subgrade model.

In the geotechnical analysis, each discrete area is subjected to a uniformly distributed load but a different load magnitude is allowed for each discrete area. Spring stiffnesses in the structural analysis and applied stresses in the geotechnical analysis of each area are changed in an iterative, trial-and-error process until the displacement patterns from the two analyses match within some error judged acceptable by the engineers involved. It is assumed that this match provides the unique solution to the problem. Because of the trial-and-error requiring and not to mention the fact that a separate series of analyses must be performed for every load case on a given project, that have discouraged its wider use in practice.

5.2.2 Multiple parameter methods

The fundamental and most significant attraction of multiple-parameter models as a group is that a variation in the coefficient of subgrade reaction never has to be assumed beforehand as it does when the Pseudo-Coupled models are used.

5.2.2.1 Filonenko-Borodich model

The Filonenko-Borodich model, which consists of a deformed, pre-tensioned membrane over a spring layer that shown in figure 5.6.

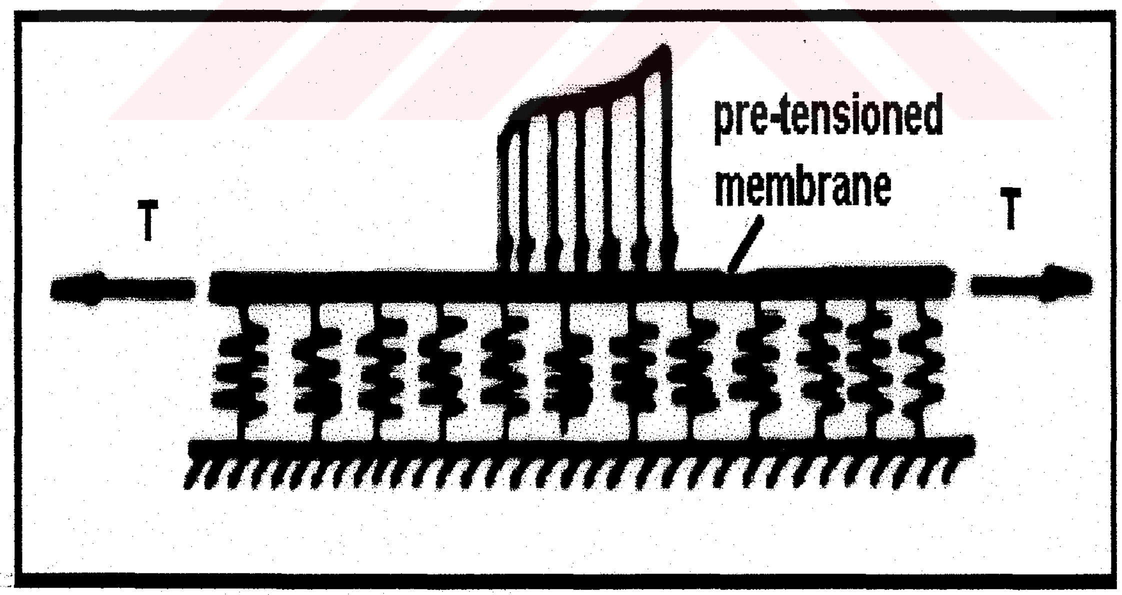

Figure 5.6 The Filonenko-Borodich subgrade model

The two elastic constants necessary to characterize the soil mass are k an T in the Filonenko-Borodich model.

The Filonenko-Borodich model has the following equation can be used to model a layer the membrane over a relatively compressible springs.

$$q(x,y) = kw(x,y) - T\nabla^2 w(x,y) \quad (5.2)$$

where

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} \quad (5.3)$$

where, q is the applied vertical surface pressure, w is the vertical surface deflection, k is the coefficient of subgrade reaction with units of stress per unit volume, T is the membrane tension, ∇^2 is the Laplace operator (Selvaduri,1979).

5.2.2.2 Hetenyi model

Hetenyi model behaves that the interaction between the individual springs is accomplished by incorporating an elastic plate.

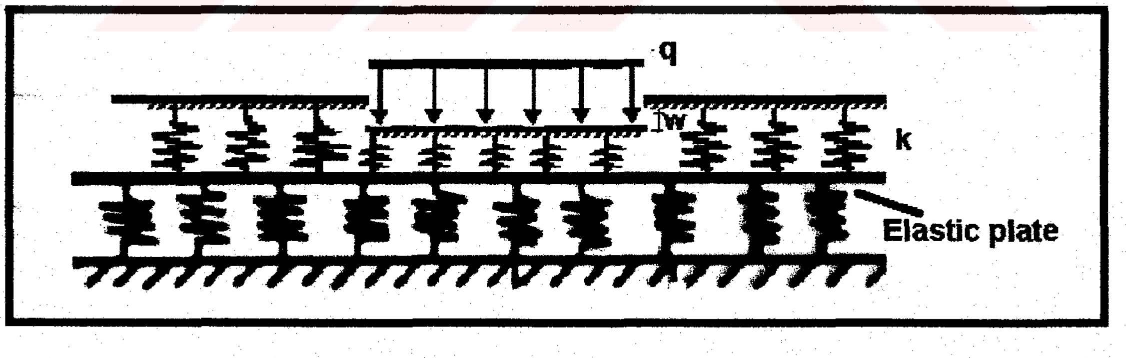

Figure 5.7 Hetenyi subgrade model

The response function in Hetenyi model is defined as

$$q(x,y) = kw(x,y) - D\nabla^2\nabla^2 w(x,y) \quad (5.4)$$

where, q is the applied vertical surface pressure, w is the vertical surface deflection, k is the coefficient of subgrade reaction with units of stress per unit volume, ∇^2 is the Laplace operator.

$$D = \frac{E_p t^3}{12(1-\vartheta_p^2)} \qquad (5.5)$$

where, D is the flexural rigidity of plate, E_p is the modulus of elasticity of plate, t is the thickness and ϑ_p is the Poisson's ratio of the plate (Selvaduri, 1979).

5.2.2.3 Pasternak model

Pasternak model assumes two distinct relatively thin subgrade strata which has been applied involves a dense coarse-grain soil over a compressible fine-grain soil. The Pasternak model which is shown in figure 5.7, as an incompressible shear layer of stiffness *g* over a spring layer of stiffness *k* and has the following governing equation is typically used to model this situation.

$$q(x,y) = kw(x,y) - g\nabla^2 w(x,y) \qquad (5.6)$$

$$k = \frac{E}{H} \qquad (5.7)$$

$$g = \frac{GH}{2} \qquad (5.8)$$

where, E is the Young's modulus of the fine-grain stratum, H_1 and H_2 is the elastic layer thickness of the stratums, G is the shear modulus of the course-grain stratum.

This is based on the assumption that the relatively incompressible (coarse-grain) stratum will effectively provide all the shearing resistance of the overall subgrade whereas the relatively soft (fine-grain) stratum will contribute all settlement (Kerr, 1964).

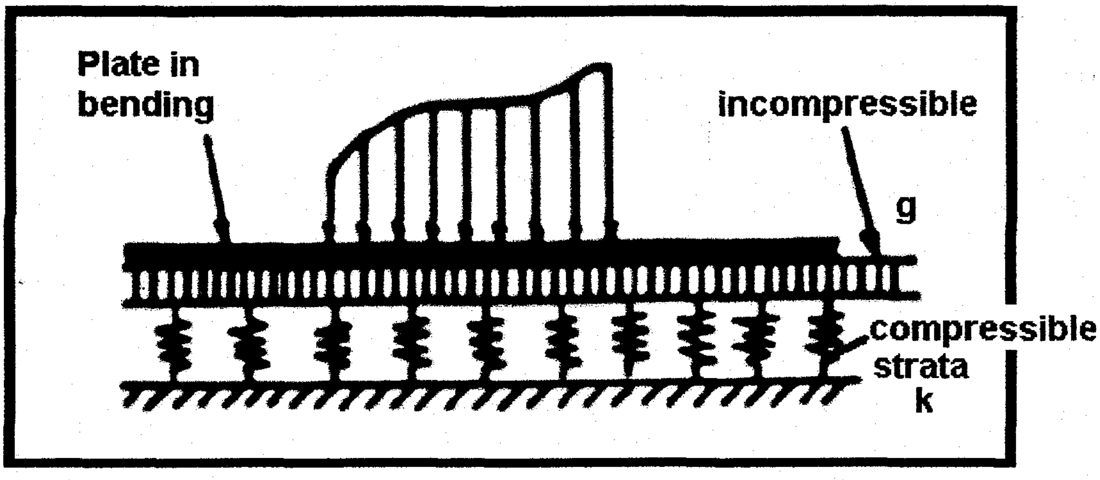

Figure 5.7 Pasternak subgrade model

5.3 Simplified Continuum Models

5.3.1 Vlasov model

The soil-structure interaction problem is considered as an three-dimensional in this model. The terms (u, v, w) represents displacements in the x-, y-, z-directions, respectively in the soil.

Vlasov model is defined by

$$D\nabla^4 w - 2t\nabla^2 w + kw = q \quad \text{.....(5.9)}$$

where

$$2t = \int_0^H \frac{E_s}{2(1+\vartheta_s)}\phi^2 dz \quad \text{.....(5.10)}$$

$$k = \int_0^H \frac{(1-\vartheta_s)E_s}{(1+\vartheta_s)(1-\vartheta_s)}\left(\frac{d\phi}{dz}\right)^2 dz \quad \text{.....(5.11)}$$

where, D is the flexural rigidity, ∇^4 is the bio-harmonic operator, w is the vertical surface deflection, 2t is represents the neglected shear modulus between the springs in Winkler model, ∇^2 is the Laplace operator, k is the coefficient of subgrade reaction with units of stress per unit volume, q is the applied vertical surface pressure, ϑ_s is the Poisson's ratio, E_s is the Young's modulus (Celik,1999).

According to Vlasov model, the horizontal displacements are assumed as

$$u(x,y,z)=0 \qquad (5.12)$$

$$v(x,y,z)=0 \qquad (5.13)$$

and vertical displacement at any point in the z-direction of the soil is defined by

$$w(x,y,z)=w(x,y)\phi(z) \qquad (5.14)$$

the variation of vertical displacement in the vertical z-direction over the foundation depth H is determined by a function of $\phi(z)$.

$$\phi(z)=\frac{\sinh\gamma(1-\frac{z}{H})}{\sinh\gamma} \qquad (5.15)$$

where

$$(\frac{\gamma}{H})^2=\frac{(1-2\vartheta_s)}{2(1-\vartheta_s)}\frac{\int_{-\infty}^{+\infty}\int_{-\infty}^{+\infty}[(\frac{\partial w(x,y)}{\partial x})^2+(\frac{\partial w(x,y)}{\partial y})]dxdy}{\int_{-\infty}^{+\infty}\int_{-\infty}^{+\infty}w^2(x,y)dxdy} \qquad (5.16)$$

where, γ is represent the variation of the deformation of the soil along the z-direction, ϑ_s is the Poisson's ratio, H is the depth of the soil.

The advantages of Vlasov model over the Winkler and other two-parameter soil models is that the parameters, k and t are determined directly from the modulus of elasticity and Poisson's ratio of the soil and have nothing to the with the dimensions of the loaded region. The disadvantages of this model is that it introduced a parameter, γ, whose value must be estimated. However, an iterative procedure is employed here to obtain the correct value of γ. By assuming γ=1, the displacement w(x) is calculated and by using equation (5.16) a new value of γ is calculated. Iteration is continued until the difference between the *i*th and (*i*+1)th value of γ is $\|\gamma_{i+1} - \gamma_i\| \leq 0.001$, Vallabhan, Das, (1988), Vallabhan et al (1991).

5.3.2 Reissner model

Reissner model assumed that plane stresses $(\sigma_x, \sigma_{y,} and \tau_{xy})$ through out the soil layer are negligibly small and that the displacement components satisfy

$$z = H.........u = v = w = 0 \quad \text{.....(5.17)}$$

$$z = 0..........\sigma_z(x,y) = -q(x,y)$$
$$W = w(x,y,0) = 0$$
$$u(x,y,0) = v(x,y,0) = 0 \quad \text{.....(5.18)}$$

Reissner concluded that the response function for the soil is given by

$$c_1 w - c_2 \nabla^2 w = q - c_3 \nabla^2 q \quad \text{.....(5.19)}$$

where

$$c_1 = \frac{E}{H} \quad \text{.....(5.20)}$$

$$c_2 = \frac{GH}{3} \quad \text{.....(5.21)}$$

$$c_3 = \frac{GH^2}{12E} \quad \text{...(5.22)}$$

$$G = \frac{E}{2(1+\vartheta_s)} \quad \text{..(5.23)}$$

where w is the vertical displacement of the soil surface and q is the external surface loading. The constants c_1, c_2 and c_3, the two parameters characterizing the soil response are related to E_s and v_s. Here again as a consequence of assuming that the in-plane stresses $\sigma_x, \sigma_{y,} and \tau_{xy}$ are zero, the shear stresses $\tau_{xz} and \tau_{yz}$ are independent of z. These stresses for a given location x, y are constant throughout the depth H. Such an assumption is unrealistic for particularly a thick layer of soil (Horvath, 1983a), (Selvaduri,1979).

CHAPTER 6

RESULTS AND DISCUSSION

6.1 Introduction

A mat foundation with 26 meter width and 26 meter length is chosen for this study. It is assumed that foundation is constructed on either clay or sand. Based on this assumption commercial software SAFE v7.01 is used for the solution.

SAFE (**S**lab **A**nalysis by the **F**inite **E**lement method) is a special purpose program that automates the analysis and design of simple to complex concrete foundation systems using powerful object based modeling. The program can analyze and design mats of arbitrary shapes and varying thickness, drop panels, openings, edge beams and discontinuities. Foundations can be combinations of mats, strip footings or isolated spread footings.

The analysis is based upon the Finite Element method in a theoretically consistent fashion that properly accounts for the effects of twisting moments. Foundations are modeled as thick plates on elastic foundations, where the compression only soil springs are automatically discretized based upon a modulus of sub grade reaction that is specified for each foundation object.

SAFE automatically defines design strips in perpendicular directions, cross sectional moments and shears are integrated along the length of each design strip for each load case and combination, and reinforcement is designed based on selected code.

Assign soil subgrade modulus for an area and SAFE automatically adjusts nodal soil spring constants as part of auto-meshing. No more time consuming error prone manual calculation of soil spring constants based on tributary areas which change with every modification. Option to assign variable soil support conditions.

6.2 Problem Definition

In this chapter a base mat is analyzed. Dimensions of the foundation is held constant, and soil type and related subgrade modulus is chanced, various thicknesses, span lengths and loads are applied to mat to observe the effects of mentioned parameters on mat foundation design.

Mat foundation is subjected to vertical loading. Loadings are equal at each node. Vertical loads consist of dead and live loads. Live loads are accepted constant for each analyses but dead load takes various magnitudes as mentioned above. Concrete design code Eurocode 2-1 is used to design the mat in these analysis. According to Eurocode load combination is defined as 1.35DL + 1.5LL and, γ_c and γ_s is described 1.5 and 1.15 respectively. The other parameters, which is accepted constant relevant the mat foundation are explained in table 6.1.

Table 6.1 Constant parameters that used in mat foundation analysis

Modulus of elasticity $_{concrete}$	25000000	kN/m^2
Poisson's ratio $_{concrete}$	0.2	----------
Unit weight $_{concrete}$	24	kN/m^3
Concrete strength, f_c	27500	kN/m^2
Reinforcing yield stress	400000	kN/m^2

6.3 Case Studies

In case studies, effects of ks, loads, thickness of mat and spans between the columns on mat foundation are able to defined. According to effects of these parameters, displacements, soil pressures and moments are computed by SAFE. Then, these values have examined and results have been illustrated by graphics. The graphics consist of CSX1 strip of the mat foundation. The distribution of displacements, soil pressures and moments are illustrated along the strip and according to the coefficient of subgrade reaction. Analyzed mat foundation is shown in figure 6.1. The size of the mat is 26m x 26m and all columns are 0.6m in this section.

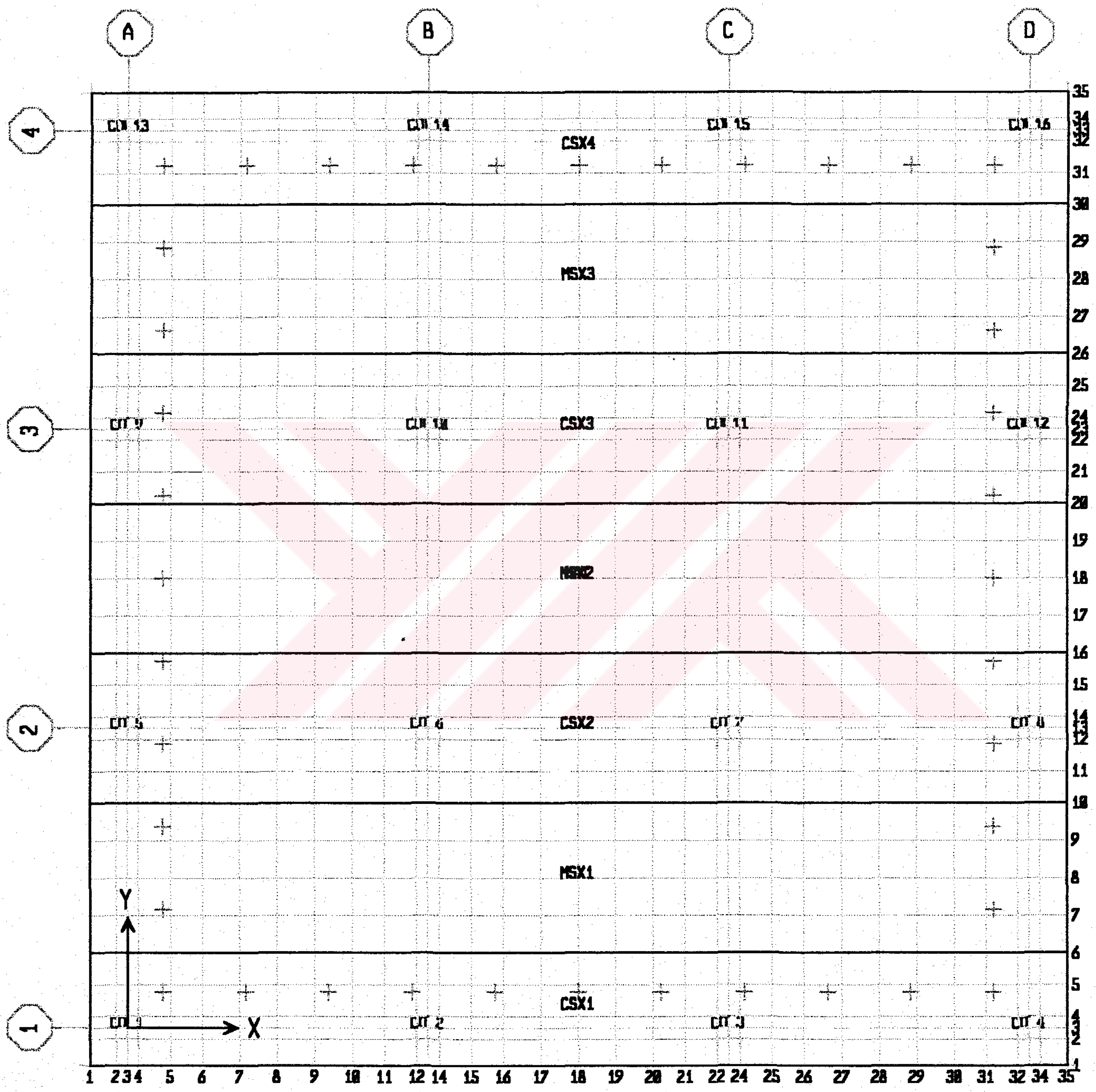

Figure 6.1 Mat foundation plan view

6.3.1 Case 1

A square mat having dimensions 26x26 m as mentioned above. The range of values for modulus of subgrade reaction is between 7500 and 120000 kN/m^3. The range of values for applied loads is between 1000 and 4000 kN. The thickness of the mat is 1m and spans of columns are 8 m. Graphics of displacements, soil pressures and moments are illustrated according to modulus of subgrade reaction and along the strip.

According to Hetenyi (1946), displacements on elastic foundations can be calculated by

$$w = \frac{P\beta}{2k} A_{\beta x} \qquad (6.1)$$

where

$$A_{\beta x} = e^{-\beta x}(\cos \beta x + \sin \beta x) \qquad (6.2)$$

$$\beta = \sqrt[4]{\frac{B * k}{4E_f I_f}} \qquad (6.3)$$

where, w is the displacement on elastic foundation, P is the applied load, k is the subgrade reaction, B is the foundation width, E_f is the elasticity modulus of foundation and I_f is the inertia moment of foundation.

Figure 6.2.a and figure 6.2.b show the relationship between the displacement and subgrade reaction according to various loading. Equation (6.1) shows that the proportion between the displacements and P, h, B, k parameters. If ks values increase, displacement values exponentially decrease.

The effect of loading factor is investigated in figure 6.2.a and figure 6.2.b. These figures show that when the soil modulus is equal to 7500kN/m^3 the difference between the displacements, which are occurred by P=1000 and 4000kN is higher than when the soil modulus is equal to 120000kN/m^3. When ks values get a move on increasing, the displacements difference decreases by degrees. It is explained as; while k is equal to 7500kN/m^3, displacements difference is 0.0337m. While k is equal to 80000kN/m^3, displacements difference is 0,0062m. While k is equal to 120000kN/m^3, displacements difference is 0.0047m. It is noted that all values is accepted as an absolute.

As mentioned above, when ks values pass-over the 80000kN/m^3, loading factor get a move on the loose its effectiveness on displacements. In an other word, these displacement values are so small to effect the mat design. So we can say that it can be ignored. According to these comments, it can be said that the effect of loading is not effective after the 80000kN/m^3 and more.

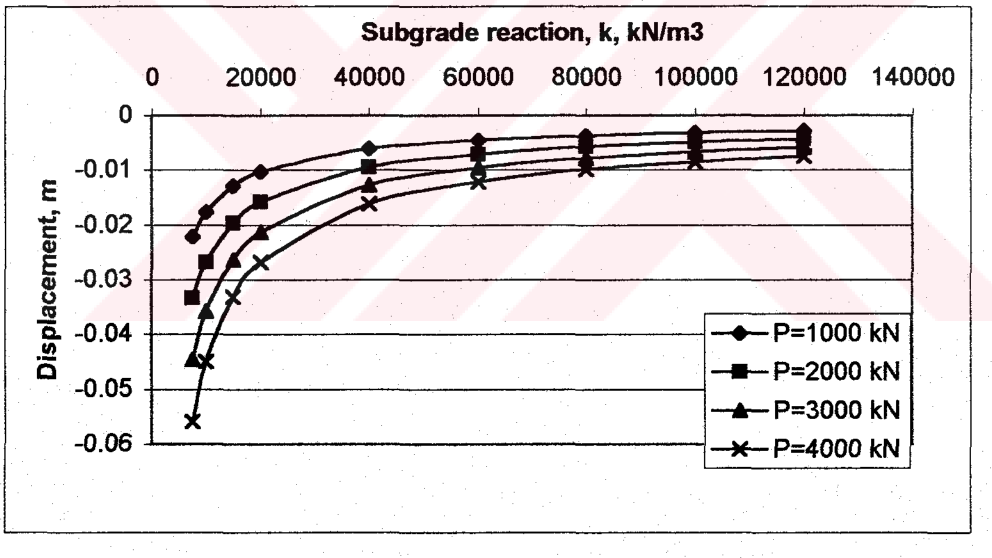

Figure 6.2.a Displacement-Subgrade reaction profiles for several load intensities

Figure 6.2.b illustrate the displacements on joint and equation (6.1) is acceptable for this figure, too.

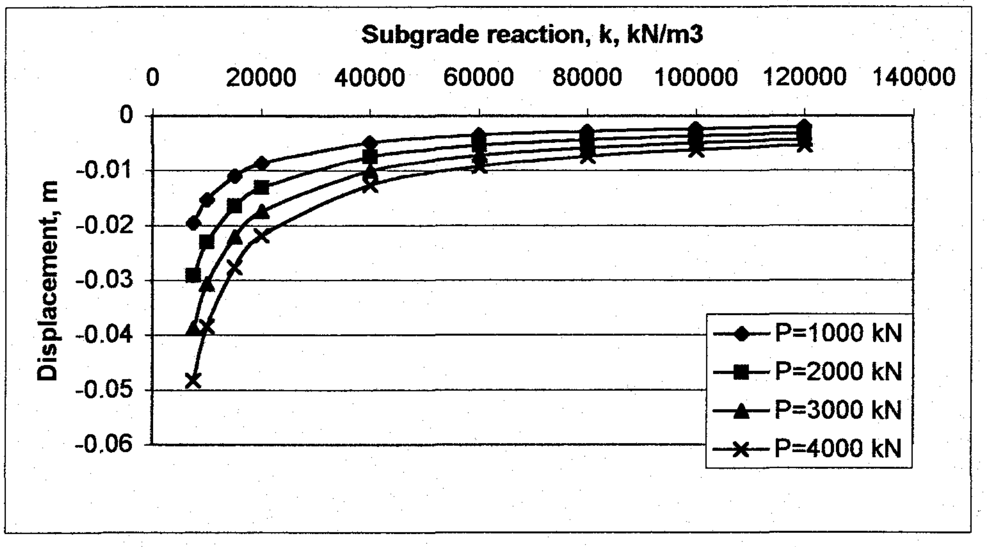

Figure 6.2.b Displacement-Subgrade reaction profiles for several load intensities on grid I3-J3

Furthermore, as cited by equation (6.1), displacement values increase across the increasing loading and increasing is closely linear that shown as in figure 6.3.a and figure 6.3.b. As mentioned above, the line, which is represents the k=120000kN/m^3 shows that load increment is not effect enough.

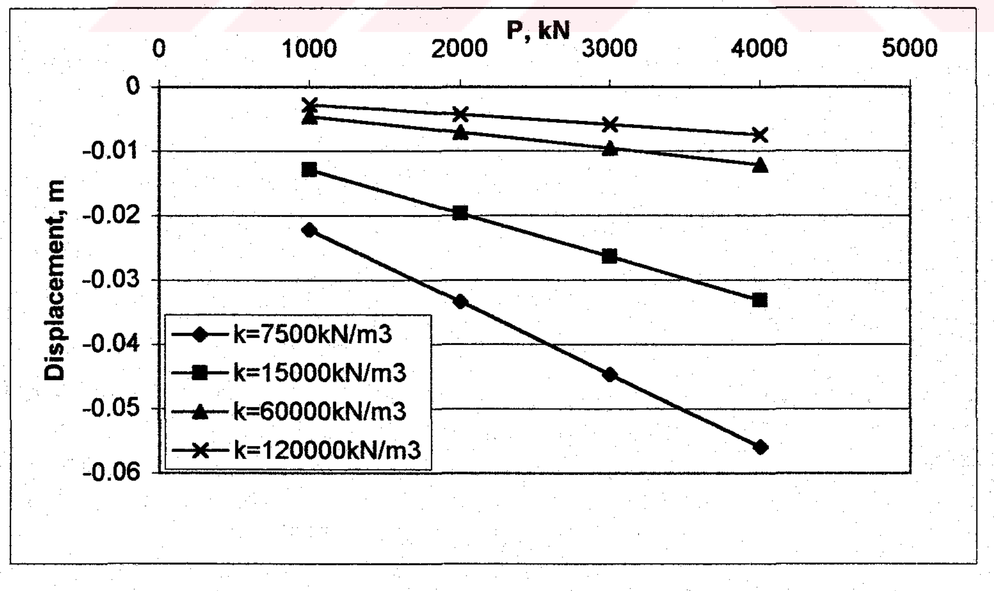

Figure 6.3.a Displacement-Load profiles for several ks values

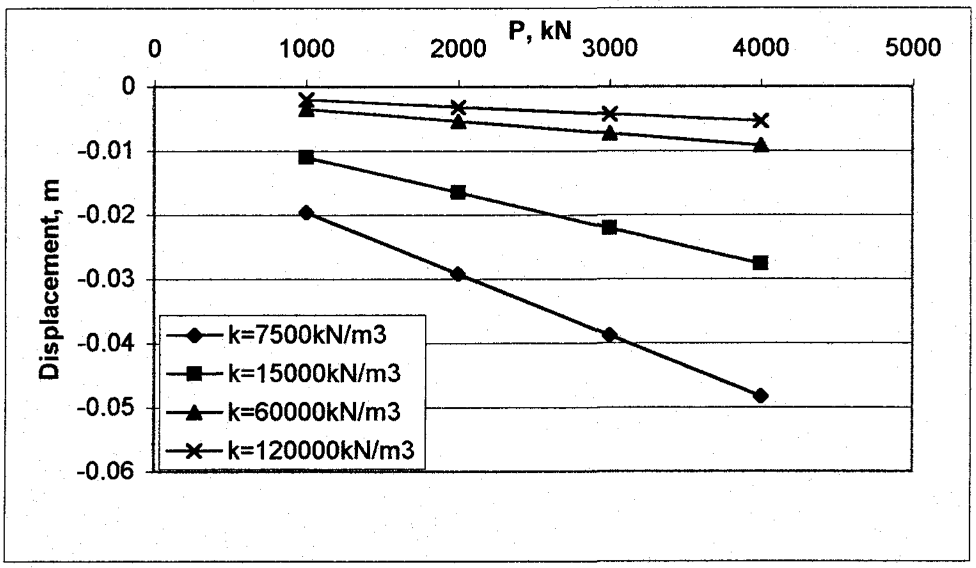

Figure 6.3.b Displacement-Load profiles for several ks values on grid I3-J3

According to q=kw formula, soil pressure values increase exponentially with ks values. In figure 6.4.a and figure 6.4.b are illustrated the increasing soil pressure. Figure 6.4.b represents soil pressure under the joint on grid I3-J3. Soil pressure values increase across the increasing loading and increasing is closely linear that shown as in figure 6.5.a and 6.5.b. These figures are supported by equation (6.1). In mentioned equation, load and settlement parameters are right proportional.

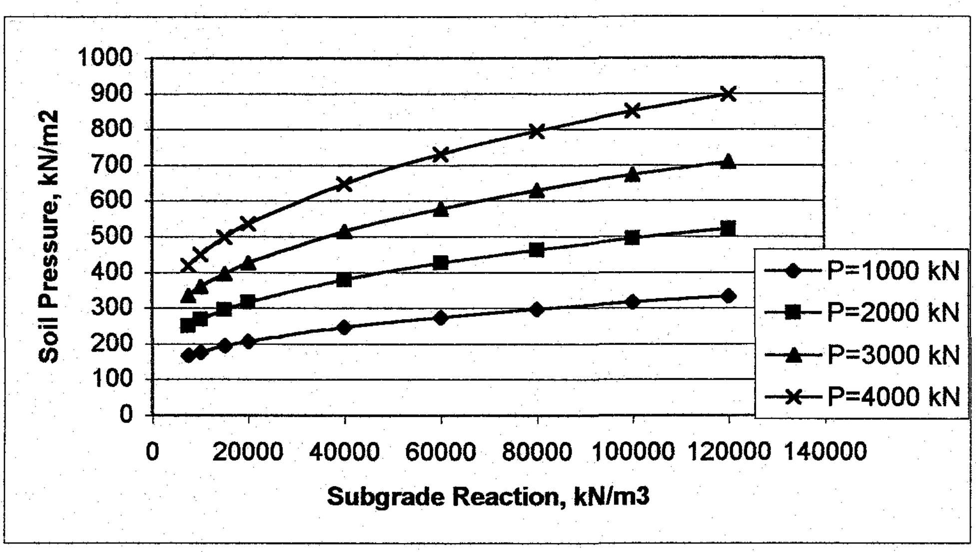

Figure 6.4.a Soil pressure-Subgrade reaction profiles for several load intensities

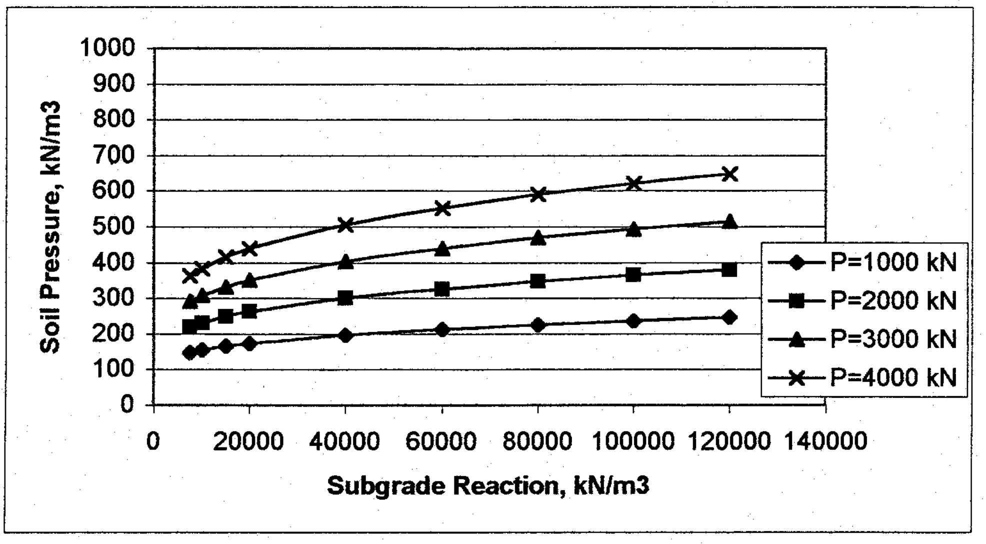

Figure 6.4.b Soil pressure-Subgrade reaction profiles for several load intensities on grid I3-J3

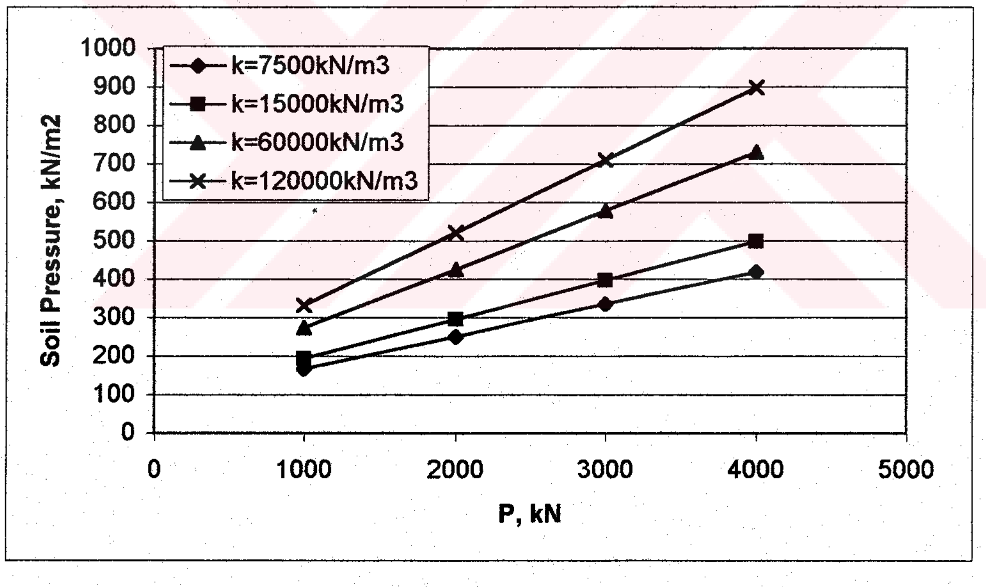

Figure 6.5.a Soil pressure-Load profiles for several k values

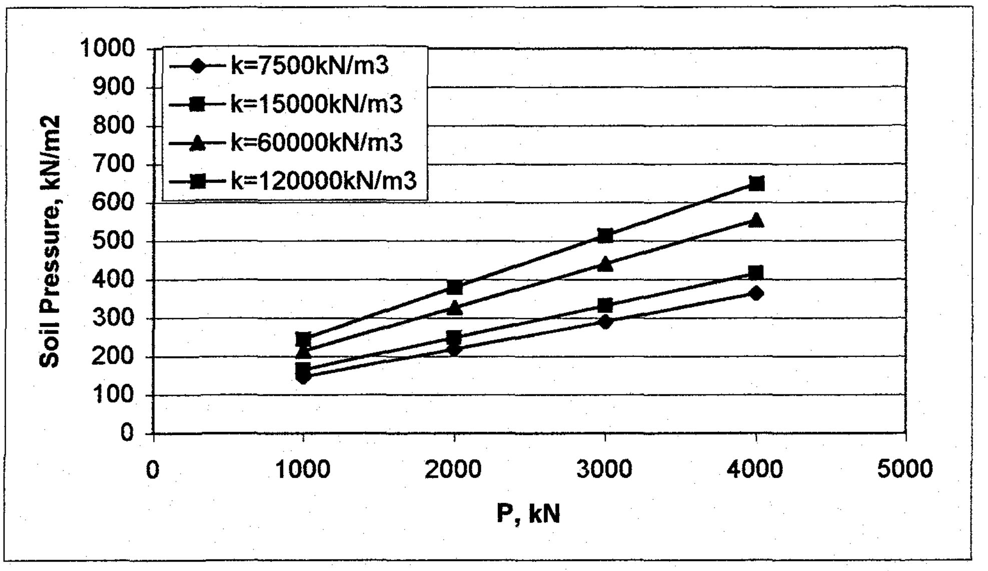

Figure 6.5.b Soil pressure-Load profiles for several k values on grid I3-J3

Figure 6.5.b represents the behavior of soil pressure in a special case. Subgrade modulus, ks, loose its effect on the soil pressure when it is smaller than 15000kN/m^3 and load is smaller than 1000kN.

According to Hetenyi, moments on elastic foundations can be calculated by

$$M = \frac{P}{4\beta} C_{\beta x} \qquad (6.4)$$

where

$$C_{\beta x} = e^{-\beta x}(\cos\beta x - \sin\beta x) \qquad (6.5)$$

where M is the moment and other parameters was mentioned above. Equation (6.4) is referred to comment about moments on mat foundation. If ks values increase, moment values exponentially decrease. Figure 6.6 shows this relationship. Moments increase linearly when the loads increase. Figure 6.7 shows this relationship.

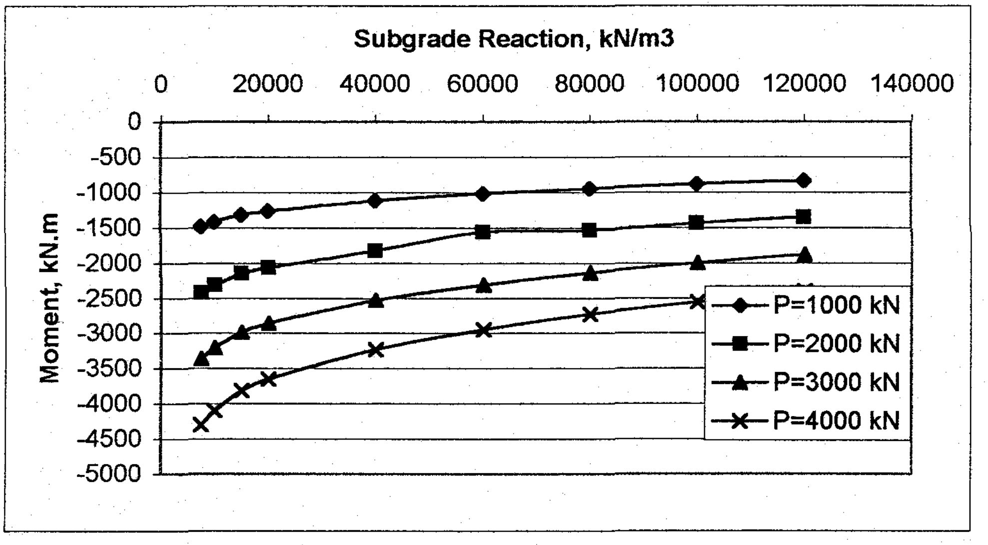

Figure 6.6 Moment-Subgrade reaction profiles for several load intensities

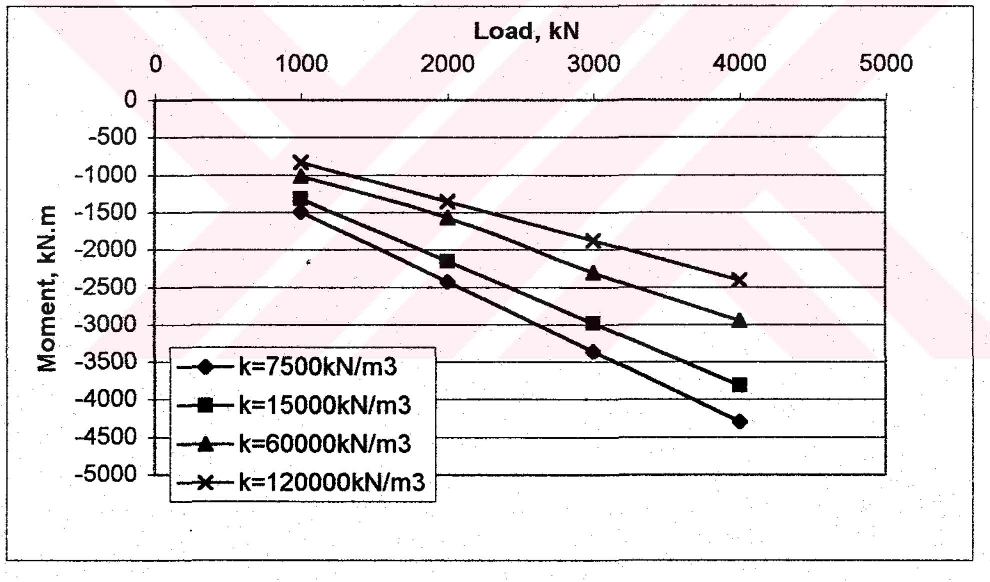

Figure 6.7 Moment-Load profiles for various k values

Each load pattern is illustrated separately , which are shown in figure 6.6. In this way the relationship between the moment and subgrade reaction is shown better than the figure 6.6. Figure 6.8.a, figure 6,8.b, figure 6.8.c, figure 6.8.d are illustrated relatively with figure 6.6.

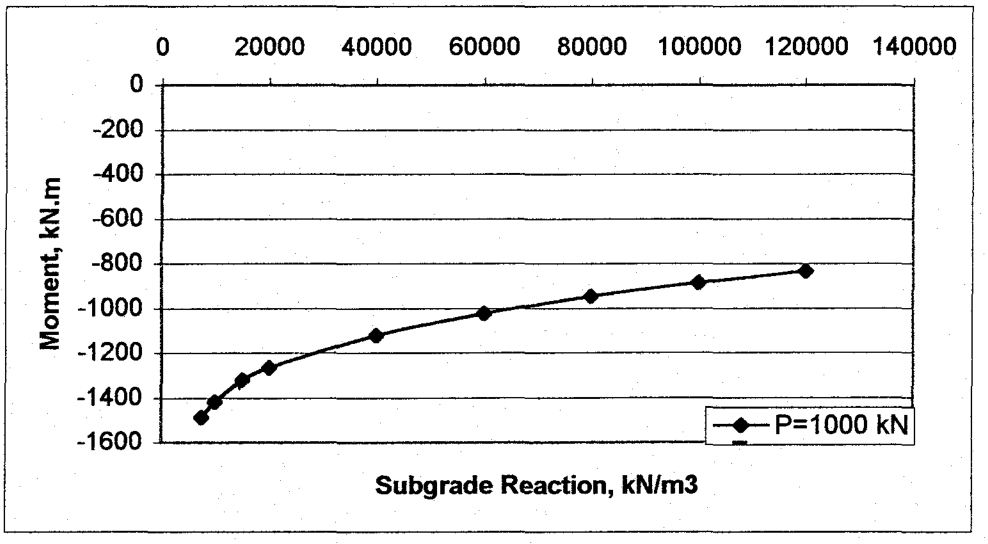

Figure 6.8.a. Moment-subgrade reaction profiles for 1000kN load

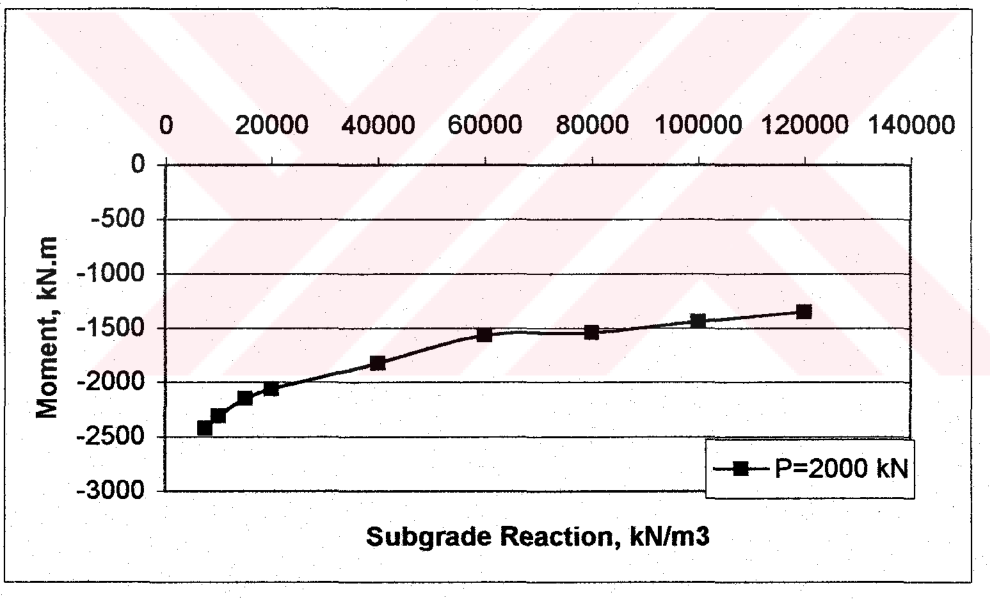

Figure 6.8.b. Moment-subgrade reaction profiles for 2000kN load

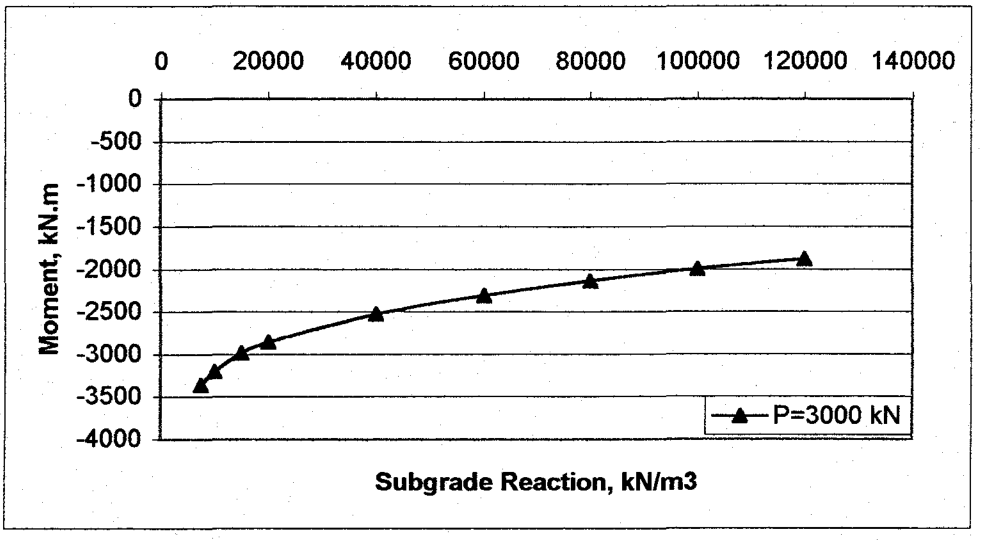

Figure 6.8.c Moment-subgrade reaction profiles for 3000kN load

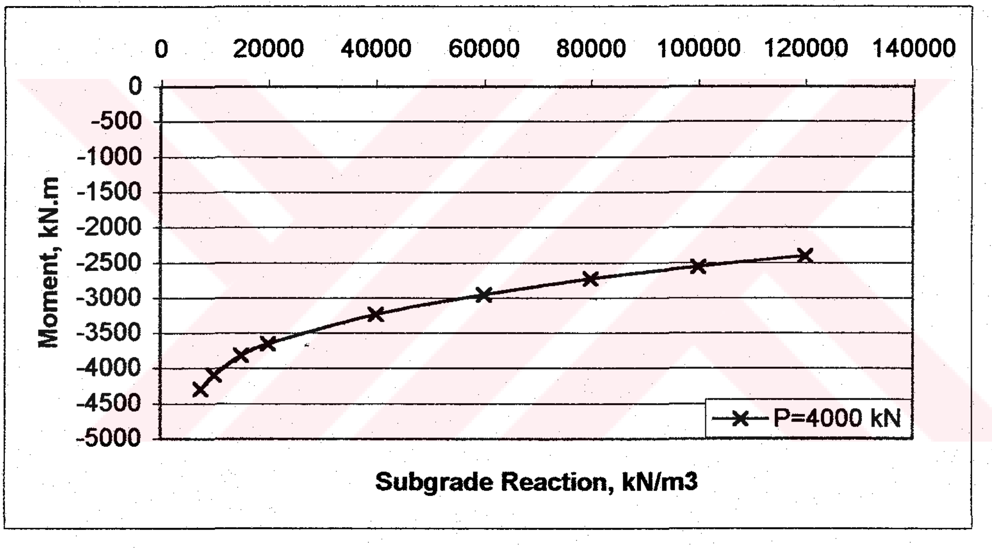

Figure 6.8.d Moment-subgrade reaction profiles for 4000kN load

Displacements are examined along the CSX1 strip of mat foundation for different load patterns and ks values. Figure 6.9 and figure 6.10 show that displacements effect from the different ks values under the constant loading. Figure 6.11 and figure 6.12 show that displacements effect from the various load intensities. Equation 6.1 is referred for this comment.

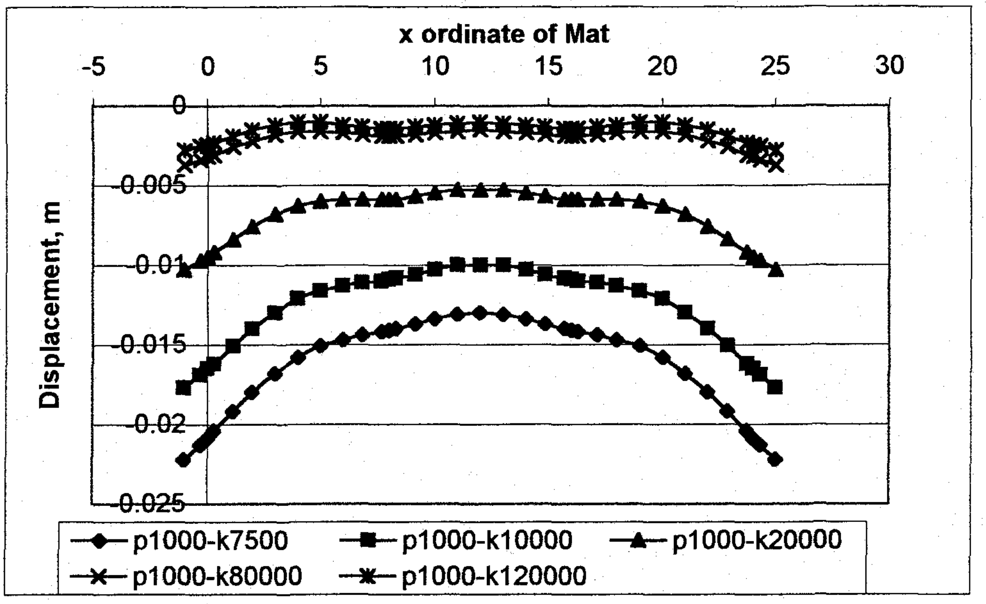

Figure 6.9 Displacement variation profiles for various subgrade reaction values for P=1000kN along the strip CSX1

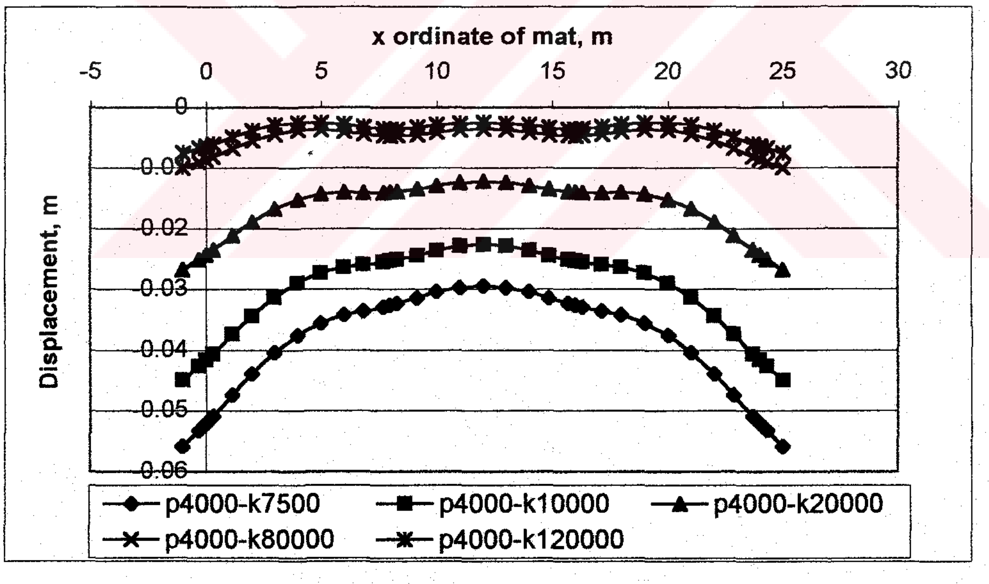

Figure 6.10 Displacement variation profiles for various subgrade reaction values for P=4000kN along the strip CSX1

Figure 6.9 and figure 6.10 show that the decrease of displacement values became less after 80000kN/m^3. So we can ignore it. It is noted that, in figure 6.10, when ks values is less than 10000kN/m^3 the differential settlement is occurred. Differential settlement limit is 0,02m and the curve, which is a representative for 7500kN/m^3 overcomes this limit with 0,0265m value. Also, the curve, which is a representative for 10000kN/m^3 overcomes this limit with 0,0224m value.

According to calculated results, we can say that the foundation, which has low ks values under the high load intensities can cause a big problem for mat design.

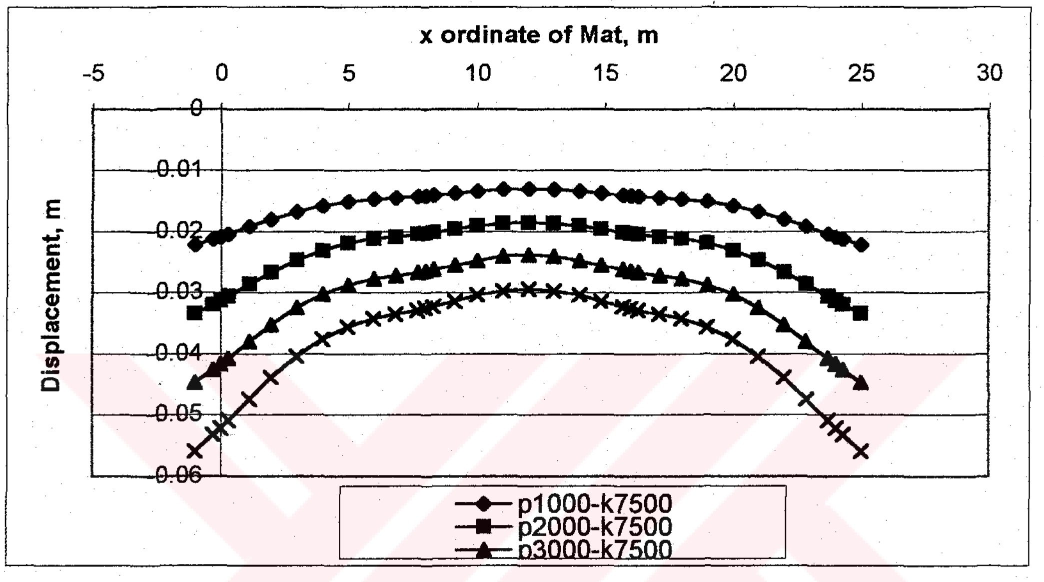

Figure 6.11 Displacement variation profiles for several load intensities across the constant ks value, 7500kN/m^3 along the strip CSX1

Increment of loading cause effective displacement values on low ks value, 7500kN/m^3. These values is not ignored. Also, differential settlement is illustrated in figure 6.11 with P=3000 and 4000kN/m^3.

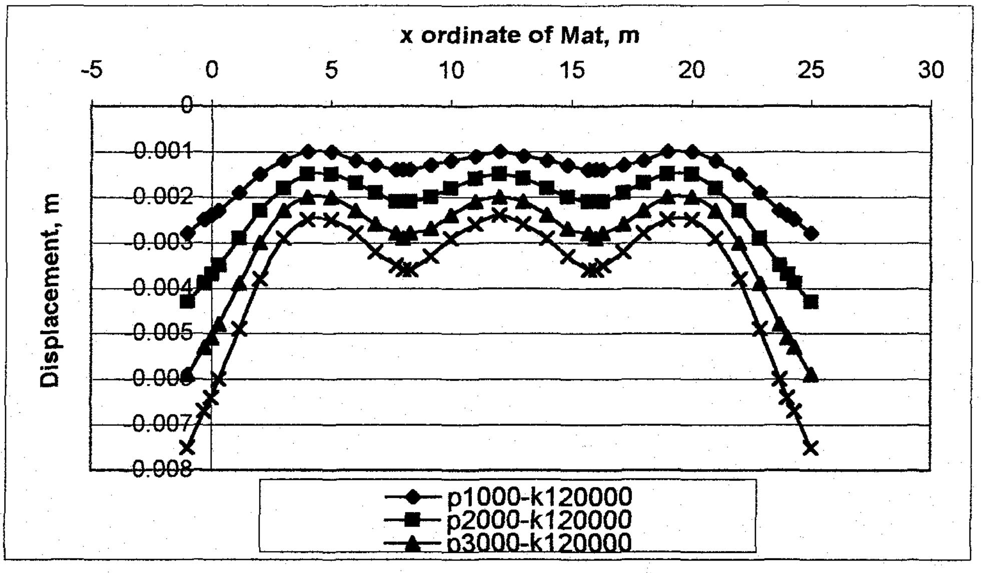

Figure 6.12 Displacement variation profiles for several load intensities across the constant ks value, 120000kN/m^3 along the strip CSX1

Figure 6.12 shows that the increment of load intensity is not effective in an important degree with high ks values. That is because displacements, which are represented the applied loads can be ignored.

Soil pressure values increase across the increasing ks value under constant loading and soil pressure values increase across the increasing loading with constant ks. These comments are supporting with q=kw and illustrated with graphics, respectively.

In figure 6.13, soil pressure distributions show that its variation is not effected enough to chance the mat design until the value of ks pass-over the 20000kN/m^3 under low loading intensities.

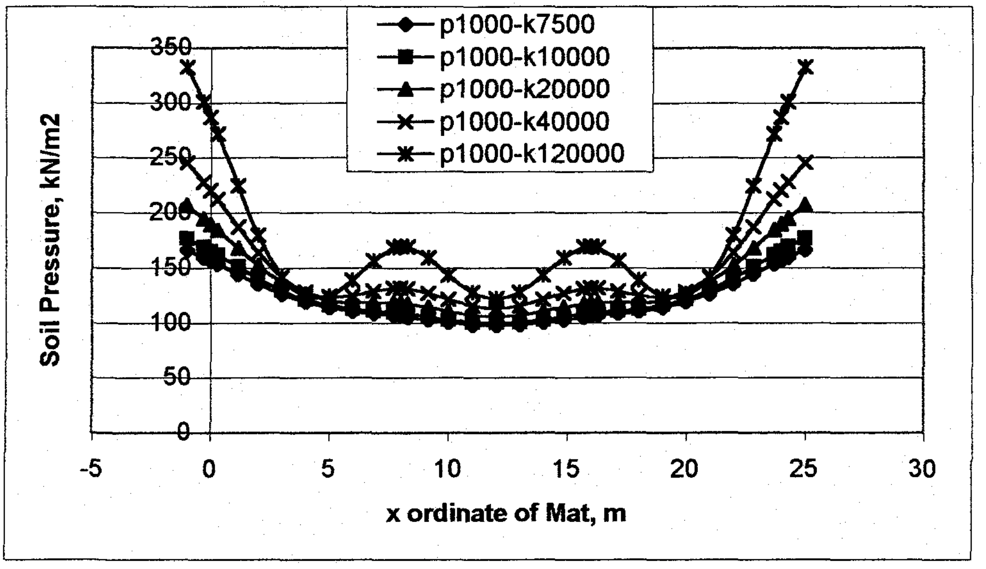

Figure 6.13 Soil pressure distribution according to various subgrade reaction for constant loading along the CSX1 strip

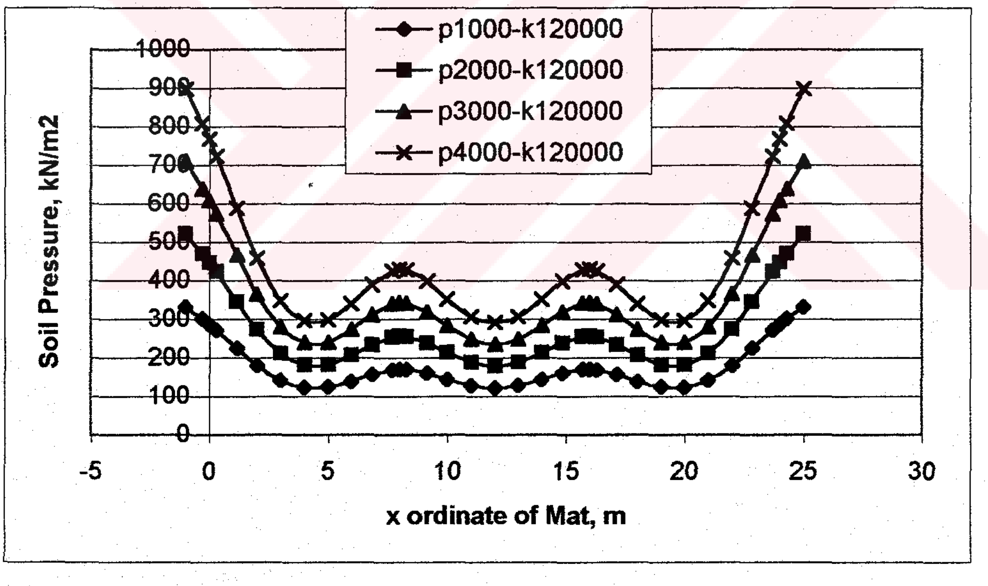

Figure 6.14 Soil pressure distribution according to various loading for constant subgrade reaction along the CSX1 strip

Moments are examined along the CSX1 strip of mat foundation for different ks values and load patterns. Figure 6.15 and figure 6.16 show that moments how they effect from the different ks values under the constant loading and effect from the various load intensities under the constant ks values. Equation (6.4) is referred to comment about moments on mat foundation. If ks values increase, moment values decrease. Figure 6.15 shows this relationship.

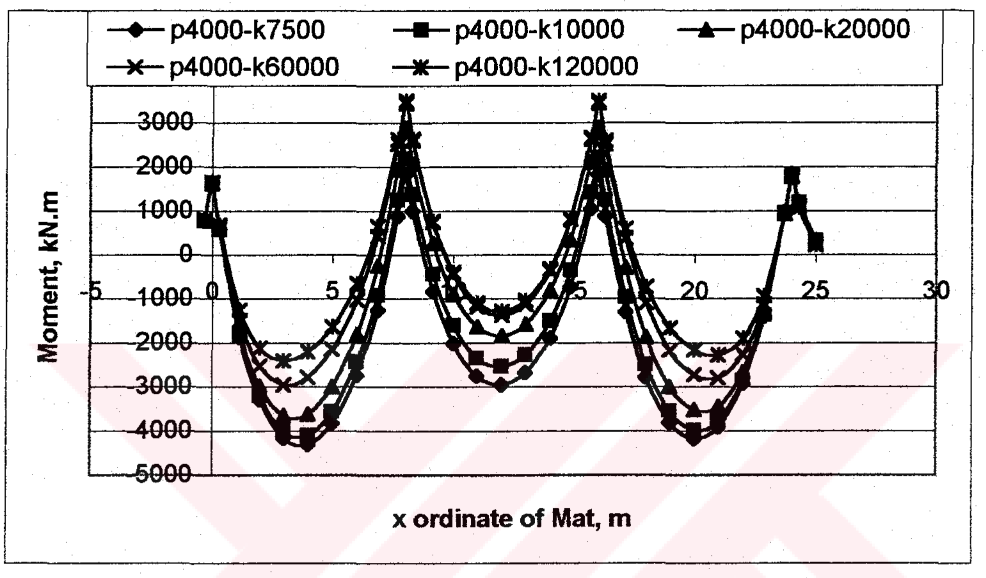

Figure 6.15 Moment distribution according to various subgrade reaction for constant loading along the CSX1 strip

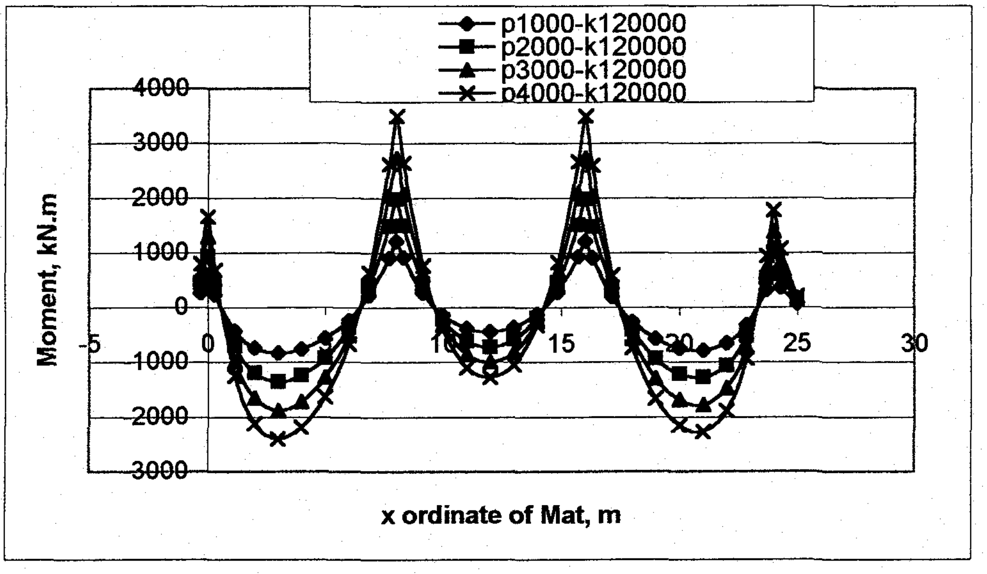

Figure 6.16 Moment distribution according to various loading for constant subgrade reaction along the CSX1 strip

Moments increase with increasing load values. This relationship is shown in figure 6.16.

6.3.2 Case 2

A square mat having dimensions 26x26 m as mentioned before. The range of values for modulus of subgrade reaction is between 7500 and 120000 kN/m^3. The range of values for thickness of mat is between 0.3 and 1.2 m. The applied load is 3000kN/m^3 and spans of columns are 8 m. Graphics of displacements, soil pressures and moments are illustrated according to modulus of subgrade reaction and along the strip.

The formula of displacement on elastic foundation is mentioned before in equation (6.1).

Figure 6.17.a and figure 6.17.b show the relationship between the displacement and subgrade reaction according to different thicknesses. According to proportion of equation (6.1), P, h, B, k parameters effect the displacements. If ks values increase, displacement values exponentially

decrease according to various thickness values.

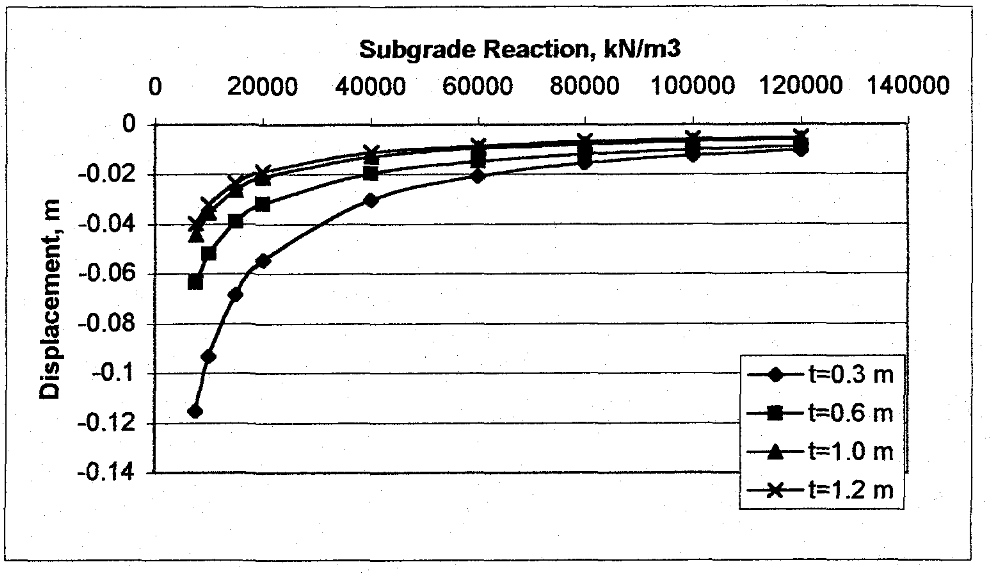

Figure 6.17.a Displacement-Subgrade reaction profiles for several thicknesses

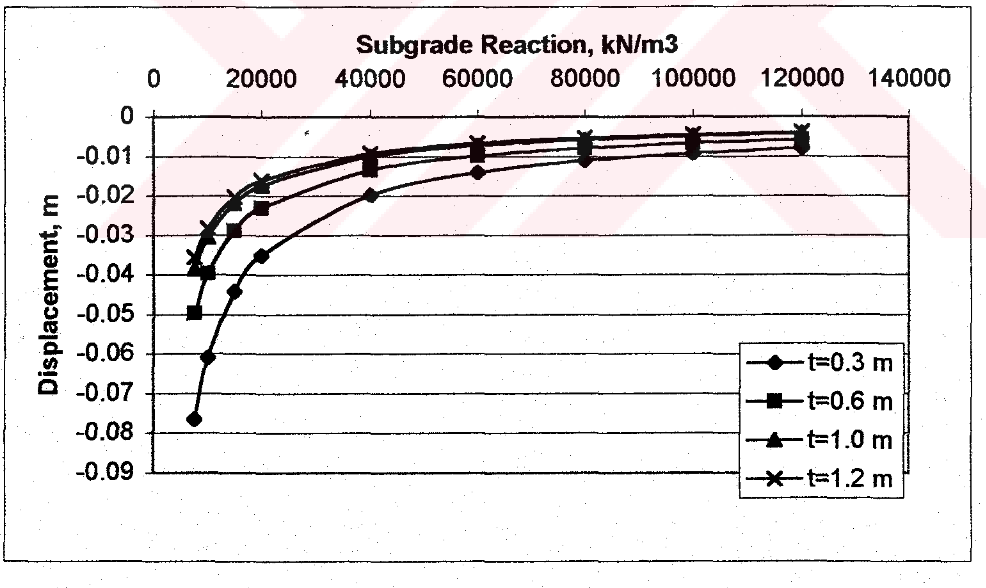

Figure 6.17.b Displacement-Subgrade reaction profiles for several thicknesses on grid I3-J3

The effect of thickness factor on displacements is investigated in figure 6.17.a and in figure 6.17.b. These figures show that when the soil modulus is equal to 7500kN/m^3, the difference between the displacements, which are occurred from the thicknesses is higher than when the soil modulus is equal to 120000kN/m^3. When ks values get a move on increasing, the displacements difference decreases gradually. It is explained as; while k is equal to 7500kN/m^3, displacements difference is 0.0754m. While k is equal to 60000kN/m^3, displacements difference is 0.0123m. While k is equal to 80000kN/m^3, displacements difference is 0.0088m. While k is equal to 120000kN/m^3 displacements difference is 0.0052m. It is noted that all values is accepted as an absolute.

As mentioned above, when ks values pass-over the 80000kN/m^3, difference between the displacements get a move on the loose its effectiveness. In an other word, these displacement values are too small to effect the mat design. So we can say that the thickness of mat is not effective when ks values pass-over the 80000kN/m^3.

Also, when the thickness of mat pass-over the 1.0m, curves are too close to each other in every ks situation. According to this comment, the thickness of mat is not effective on displacements when its value pass-over the 1.0m in every ks situation.

According to q=kw formula, soil pressure values increase exponentially with ks values. In figure 6.18.a and figure 6.18.b are illustrated the increasing soil pressure. Figure 6.18.b represents soil pressure under the joint on grid I3-J3. Soil pressure values increase across the decreasing thickness. These figures are supported by equation (6.1). In mentioned equation, thickness and settlement parameters are cross proportional.

Figure 6.18.a and figure 6.18.b show that thickness loose its effect on the soil pressure when it is smaller than 20000kN/m^3 and thickness is greater than 1m.

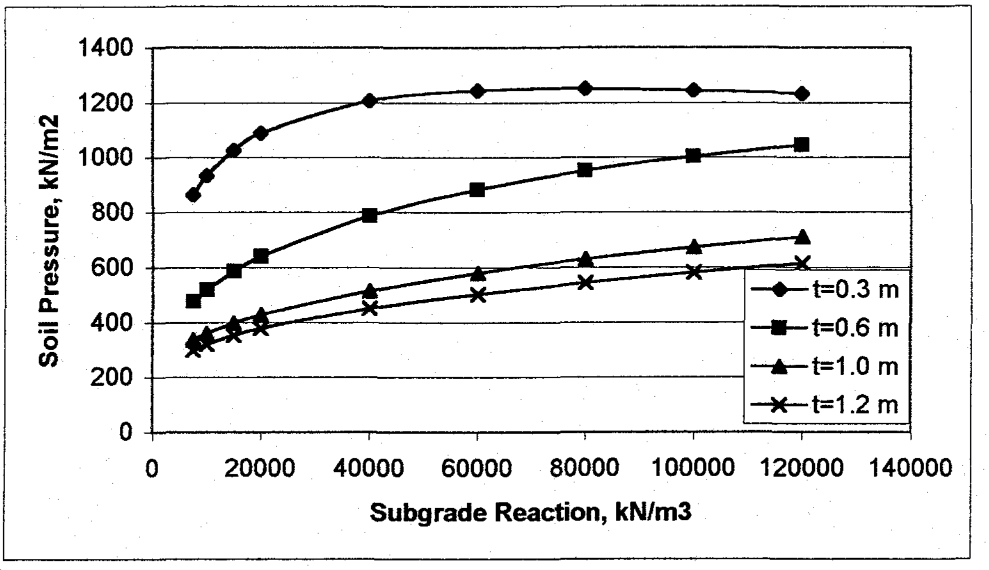

Figure 6.18.a Soil pressure-Subgrade reaction profiles for various thicknesses

In figure 6.18.a, the effect of ks is ignored, when the ks values pass-over the 40000kN/m^3 and the thickness of mat is 0.3m.

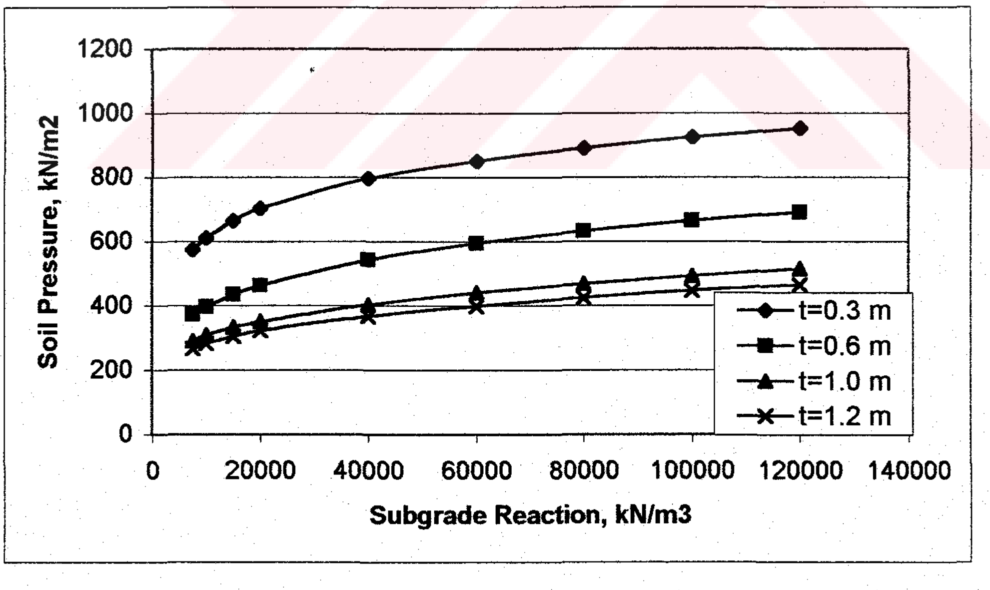

Figure 6.18.b Soil pressure-Subgrade reaction profiles for various thicknesses on grid I3-J3

Moments on elastic foundations can be calculated by Hetenyi's formula as defined by equation (6.4). Equation (6.4) is referred to comment about moments on mat foundation. If ks values increase, moment values exponentially decrease. Figure 6.19 shows this relationship.

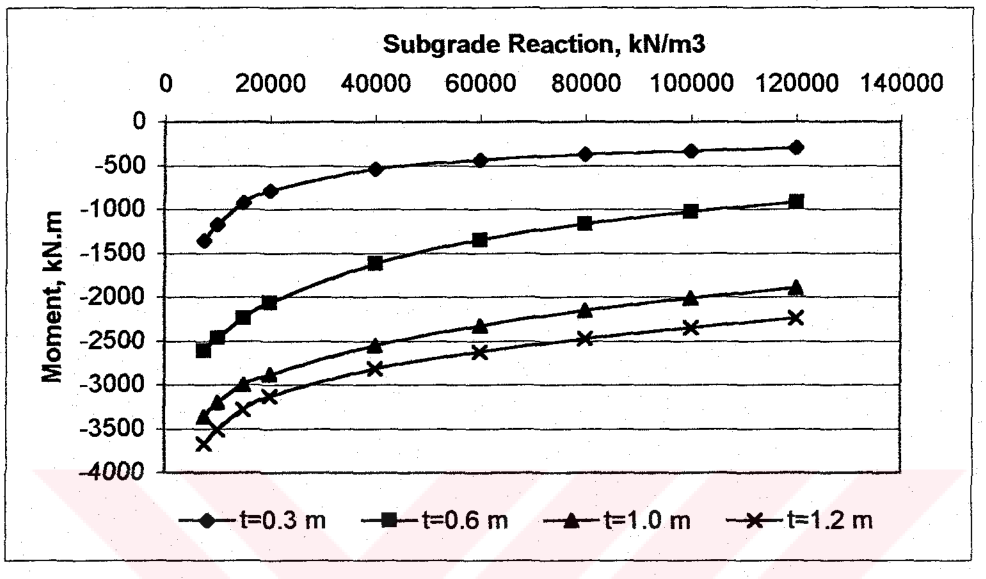

Figure 6.19 Moment-Subgrade reaction profiles for several thicknesses

Figure 6.19 shows that ks loose its effect on the moment when it is greater than 80000kN/m^3 and thickness is smaller than 0.3m.

Displacements are examined along the CSX1 strip of mat foundation for different thicknesses and ks values. Figure 6.20, figure 6.21, figure 6.22 and figure 6.23 show that displacements effect from the different thicknesses under the constant ks values.

Figure 6.20, figure 6.21, figure 6.22 and figure 6.23 show that the decrease of displacement values became less when the thickness is increase. It is noted that, in figure 6.20, when thickness is less than 1.2m and ks values is less than or equal to 7500kN/m^3 the differential settlement is occurred. Differential settlement limit is 0,02m and the curves, which are representative with t=0.3, 0.6 and 1.0m, overcomes this limit. In figure 6.21, when thickness is less than 1.0m and ks values is less than or equal to 20000kN/m^3 the differential settlement is occurred. The curves, which are representative with t=0.3 and 0.6m, overcomes differential settlement limit.

According to calculated results, we can say that the foundation, which has low ks values with thin plate can cause different settlements.

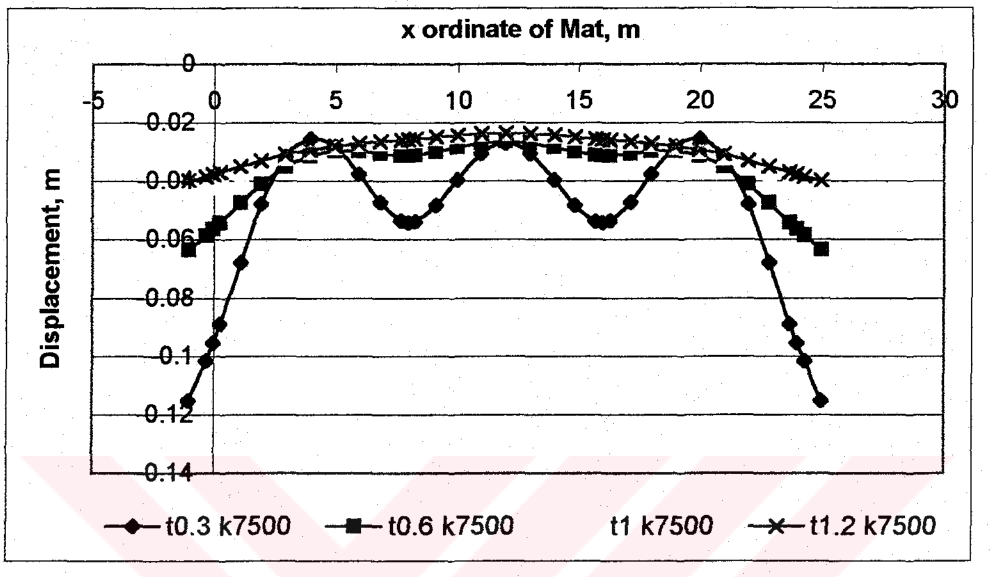

Figure 6.20 Displacement variation profiles for various thicknesses across the constant ks value, 7500kN/m^3 along the strip CSX1

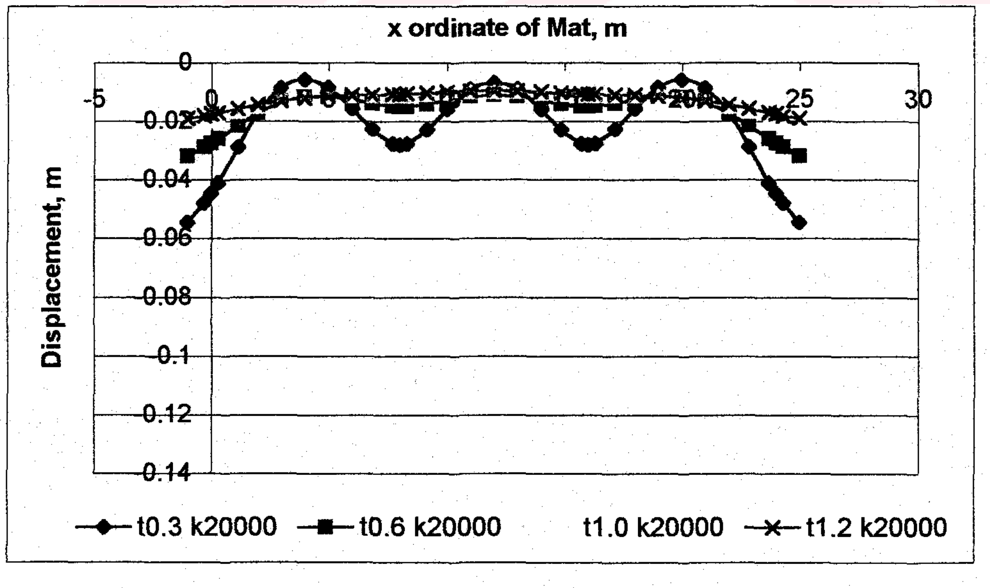

Figure 6.21 Displacement variation profiles for various thicknesses across the constant ks value, 20000kN/m^3 along the strip CSX1

Figure 6.22 and figure 6.23 show that the increment of thickness is not effective in an important degree with high ks values. Displacement curves lay into each other. Therefore, it can be said that thickness is not effective when the ks pass-over the 80000kN/m^3.

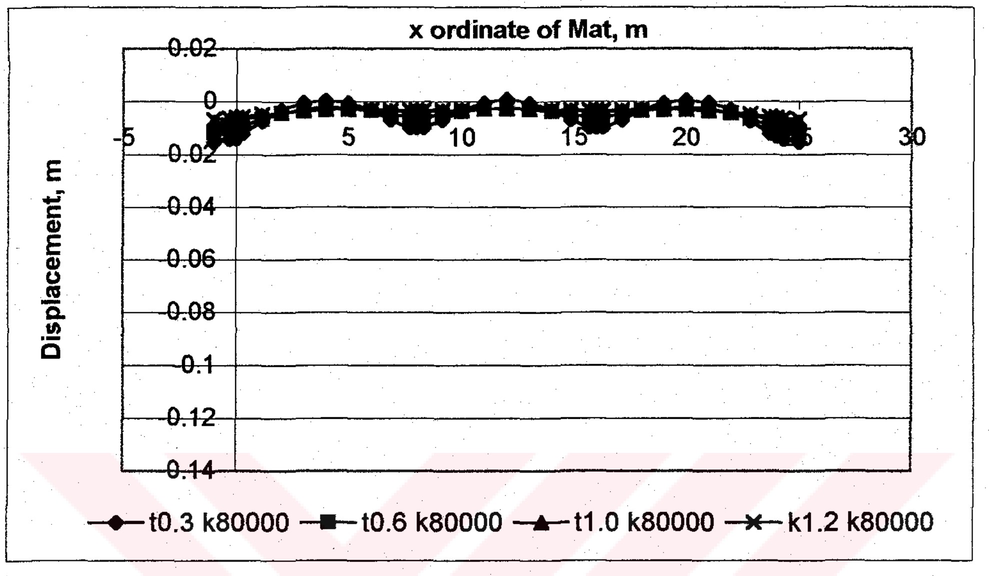

Figure 6.22 Displacement variation profiles for various thicknesses across the constant ks value, 80000kN/m^3 along the strip CSX1

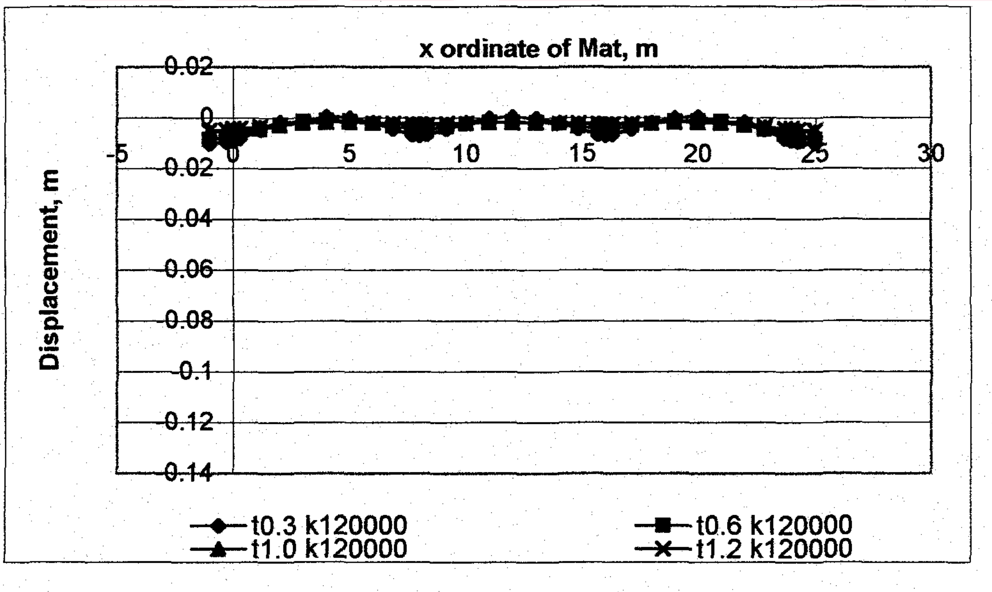

Figure 6.23 Displacement variation profiles for various thicknesses across the constant ks value, 120000kN/m^3 along the strip CSX1

Soil pressure values increase across the decreasing thickness under constant ks values and soil pressure values increase across the increasing ks values. These comments are supporting with q=kw and illustrated with following graphics.

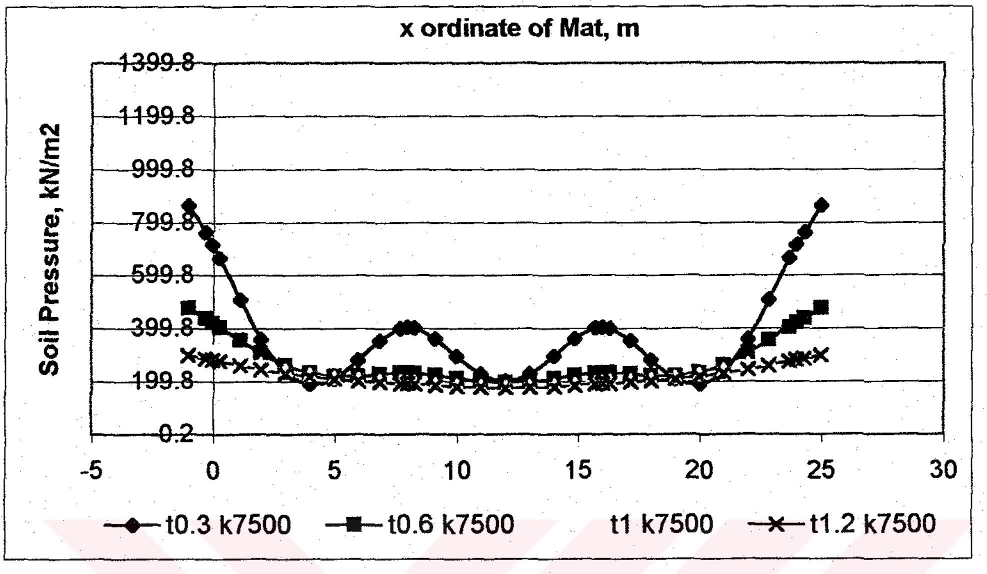

Figure 6.24 Soil pressure distribution according to various thicknesses for constant ks value, 7500kN/m^3 along the CSX1 strip

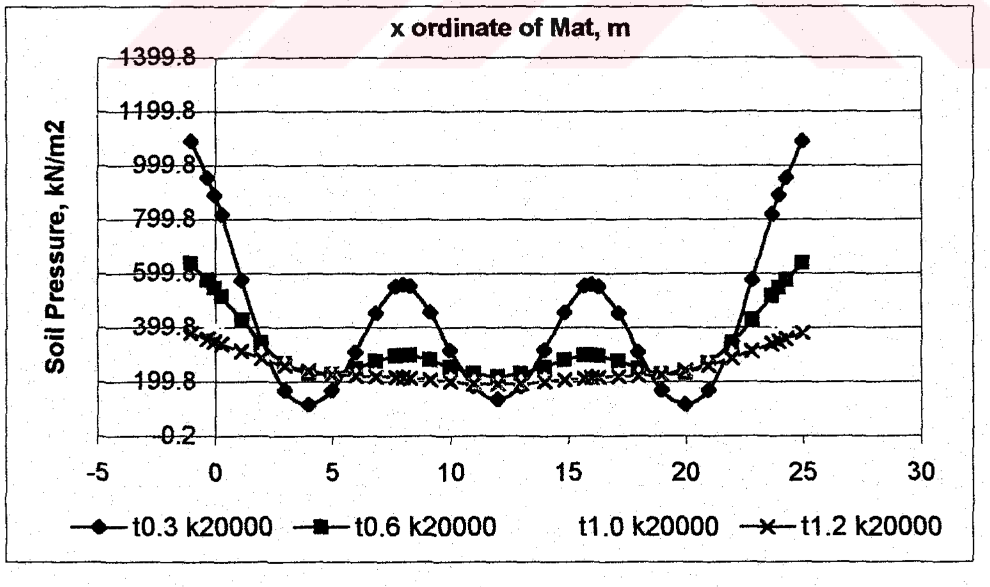

Figure 6.25 Soil pressure distribution according to various thicknesses for constant ks value, 20000kN/m^3 along the CSX1 strip

In figure 6.24 and in figure 6.25, soil pressure distributions show that its variation is not effected enough to chance the thickness of the mat when the values of thickness are greater than 1.0m under low ks values, such as lower than 20000kN/m^3.

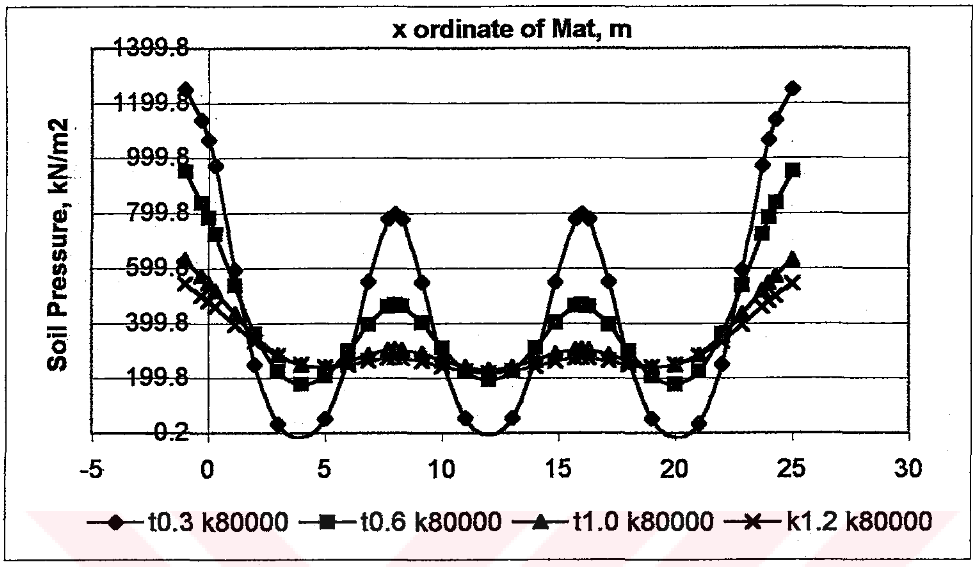

Figure 6.26 Soil pressure distribution according to various thicknesses for constant ks value, 80000kN/m^3 along the CSX1 strip

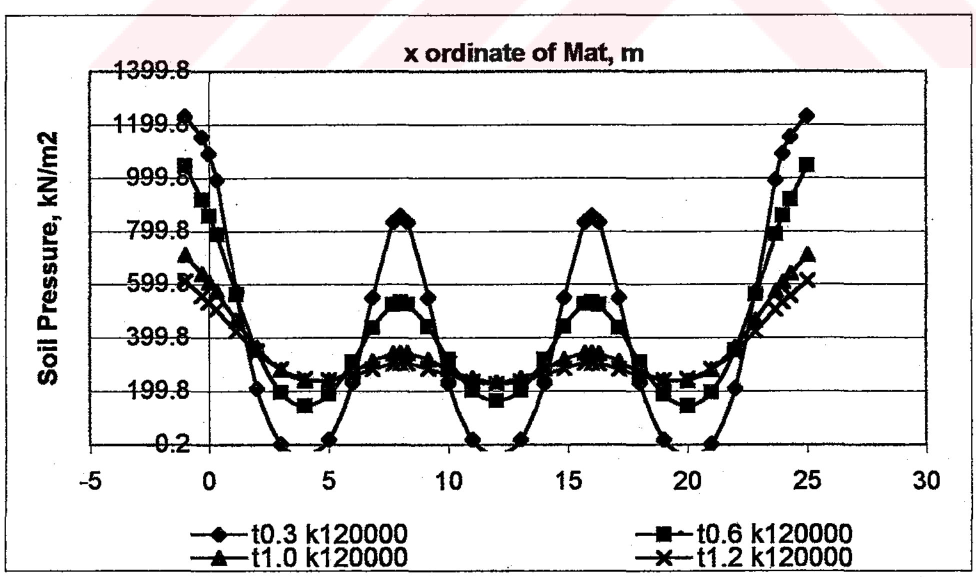

Figure 6.27 Soil pressure distribution according to various thicknesses for constant ks value, 120000kN/m^3 along the CSX1 strip

Moments are examined along the CSX1 strip of mat foundation for various thicknesses and different ks values. Figure 6.28 figure 6.29, figure 6.30 and figure 6.31 show that moments how they effect from the various thicknesses with the constant ks values. Equation (6.4) is referred to comment about moments on mat foundation. Moment values is increase with increasing mat thickness. If ks values increase, moment values of mat decrease. Following figures show these relationships.

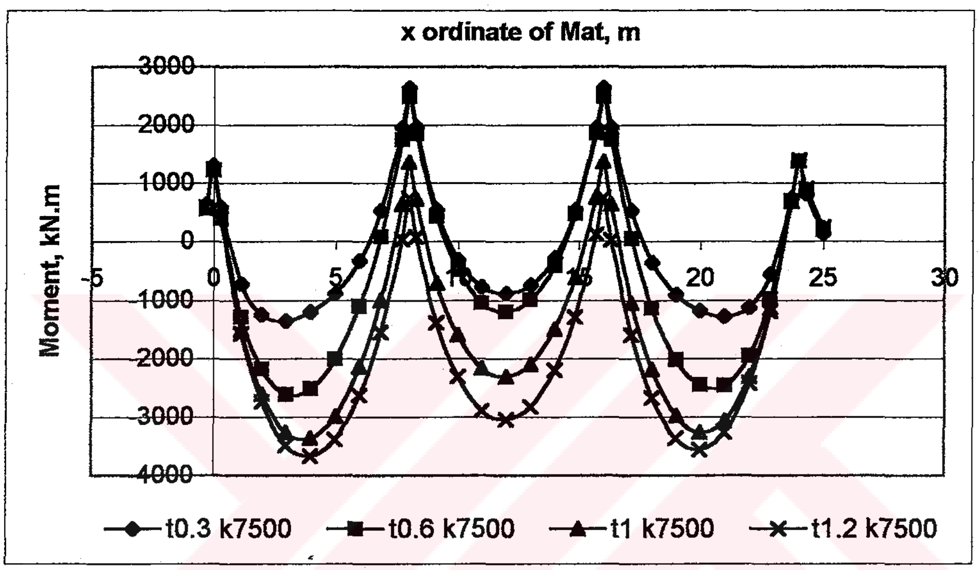

Figure 6.28 Moment distribution according to various thicknesses for constant subgrade reaction, 7500kN/m^3 along the CSX1 strip

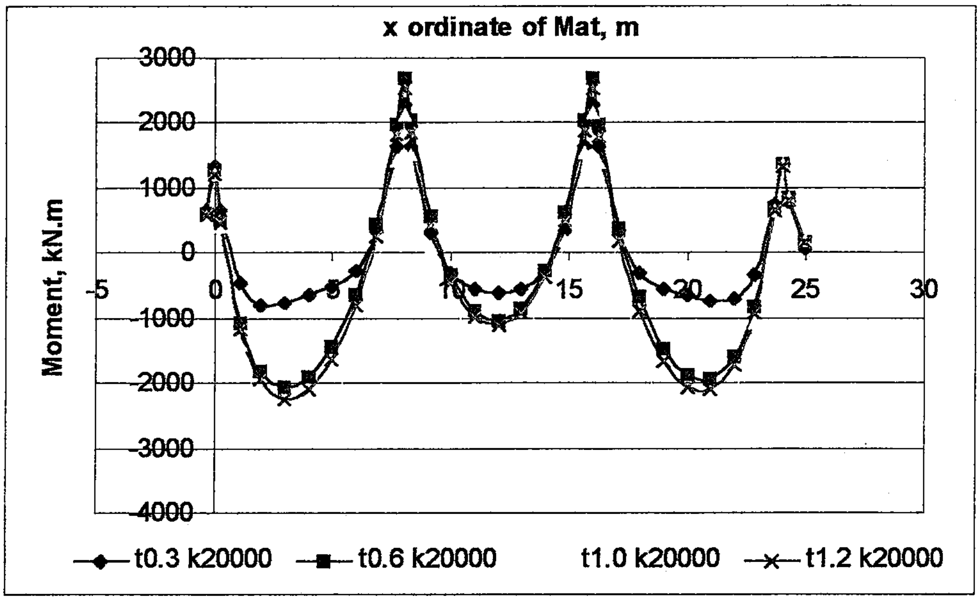

Figure 6.29 Moment distribution according to various thicknesses for constant subgrade reaction, 20000kN/m^3 along the CSX1 strip

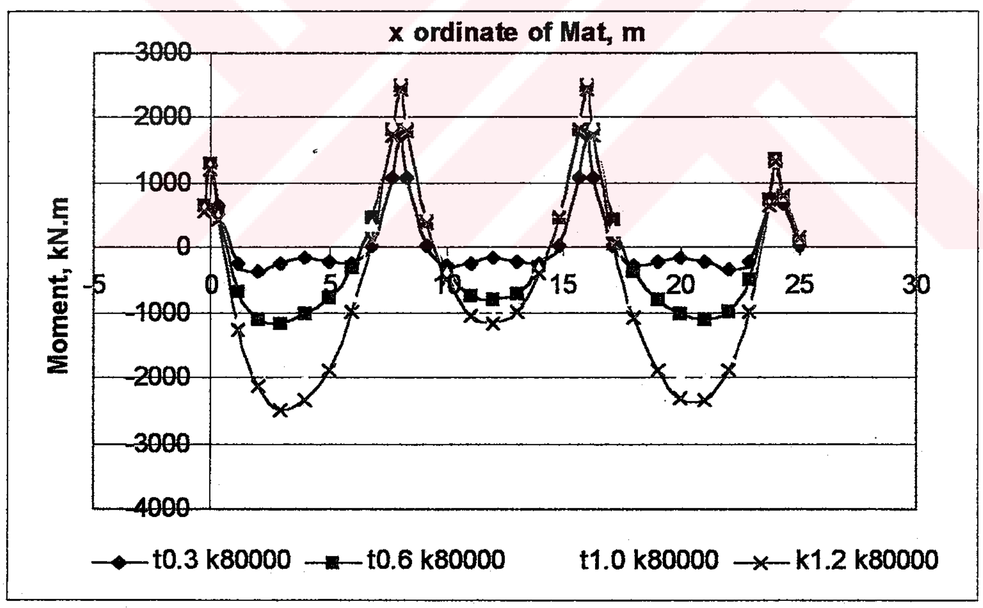

Figure 6.30 Moment distribution according to various thicknesses for constant subgrade reaction, 80000kN/m^3 along the CSX1 strip

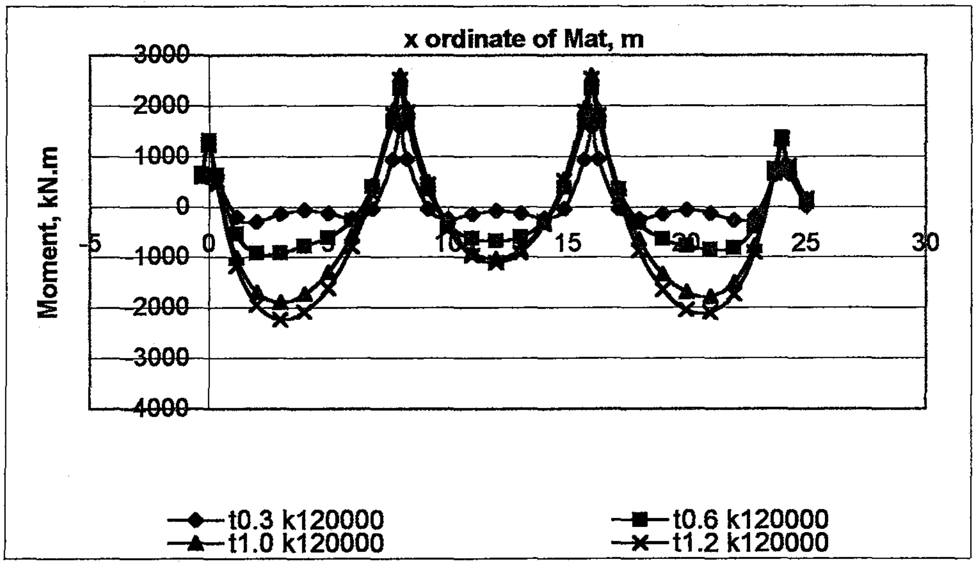

Figure 6.31 Moment distribution according to various thicknesses for constant subgrade reaction, 120000kN/m^3 along the CSX1 strip

Displacements are examined along the CSX1 strip of mat foundation for various ks values and different thickness values. Figure 6.32 and figure 6.33 show that displacements effect from the different ks values under the constant thickness. Equation 6.1 supports the following comment. Figure 6.32, and figure 6.33 show that if the ks value is increase, the displacement values is decrease.

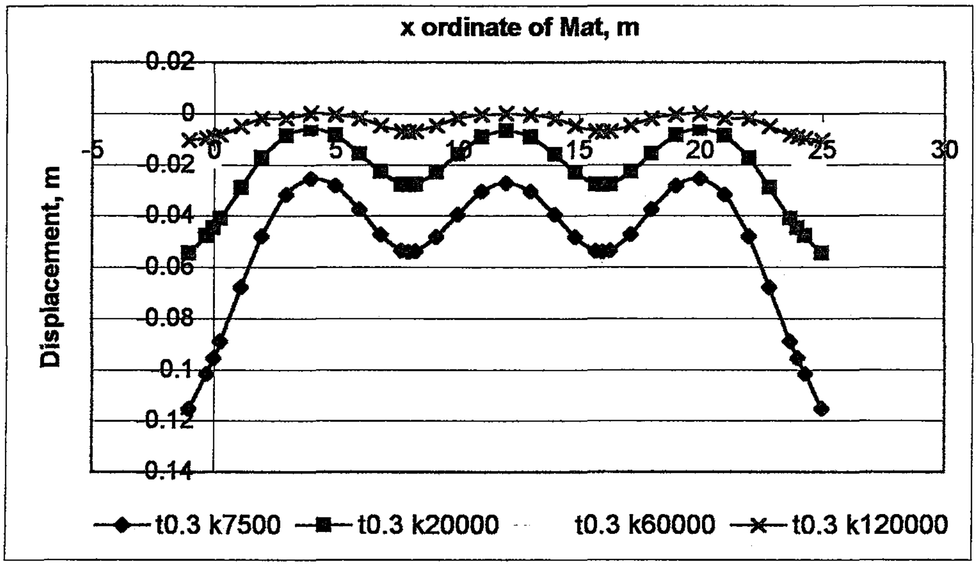

Figure 6.32 Displacement variation profiles for various ks value across the constant thickness, 0.3m along the strip CSX1

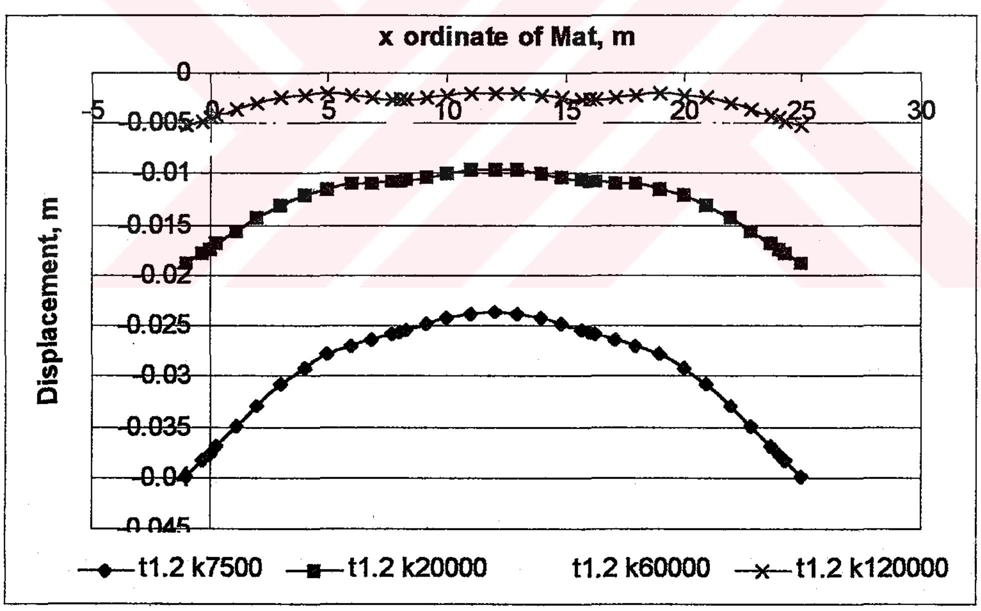

Figure 6.33 Displacement variation profiles for various ks value across the constant thickness, 0.3m along the strip CSX1

Soil pressure values increase with increasing the ks values with constant thickness. These comments are supporting with q=kw and illustrated with following graphics.

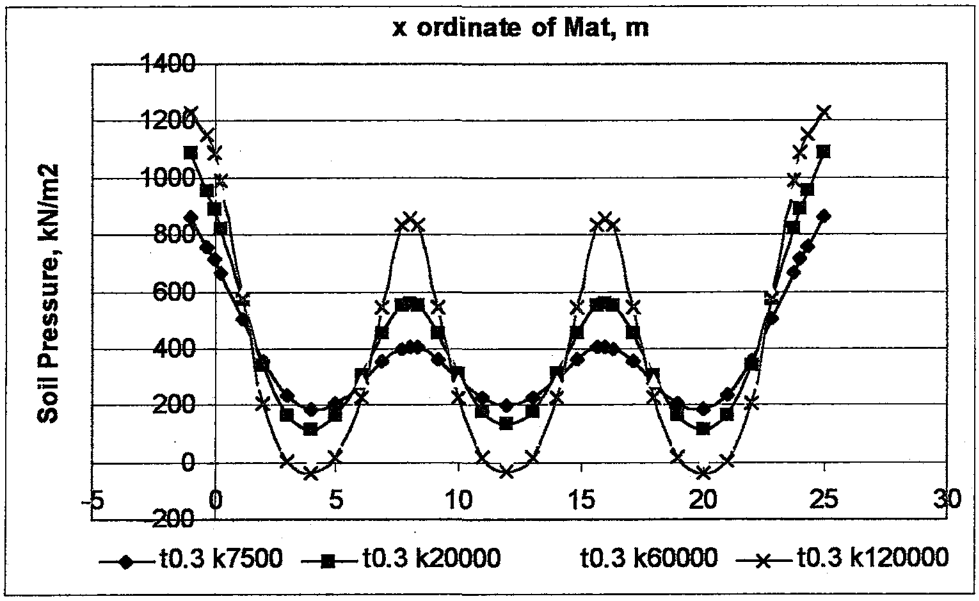

Figure 6.34 Soil pressure distribution according to various ks value for constant thickness, 0.3m along the CSX1 strip

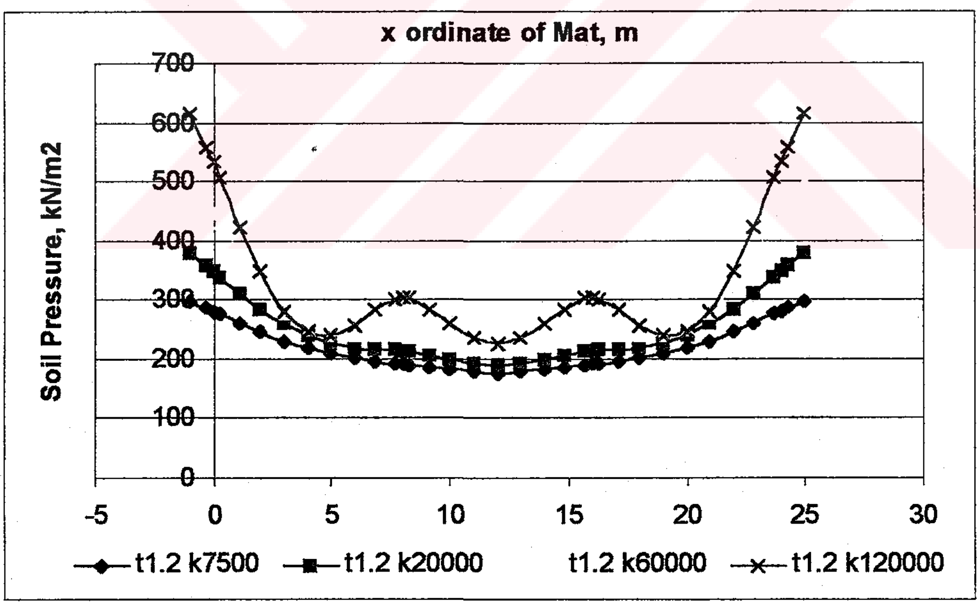

Figure 6.35 Soil pressure distribution according to various ks value for constant thickness, 1.2m along the CSX1 strip

Moments are examined along the CSX1 strip of mat foundation for different thicknesses and various ks values. Figure 6.36 and figure 6.37 show that moments how they effect from the various ks values with the constant thickness. Equation (6.4) is referred to comment about moments on mat foundation. If ks values increase, moment values of mat decrease. Following figures show these relationships.

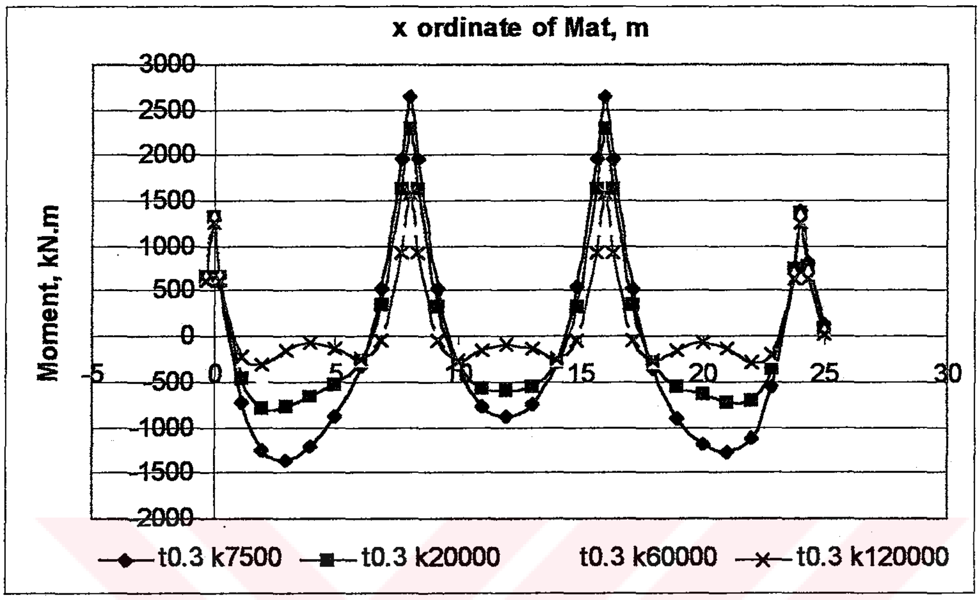

Figure 6.36 Moment distribution according to various subgrade reaction for constant thickness, 0.3m along the CSX1 strip

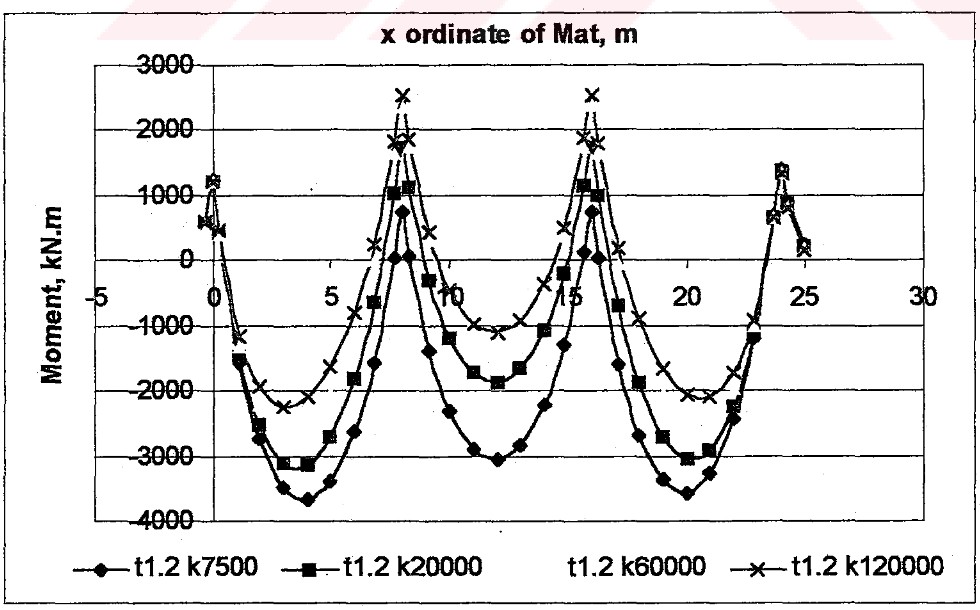

Figure 6.37 Moment distribution according to various subgrade reaction for constant thickness, 1.2m along the CSX1 strip

6.3.3 Case 3

A square mat, which have different span values is defined in this study case. The range of values for modulus of subgrade reaction is between 7500 and 120000 kN/m^3. The value of the applied load is 3000 kN. The thickness of the mat is 1m. Figures of displacements, soil pressures and moments are illustrated according to modulus of subgrade reaction.

The formula of displacement on elastic foundation is mentioned before in equation (6.1) and following figures are supported by this equation.

Figure 6.38 shows the relationship between the displacement and subgrade reaction according to different thicknesses. If ks values increase, displacement values exponentially decrease according to various span lengths values.

The effect of spans is illustrated in figure 6.38. This figure shows that when the soil modulus is equal to $7500kN/m^3$, the difference between the displacements, which are occurred from the spans is higher than when the soil modulus is equal to $120000kN/m^3$. When ks values get a move on increasing, the displacements difference decreases gradually. It is explained as; while k is equal to $7500kN/m^3$, displacements difference between the L=4 and 12m is 0.0283m. While k is equal to $7500kN/m^3$ displacements difference between the L=10 and 12m is 0.0021m. While k is equal to $40000kN/m^3$ displacements difference between the L=6 and 12m is 0.0012m. While k is equal to $60000kN/m^3$ displacements difference between the L=4 and 12m is 0.0021m. While k is equal to $120000kN/m^3$ displacements difference between the L=4 and 12m is 0.0005m. It is noted that all values is accepted as an absolute.

As mentioned above, when ks values pass-over the $40000kN/m^3$, the difference between the displacements except 4m span, get a move on the loose its effectiveness. When ks values pass-over the $60000kN/m^3$, the difference between the displacements get a move on the loose its effectiveness. In an other word, these displacement values are too small to effect the mat design. So we can say that the defined spans are not effective when ks values pass-over the defined ks values. Also, when the spans, which are defined as L=10 and 12m curves are too close to each other in every ks situation. According to this comment, the spans are not effective when its value pass-over the 10m in every ks situation.

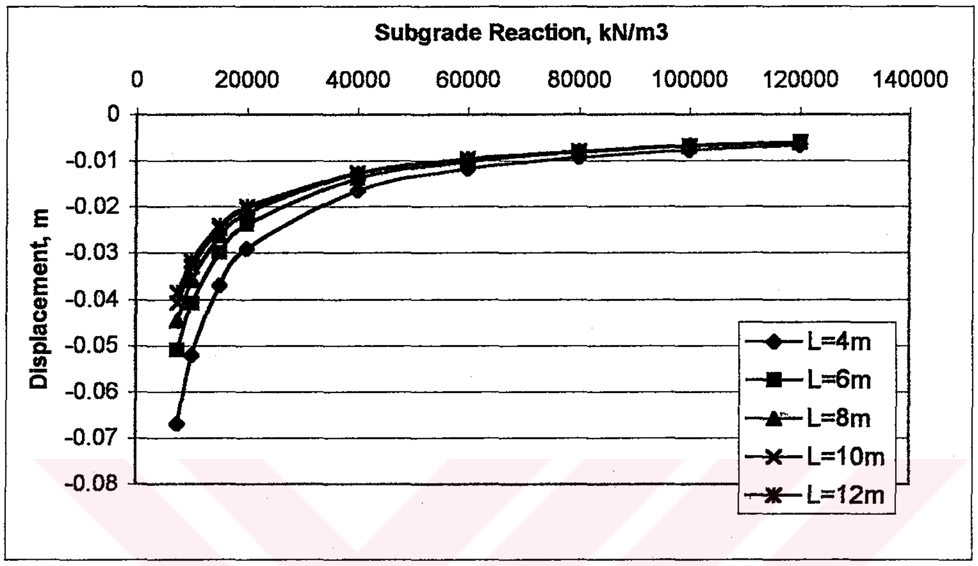

Figure 6.38 Displacement variation across the increasing subgrade reaction according to column space, L

According to q=kw formula, soil pressure values increase exponentially with ks values. In figure 6.39 is illustrated the increasing soil pressure. Soil pressure values increase across the decreasing span values.

Figure 6.39 shows that span values loose their effect on the soil pressure when it is greater than L=6m and ks value is greater than 60000kN/m^3.

Also, when the span values are pass-over the 8m, the curves approximately lay into each other on every ks values. Therefore, it can be said that the effect of span values is ignored, when the span values are greater than 8m.

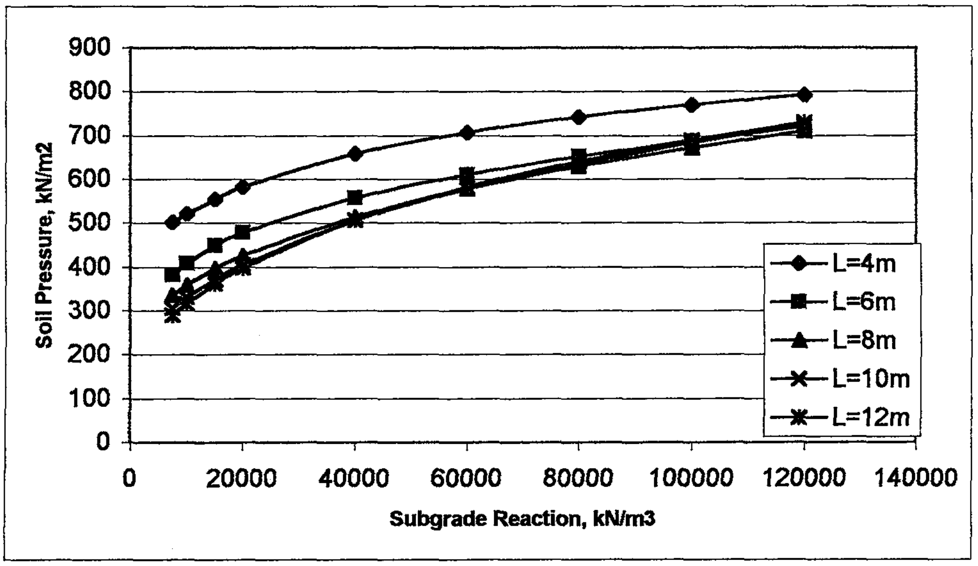

Figure 6.39 Soil pressure-Subgrade reaction profiles for various spans

Moments on elastic foundations can be calculated by Hetenyi's formula as defined by equation (6.4). Equation (6.4) is referred to comment about moments on mat foundation. If ks values increase, moment values exponentially decrease. Figure 6.40 shows this relationship.

Figure 6.40 shows that ks is less effect on the moment when it is greater than 80000kN/m^3 and span value is smaller than 4m. Also, the curves approximately lay into each other, when the span values are greater than 8m and ks values pass-over the 60000kN/m^3. Furthermore, all curves except L=4m intersect, when the ks value is 120000kN/m^3.

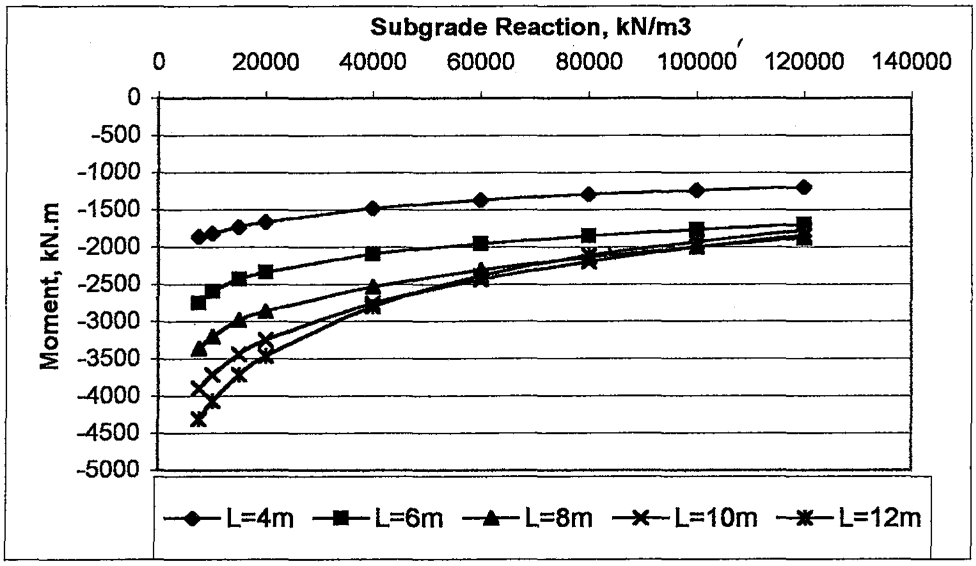

Figure 6.40 Moment variation across the increasing subgrade reaction according to span values, L

6.3.4 Summarized case studies by numerical values

In literature, as mentioned in table 6.2 that the ranges of ks values are different from each other according to authors. When the ks value is selected, these differences occur ambiguity on foundation engineering.

Table 6.2 Ranges of ks values on sandy soils

Authors	Celep-Kumbasar	Bowles	Das
Soil type	**ks, kN/m^3**	**ks, kN/m^3**	**ks, kN/m^3**
Loose sand	10000~20000	**4800**~16000	8000~**25000**
Medium dense sand	20000~50000	**9600**~80000	25000~**125000**
Dense sand	**50000**~100000	64000~128000	125000~**375000**

Minimum and maximum ks values at each soil type are taken from the table 6.2 to calculate the displacements, soil pressures and moments. The size of mat is 26x26 m, thickness of foundation is 1 m and applied loads are 1000 kN and 4000 kN.

Table 6.3 Loose sand – 1000kN

Bowles, ks=4800 kN/m³								
Max. Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.016	1	1	79.49	4	-754.41	62
Das, ks=25000kN/m³								
Max. Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.0044	1	1	111.02	3	-583.69	36.89

Table 6.4 Medium dense sand – 1000kN

Bowles, ks = 9600 kN/m³								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00946	1	1	90.86	4	-675.76	48.36
Das, ks = 125000 kN/m³								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00134	1	1	167.42	3	-392.68	25.82

Table 6.5 Dense sand – 1000kN

Celep, ks = 50000 kN/m3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00263	1	1	131.32	3	-510.68	31.78
Das, ks = 375000 kN/m3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00058	1	1	219.43	3	-237.93	18.01

Related figures about X-strip reinforcement of table 6.3, 6.4 and 6.5 are attached to appendix.

Table 6.6 Loose sand – 4000kN

Bowles, ks = 4800 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.05125	1	1	245.98	4	-3017.66	254.54
Das, ks = 25000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.01488	1	1	372.1	3	-2334.76	150.47

Table 6.7 Medium dense sand – 4000kN

Bowles, ks = 9600 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.03036	1	1	291.45	4	-2703.03	197.69
Das, ks =125000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00478	1	1	597.69	3	-1570.73	105.44

Table 6.8 Dense sand – 4000kN

Celep-Kumbasar, ks = 50000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00907	1	1	453.26	3	-2042.71	129.78
Das, ks = 375000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00215	1	1	805.72	3	-951.72	74.65

Related figures about X-strip reinforcement of table 6.6, 6,7 and 6.8 are attached to appendix.

Table 6.9 Effects of ks on design parameters – 1000kN

Thickness of foundation		1 m		
Applied Load		1000 kN		
Size of foundation		26x26 m		
Span lengths		8 m		
Loose Sand [4800 kN/m^3 ~ 25000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
5.21	3.58	1.4	1.29	1.68
Medium Dense Sand [9600 kN/m^3 ~ 125000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
13.02	7.06	1.84	1.72	1.88
Dense Sand [50000 kN/m^3 ~ 375000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
7.5	4.53	1.67	2.15	1.76

Table 6.9 is obtained from table 6.3, 6.4 and 6.5. (+) sign represents increasing and (-) sign represents decreasing. According to these;

For loose sand, when ks value increases 5.21 times;

- Displacement decreases 3.58 times
- Soil pressure increases 1.40 times
- Moment decreases 1.29 times
- Steel decreases 1.68 times

For medium dense sand, when ks value increases 13.02 times;

- Displacement decreases 7.06 times
- Soil pressure increases 1.84 times
- Moment decreases 1.72 times
- Steel decreases 1.88 times

For dense sand, when ks value increases 7.50 times;

- Displacement decreases 4.53 times
- Soil pressure increases 1.67 times
- Moment decreases 2.15 times
- Steel decreases 1.76 times

Table 6.10 Effects of ks on design parameters – 4000kN

Thickness of foundation		1m		
Applied Load		4000 kN		
Size of foundation		26x26 m		
Span length		8 m		
Loose Sand				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
5.21	3.44	1.5	1.29	1.69
Medium Dense Sand				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
13.02	6.35	2.05	1.72	1.87
Dense Sand				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
7.5	4.22	1.78	2.15	1.74

Table 6.10 is obtained from table 6.6, 6.7 and 6.8. (+) sign represents increasing and (-) sign represents decreasing. According to these;

For loose sand, when ks value increases 5.21 times;

- Displacement decreases 3.44 times
- Soil pressure increases 1.15 times
- Moment decreases 1.29 times
- Steel decreases 1.69 times

For medium dense sand, when ks value increases 13.02 times;

- Displacement decreases 6.35 times
- Soil pressure increases 2.05 times
- Moment decreases 1.72 times
- Steel decreases 1.87 times

For dense sand, when ks value increases 7.50 times;

- Displacement decreases 4.22 times
- Soil pressure increases 1.78 times
- Moment decreases 2.15 times
- Steel decreases 1.74 times

Minimum and maximum ks values at each soil type are taken from the table 6.2 to calculate the displacements, soil pressures and moments. The size of mat is 26x26 m, applied load is 3000kN and depths of foundation are 0.3 m and 1.2 m.

Table 6.11 Loose sand – 0.3 m

Bowles, ks = 4800 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.15834	1	1	760.05	3	-1646.23	412.23
Das, ks = 25000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.04534	1	1	1133.5	2	-703.01	215.29

Table 6.12 Medium dense sand – 0.3 m

Bowles, ks = 9600 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.09613	1	1	922.81	3	-1200.8	323.8
Das, ks = 125000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00982	1	1	1226.89	2	-295.53	114.9

Table 6.13 Dense sand – 0.3 m

Celep, ks = 50000 kN/m3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.0246	1	1	1229.93	2	-481.17	159.76
Das, ks = 375000 kN/m3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00259	1	1	1314.83	1.15	-171.03	78.49

Table 6.14 Loose sand – 1.2 m

Bowles, ks = 4800 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.05579	1	1	267.78	4	-3922.29	225
Das, ks = 25000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.01595	1	1	398.65	3	-3020.38	125.53

Table 6.15 Medium dense sand – 1.2 m

Bowles, ks = 9600 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.033	1	1	316.82	4	3531.65	174.57
Das, ks =125000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00497	1	1	621.34	3	-2213.02	86.13

Table 6.16 Dense sand – 1.2 m

Celep-Kumbasar, ks = 50000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00954	1	1	477.1	3	-2716.12	104.89
Das, ks = 375000 kN/m^3								
Max Displacement			Max. Soil pressure			Max. Moment		Steel
Grid I	Grid J	Disp.	Grid I	Grid J	Pressure	X ordinate, m	Moment	Weight
1	1	-0.00228	1	1	856.35	3	-1484.33	64.22

Related figures about X-strip reinforcement of table 6.11, 6.12, 6.13, 6.14, 6.15 and 6.16 are attached to appendix.

Table 6.17 Effects of ks on design parameters – 0.3m

Thickness of foundation		0.3 m		
Applied Load		3000 kN		
Size of foundation		26x26 m		
Span lengths		8 m		
Loose Sand [4800 kN/m^3 ~ 25000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
5.21	3.49	1.49	2.34	1.9
Medium Dense Sand [9600 kN/m^3 ~ 125000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
13.02	9.79	1.33	4.06	2.82
Dense Sand [50000 kN/m^3 ~ 375000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
7.5	9.5	1.07	2.81	2.03

Table 6.17 is obtained from table 6.11, 6.12 and 6.13. (+) sign represents increasing and (-) sign represents decreasing. According to these;

For loose sand, when ks value increases 5.21 times;

- Displacement decreases 3.49 times
- Soil pressure increases 1.49 times
- Moment decreases 2.34 times
- Steel decreases 1.9 times

For medium dense sand, when ks value increases 13.02 times;

- Displacement decreases 9.79 times
- Soil pressure increases 1.33 times
- Moment decreases 4.06 times
- Steel decreases 2.82 times

For dense sand, when ks value increases 7.50 times;

- Displacement decreases 9.5 times
- Soil pressure increases 1.07 times
- Moment decreases 2.81 times
- Steel decreases 2.03 times

Table 6.18 Effects of ks on design parameters – 1.2 m

Thickness of foundation		1.2 m		
Applied Load		3000 kN		
Size of foundation		26x26 m		
Span length		8 m		
Loose Sand [4800 kN/m^3 ~ 25000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
5.21	3.5	1.49	1.3	1.79
Medium Dense Sand [9600 kN/m^3 ~ 125000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
13.02	6.64	1.96	1.59	2.03
Dense Sand [50000 kN/m^3 ~ 375000 kN/m^3]				
ks (+)	Disp (-)	Press. (+)	Mom. (-)	Steel (-)
7.5	4.18	1.79	1.83	1.63

Table 6.18 is obtained from table 6.14, 6.15 and 6.16. (+) sign represents increasing and (-) sign represents decreasing. According to these;

For loose sand, when ks value increases 5.21 times;

- Displacement decreases 3.5 times
- Soil pressure increases 1.49 times
- Moment decreases 1.3 times
- Steel decreases 1.79 times

For medium dense sand, when ks value increases 13.02 times;

- Displacement decreases 6.64 times
- Soil pressure increases 1.96 times
- Moment decreases 1.59 times
- Steel decreases 2.03 times

For dense sand, when ks value increases 7.50 times;

- Displacement decreases 4.18 times
- Soil pressure increases 1.79 times
- Moment decreases 1.83 times
- Steel decreases 1.63 times

CHAPTER 7

CONCLUSION

7.1 Conclusion

The primary goal of this study is to illustrate the effects of subgrade modulus on mat foundation design. Therefore, the determination of subgrade modulus is taken priority on mat foundation design. To determinate the ks, in situ tests and theoretical approaches have been used. Three different case is studied in this thesis. These relationships are ks and load, ks and thickness of mat, and ks and span length of columns.

In the first case, the effect of loading factor on mat foundation design is investigated. Several load intensities are applied with increasing ks values on mat foundation. Displacements, soil pressures and moments are investigated. According to these investigations loading factor loose its effectiveness on displacement values gradually with increasing ks values. However, soil pressure increase with increasing loading factors and ks values. Also this case shows that, moments increase with increasing load patterns but decrease with increasing ks values.

The effects of mat thickness for a given ks values are investigated in the second study case. Displacement values of mat foundation are increase when the mat thickness is decrease. The similar comments can be stated for soil pressure, its value increase across the decreasing mat thickness values. However, moments increase with increasing mat thickness. Subgrade modulus is not effected on moments so much when it is greater than marked ks values and thickness of mat. Furthermore, thickness of mat is not effective so much on displacement and soil pressure when it passes over the marked ks values so these small magnitudes does not have greater effect on mat foundation design.

In the third case, investigation on relationship between the span length of columns and ks show that span length of columns loose its effectiveness gradually on displacements when ks values increase. Displacement values of mat foundation are increase across the decreasing span lengths of columns. Soil pressure values increase across the decreasing span lengths of columns. However, moment values increase if the span length is getting large.

Subgrade modulus takes different values for each soil type but each soil type don't have same ks values. In published resources, the range of ks values of same soil type are different from each other. These differences occur ambiguity for selecting ks values. Minimum and maximum ks value of same soil type are used to investigate the error ratio. Firstly, 1000kN and 4000kN are defined as an applied load and used to obtain the displacement, soil pressure, moment and steel. Results show that when ks value increases 5.21 times, displacement decreases 3.58 times, soil pressure increases 1.4 times, moment decreases 1.29 times and steel decreases 1.68 times. These results are acceptable for 1000kN and loose sand. Furthermore, the ratio of mentioned parameters do not have greater effect from the load intensities. Secondly, 0.3 m and 1.2 m are defined as a mat thickness and used to obtain the displacement, soil pressure, moment and steel. Results show that when ks value increases 5.21 times, displacement decreases 3.49 times, soil pressure increases 1.49 times, moment decreases 2.34 times and steel decreases 1.9 times. These results are acceptable for 0.3 m and loose sand.

As a result, experience and mature judgment are essential for determining subgrade modulus because subgrade modulus not just a soil parameter. It also depends on structural member. Therefore, any effort to measure or calculate its value beforehand based on some soil tests is inherently flawed. Subgrade modulus is actually an observed, calculated results in any geotechnical problem involving soil - structure interaction.

7.2 Recommendations for Future Work

The determination of the value of subgrade reaction and its variation below a foundation can be studied. The distribution of the modulus of subgrade reaction, can be determined by a parametric study related to foundation shapes ground depths and loading patterns.

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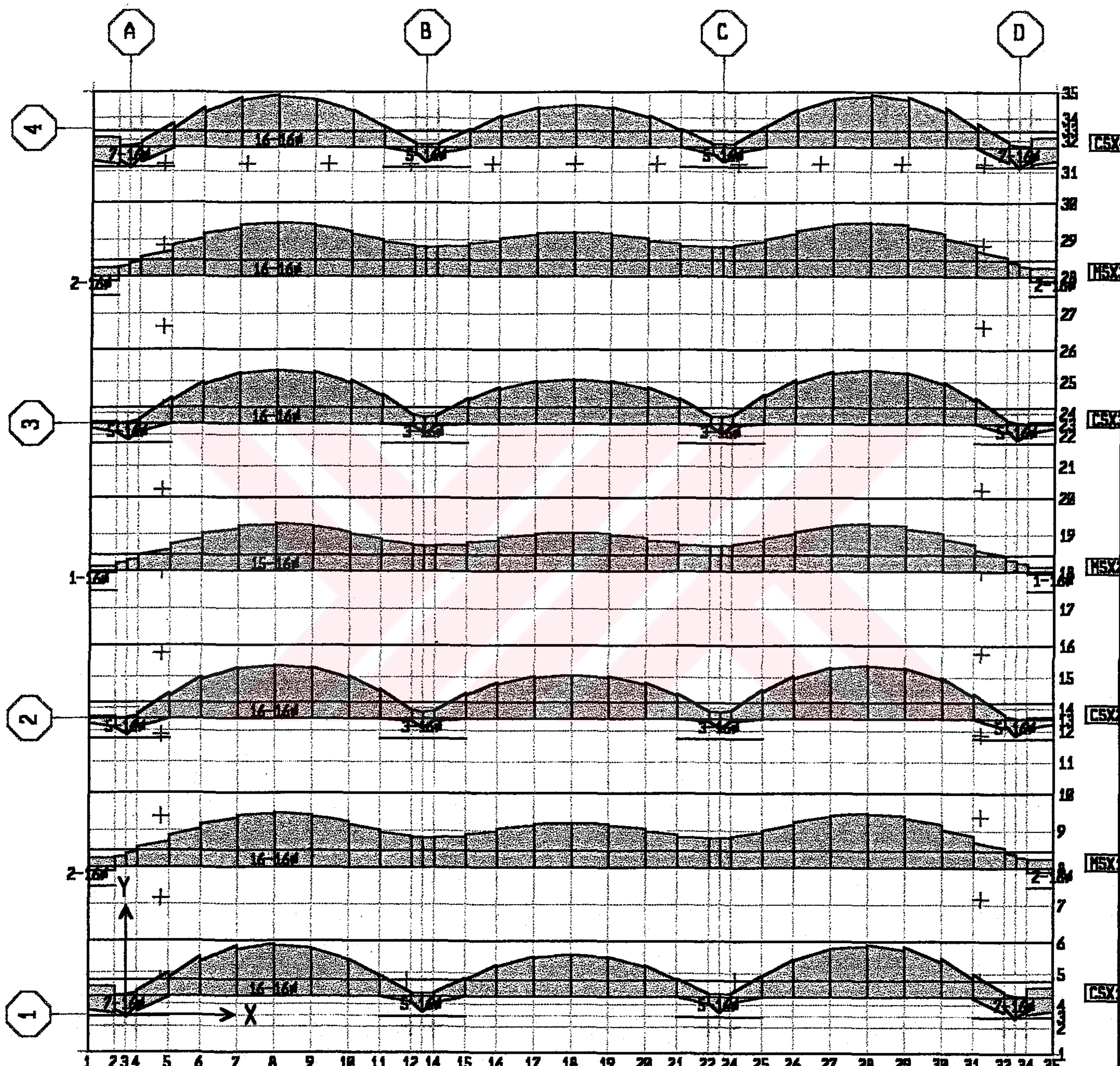
APPENDIX



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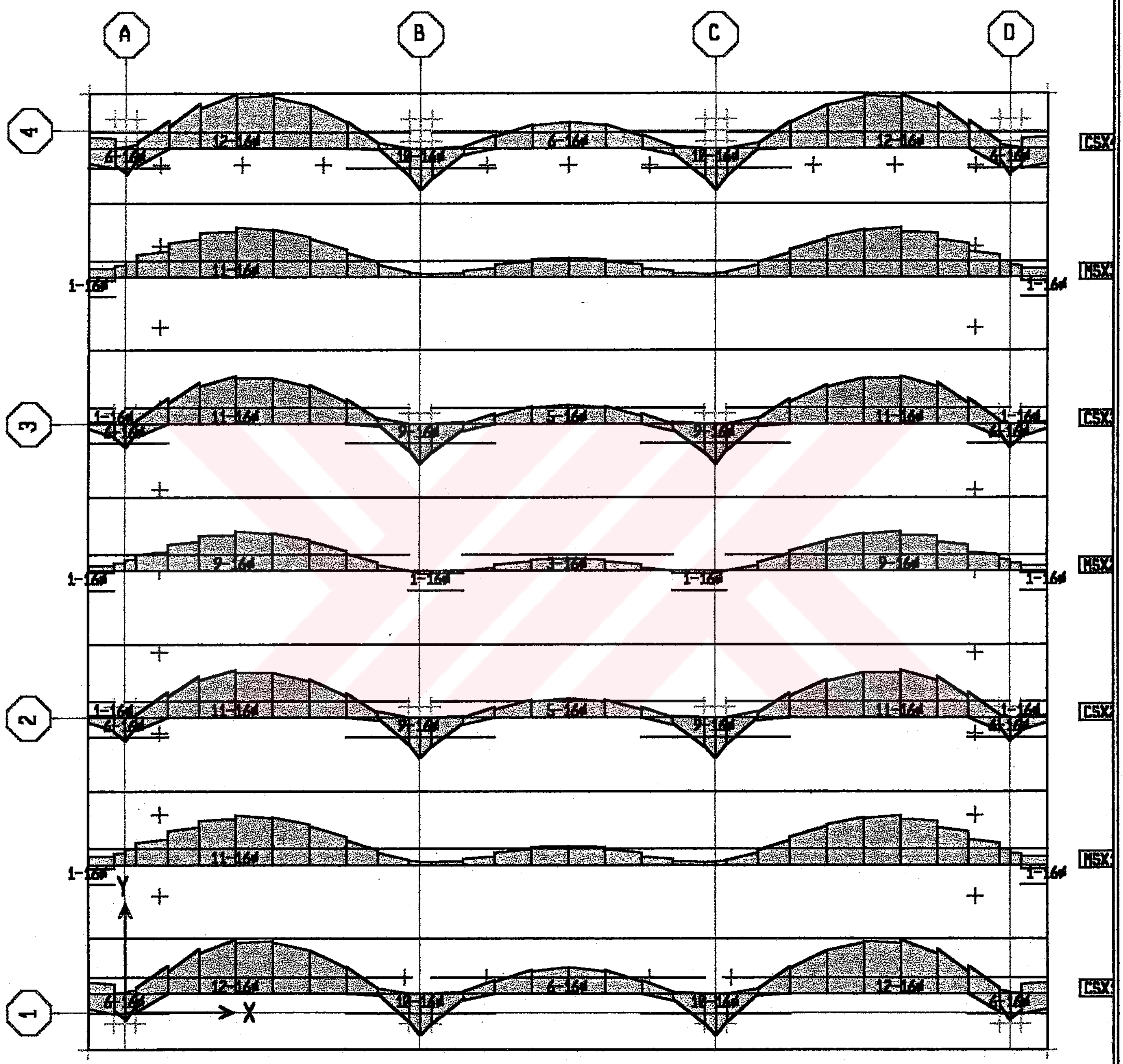
X-Strip Reinforcement (Sq-cm) - KN-m Units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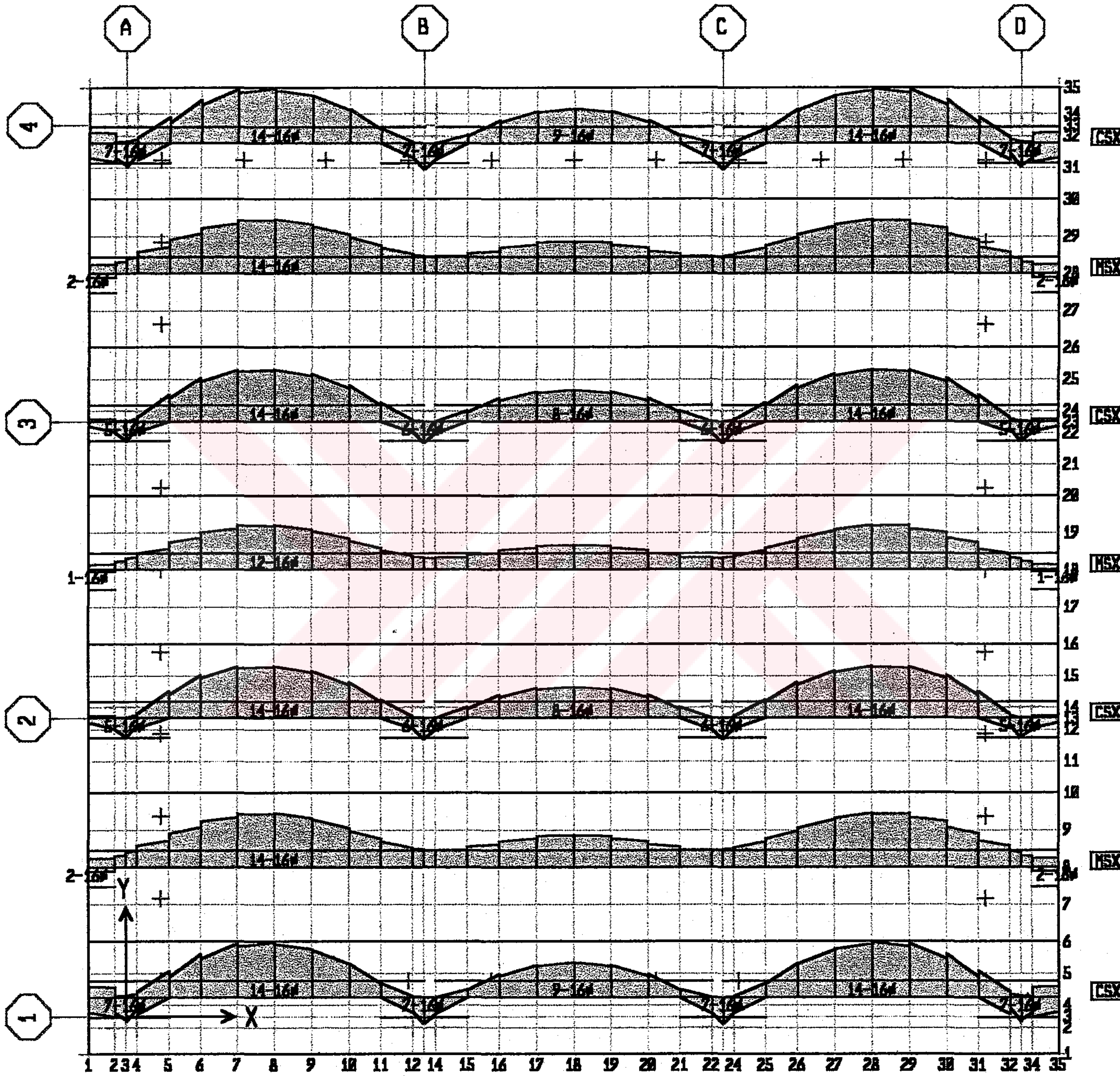
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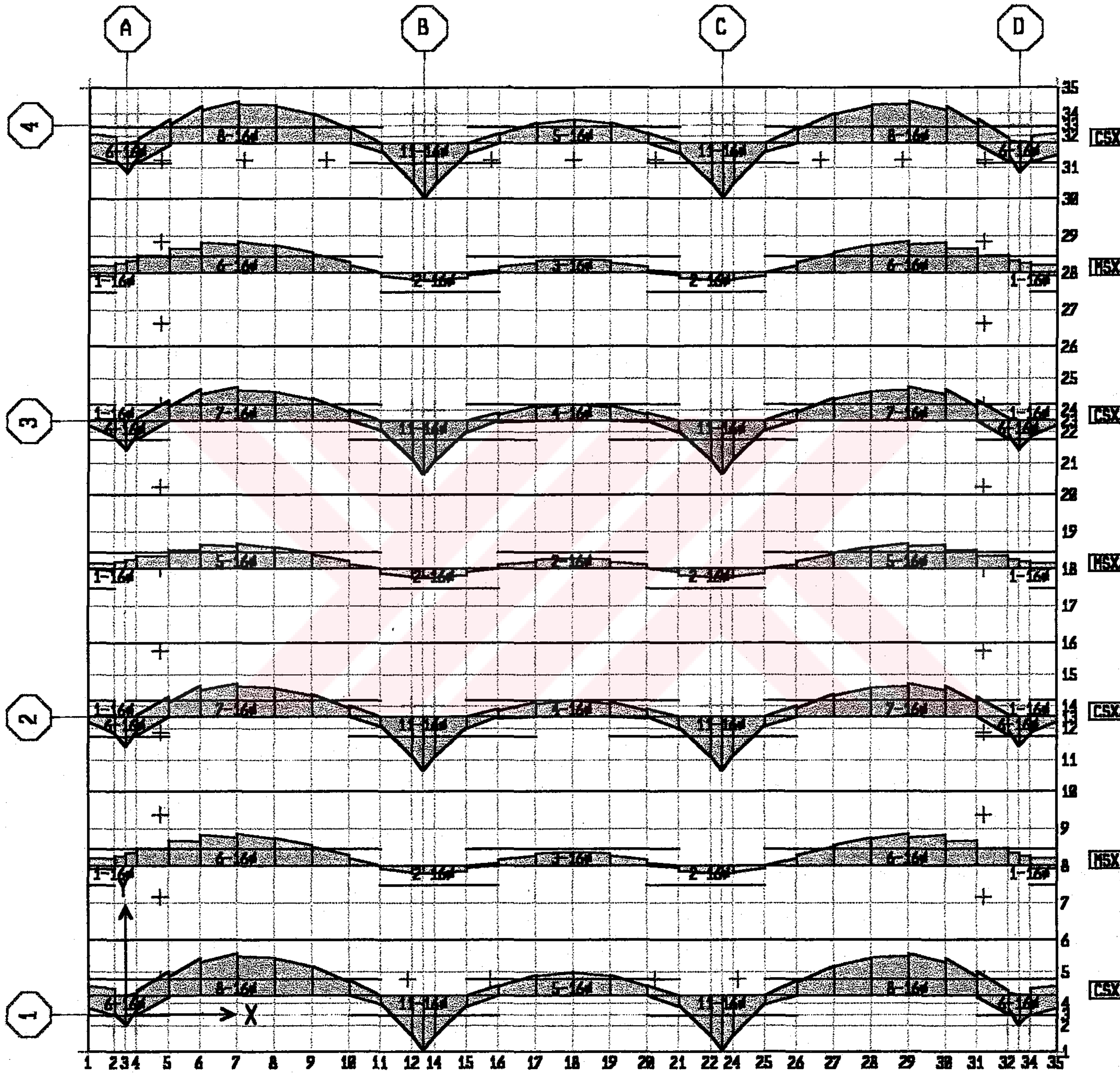
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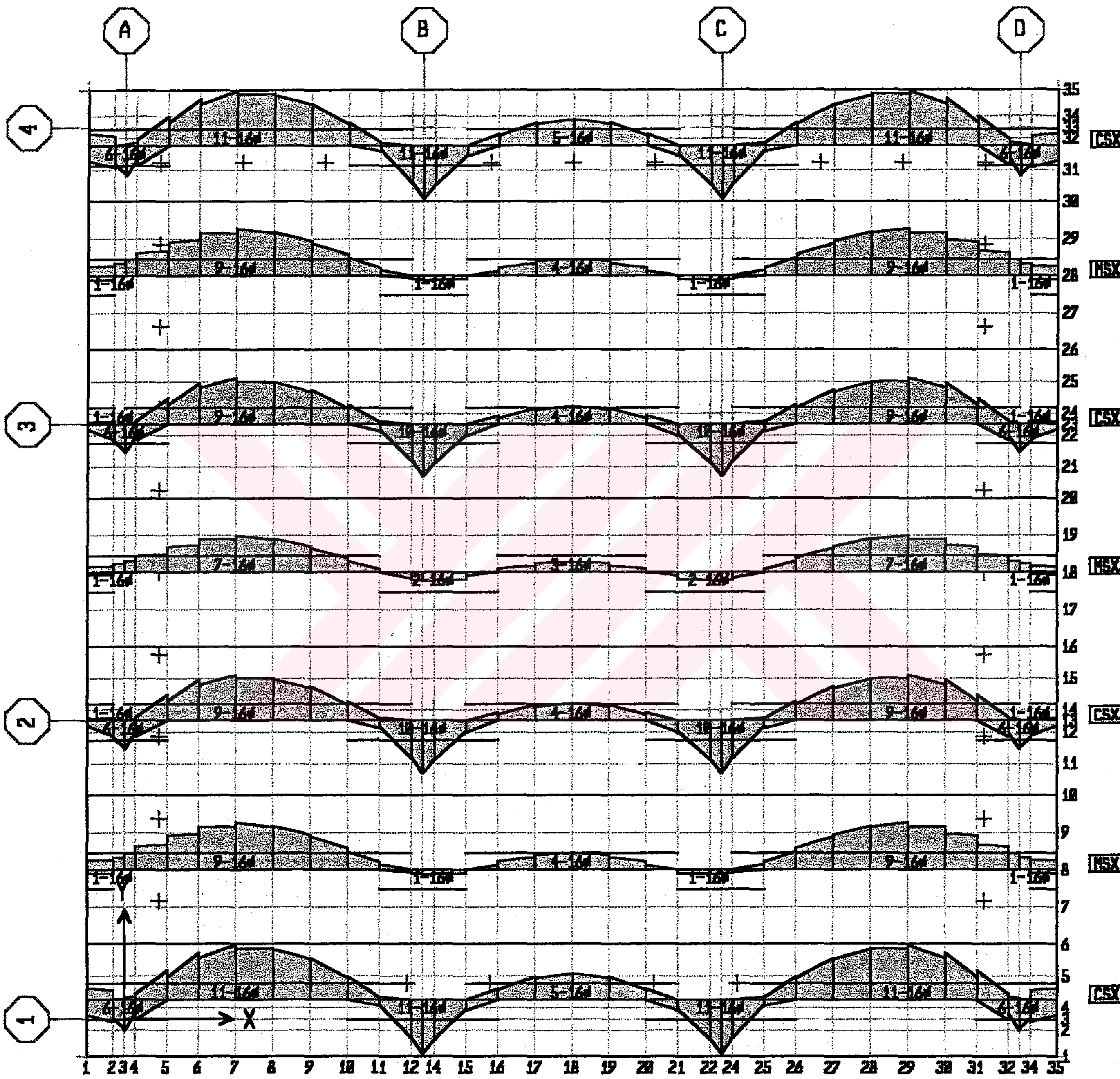
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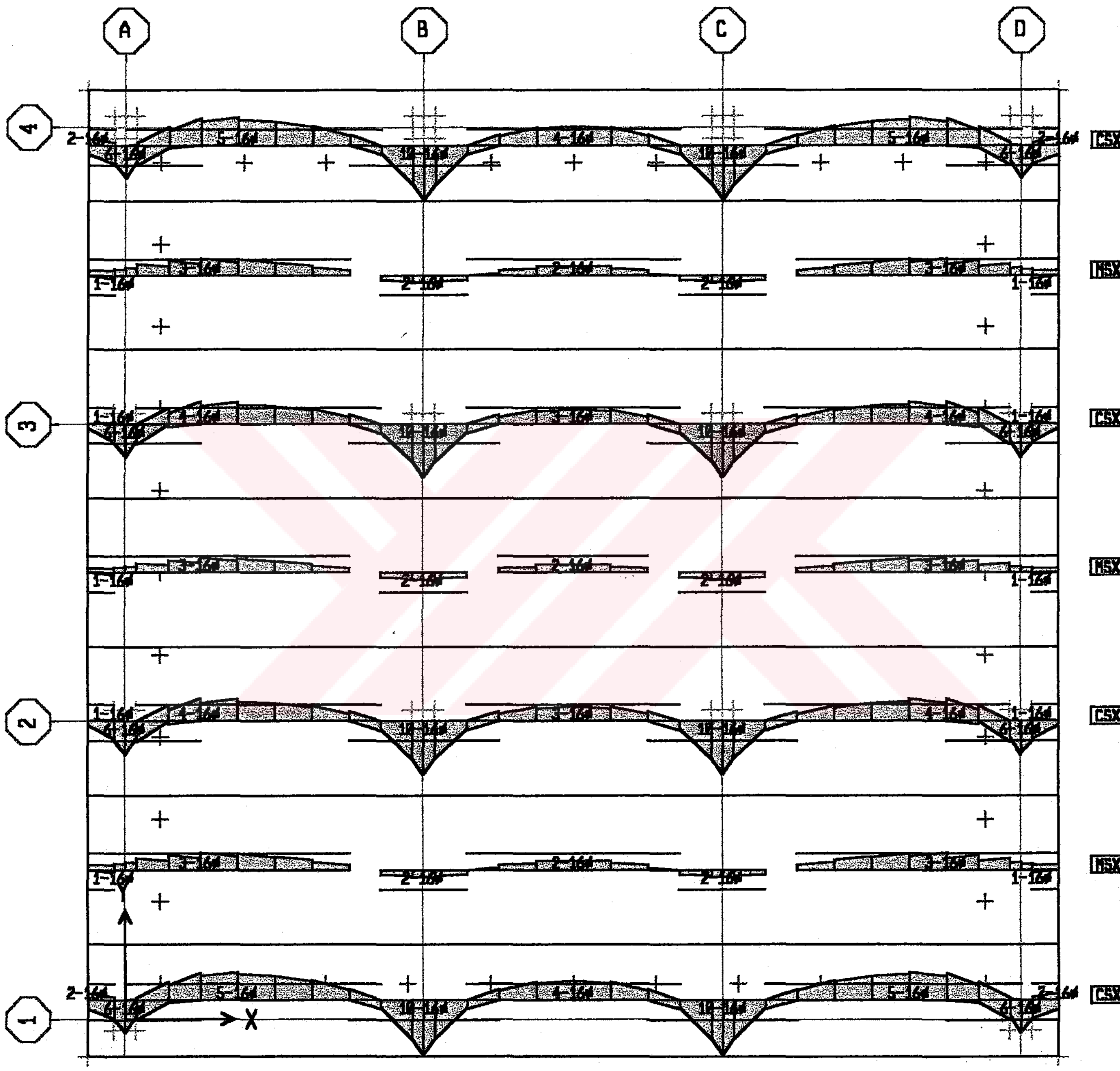

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C-Strip Reinforcement (Sq-cm) - KN-m Units

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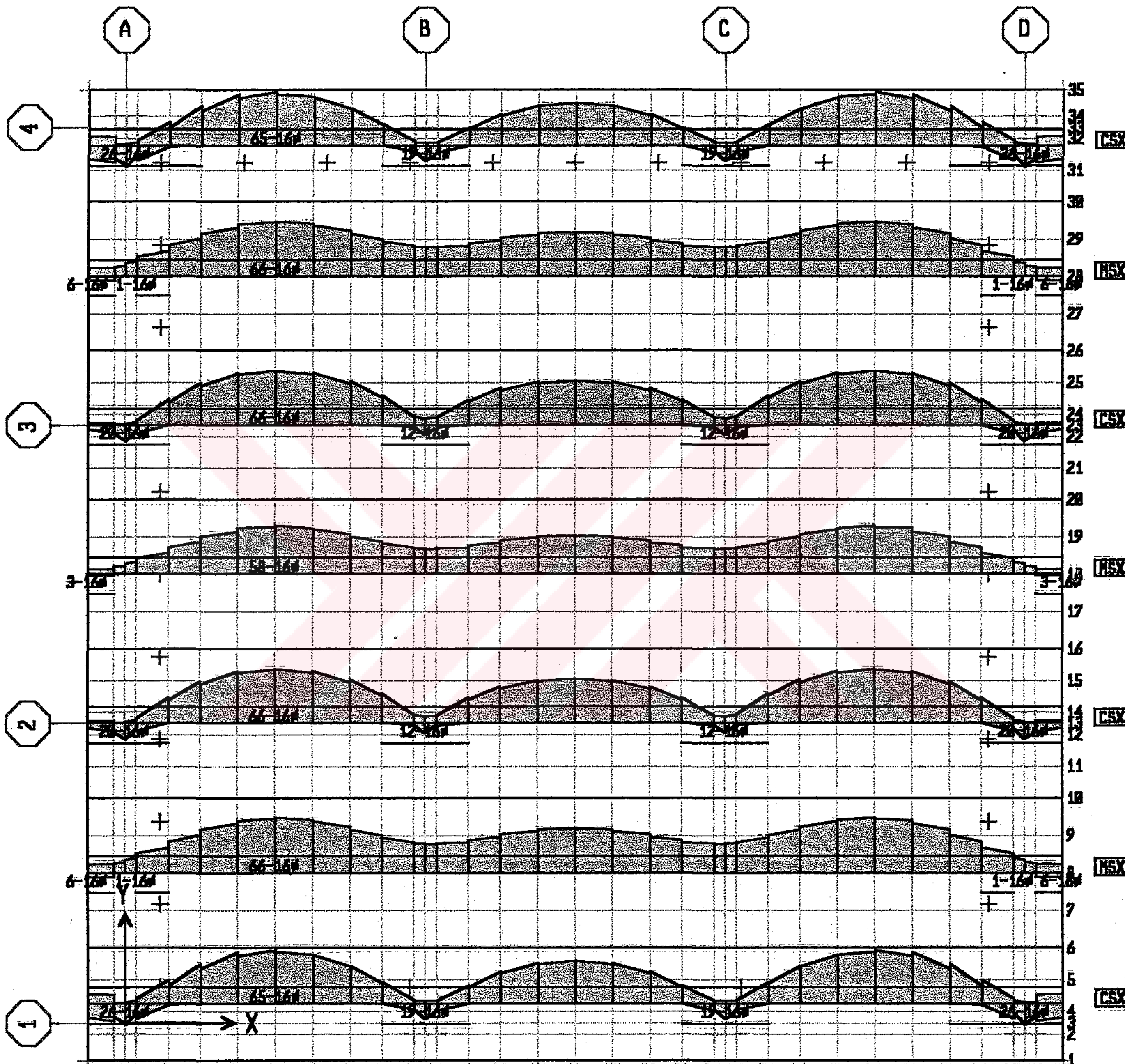

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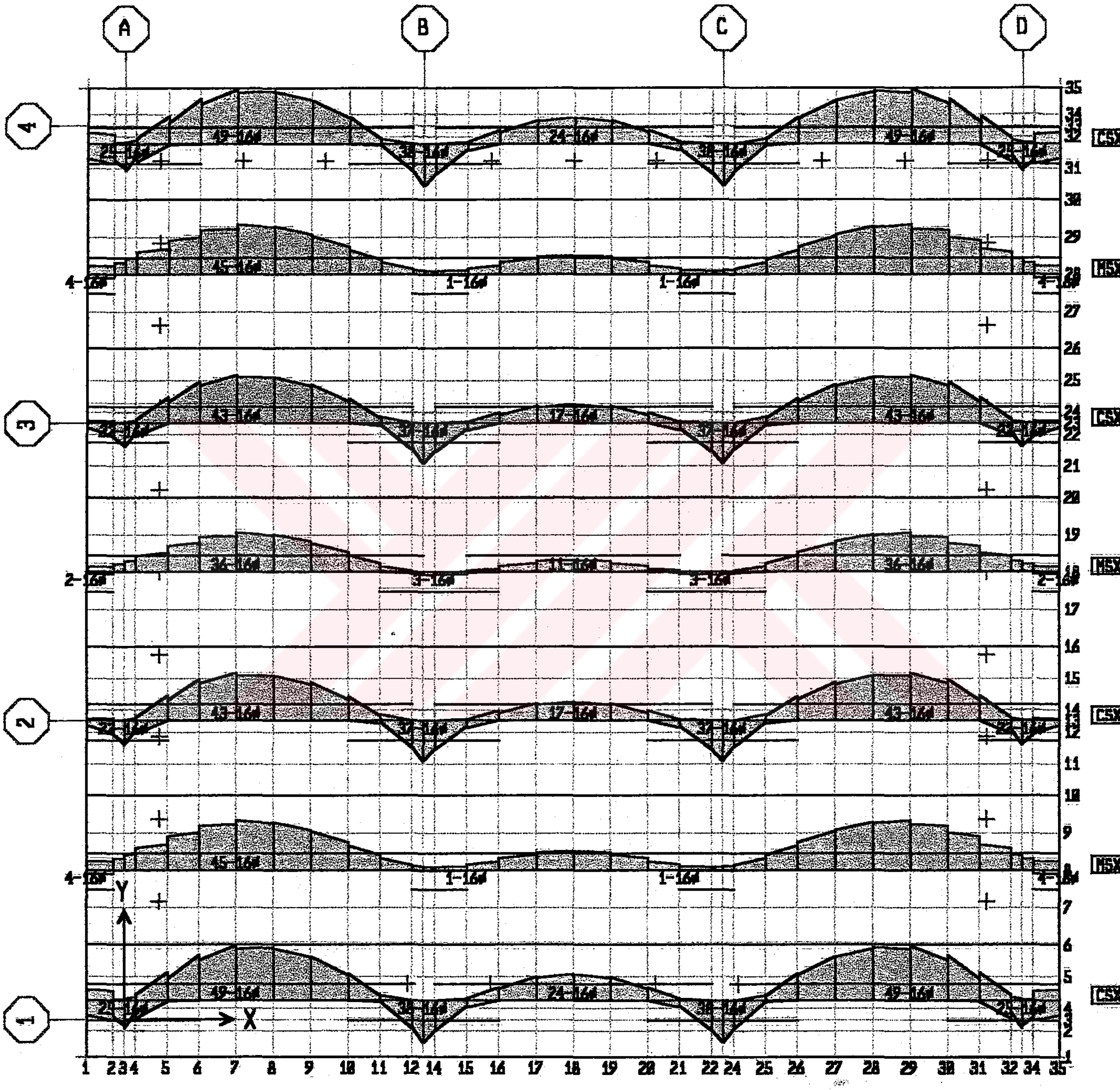

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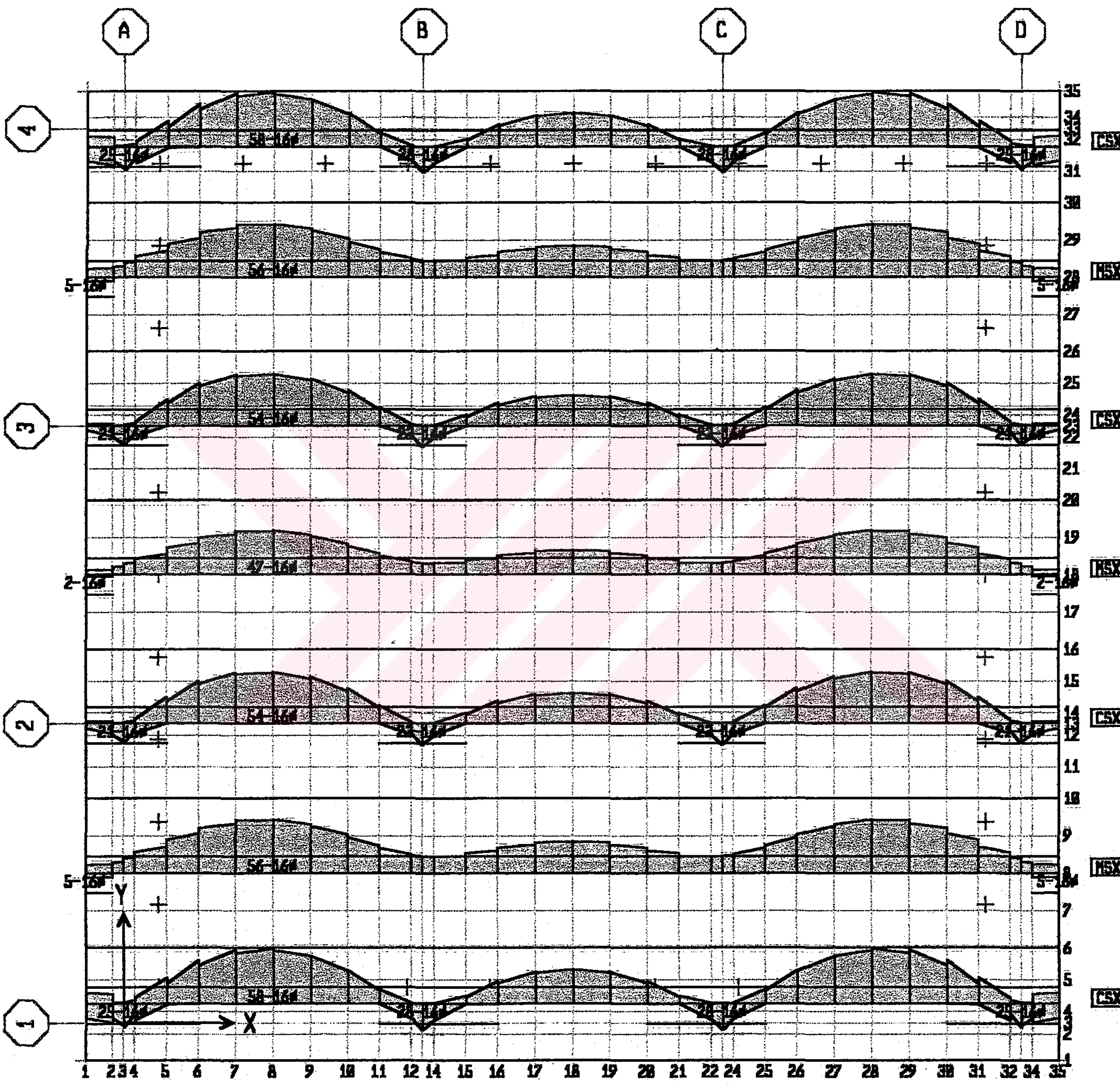
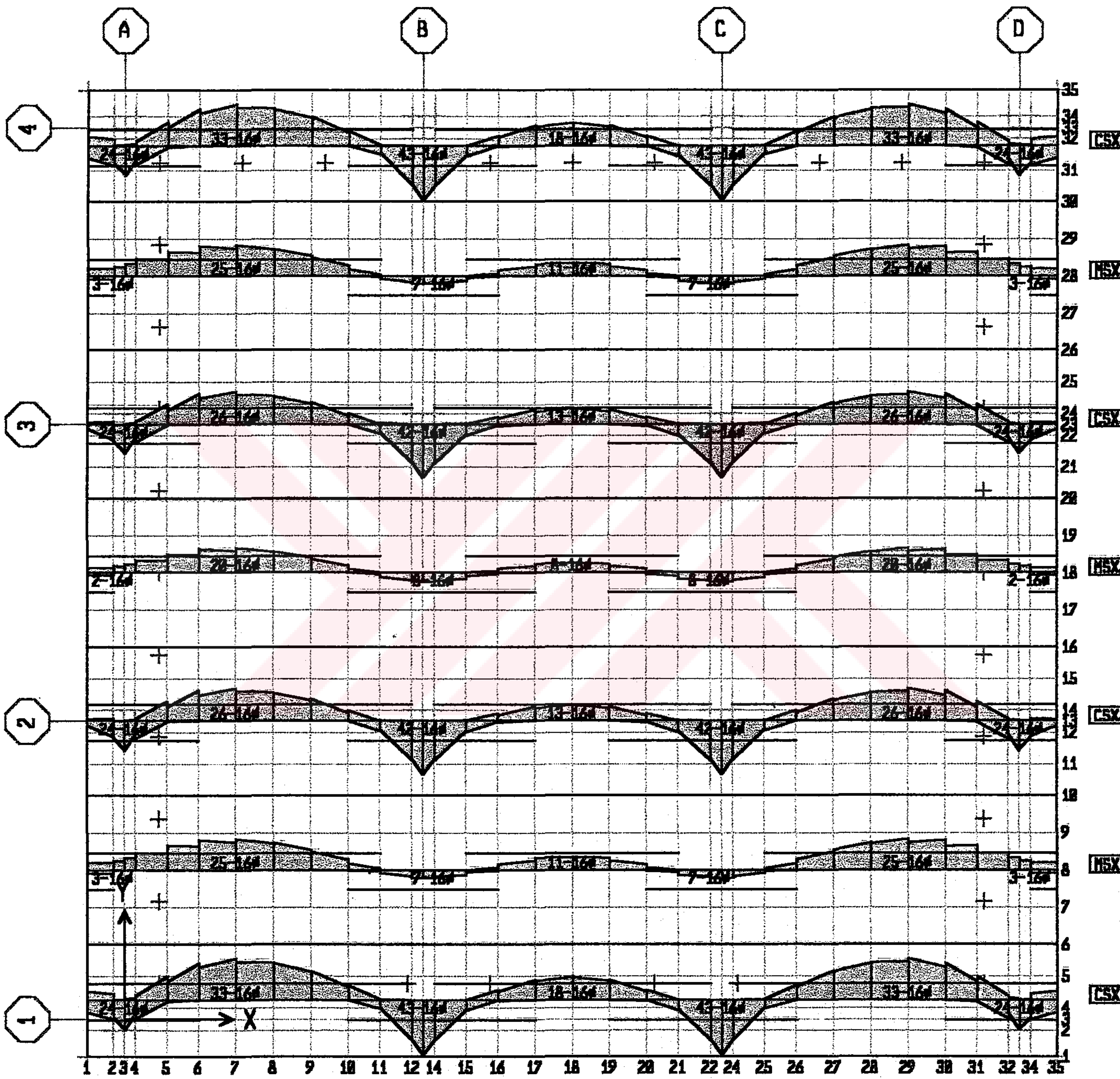
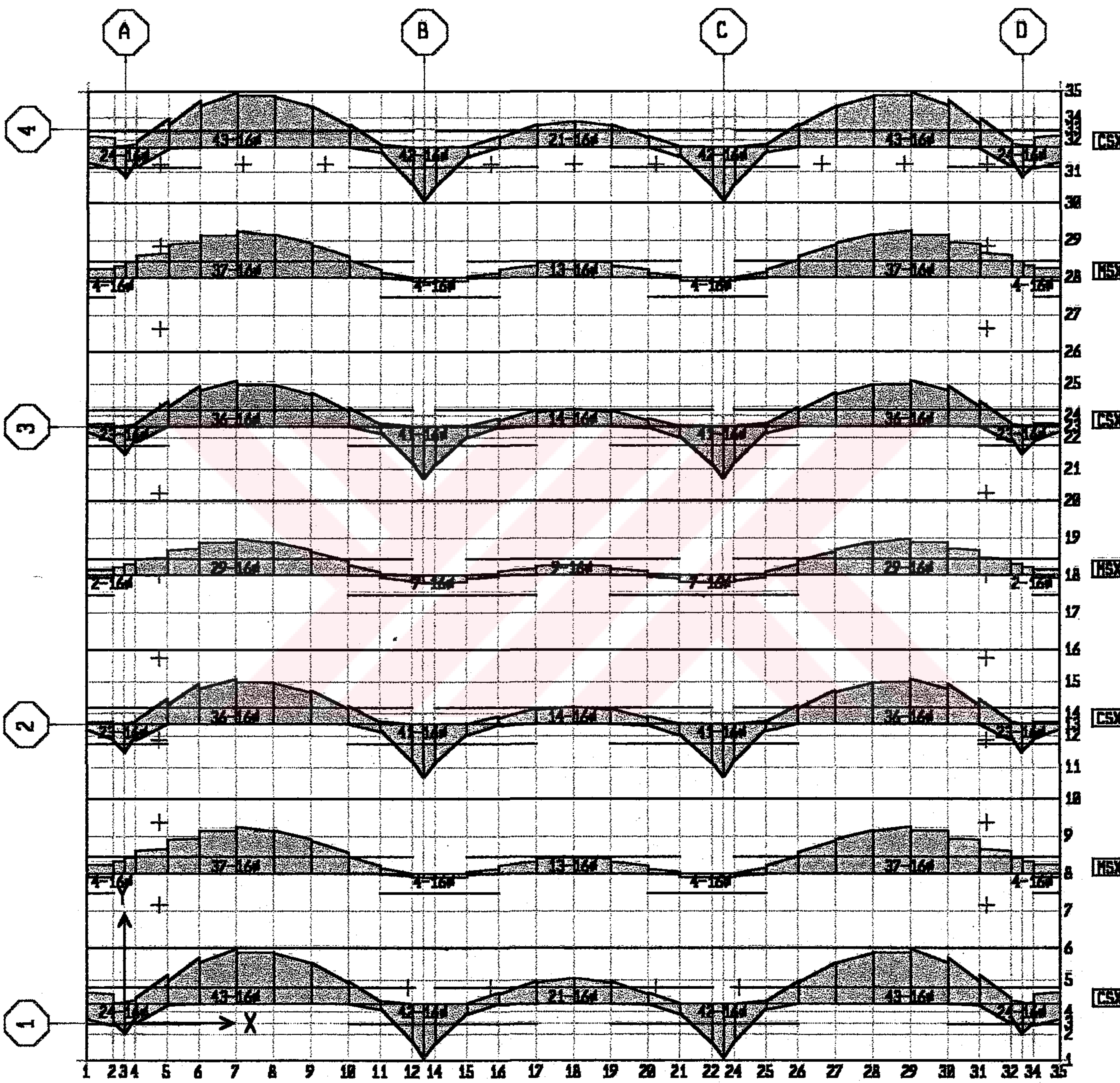
-Strip Reinforcement (Sq-cm) - KN-m Units


A
B
C
D
4
3
2
1
CSX
MSX
X
Y

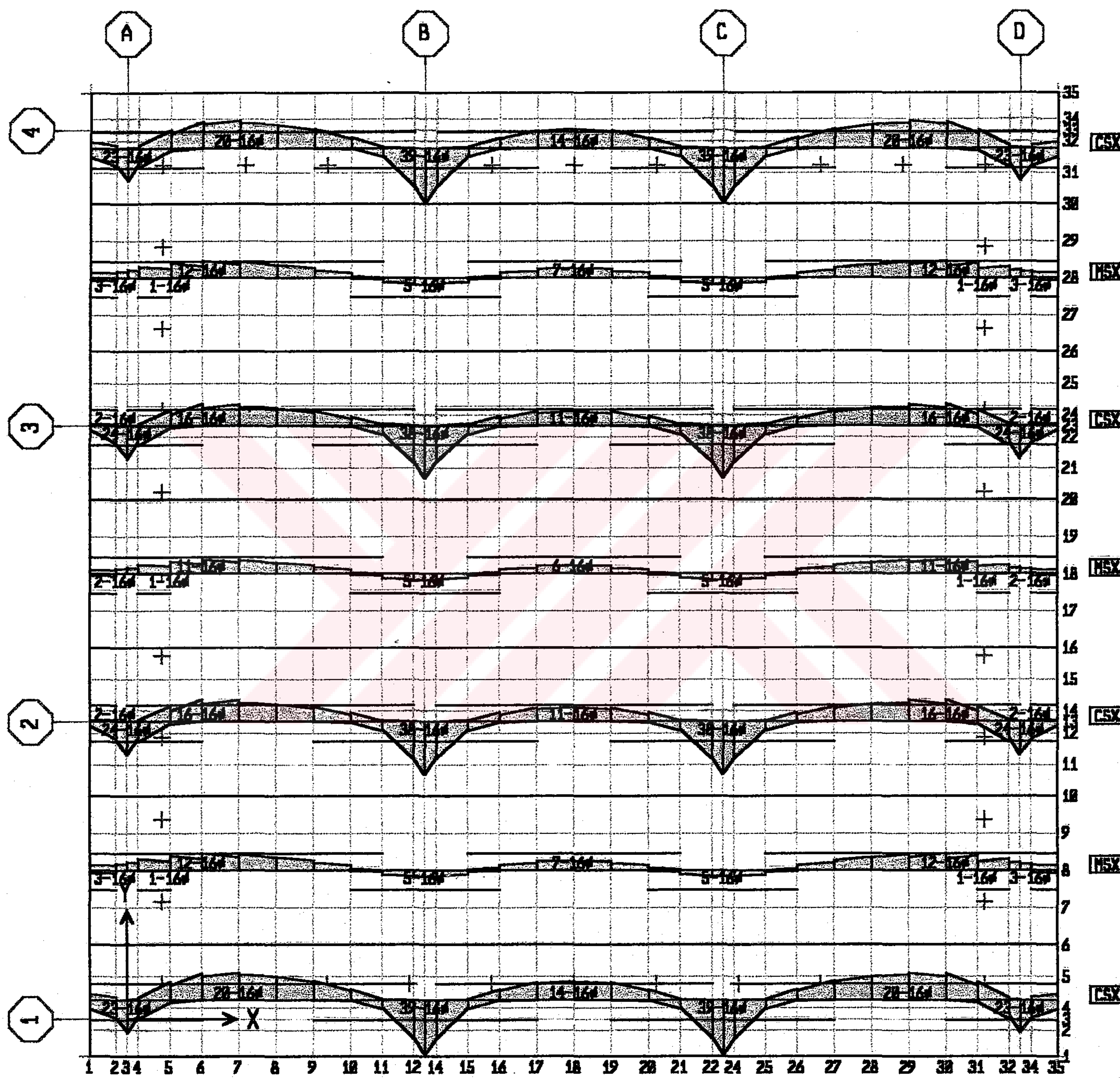

A
B
C
D
4
3
2
1
Y
X
CSX
MSX



AFE v7.01 - File: Mat Foundation - Þubat 2,2005 6:06 - Scale: Fit to Page
-Strip Reinforcement (Sq-cm) - KN-m Units

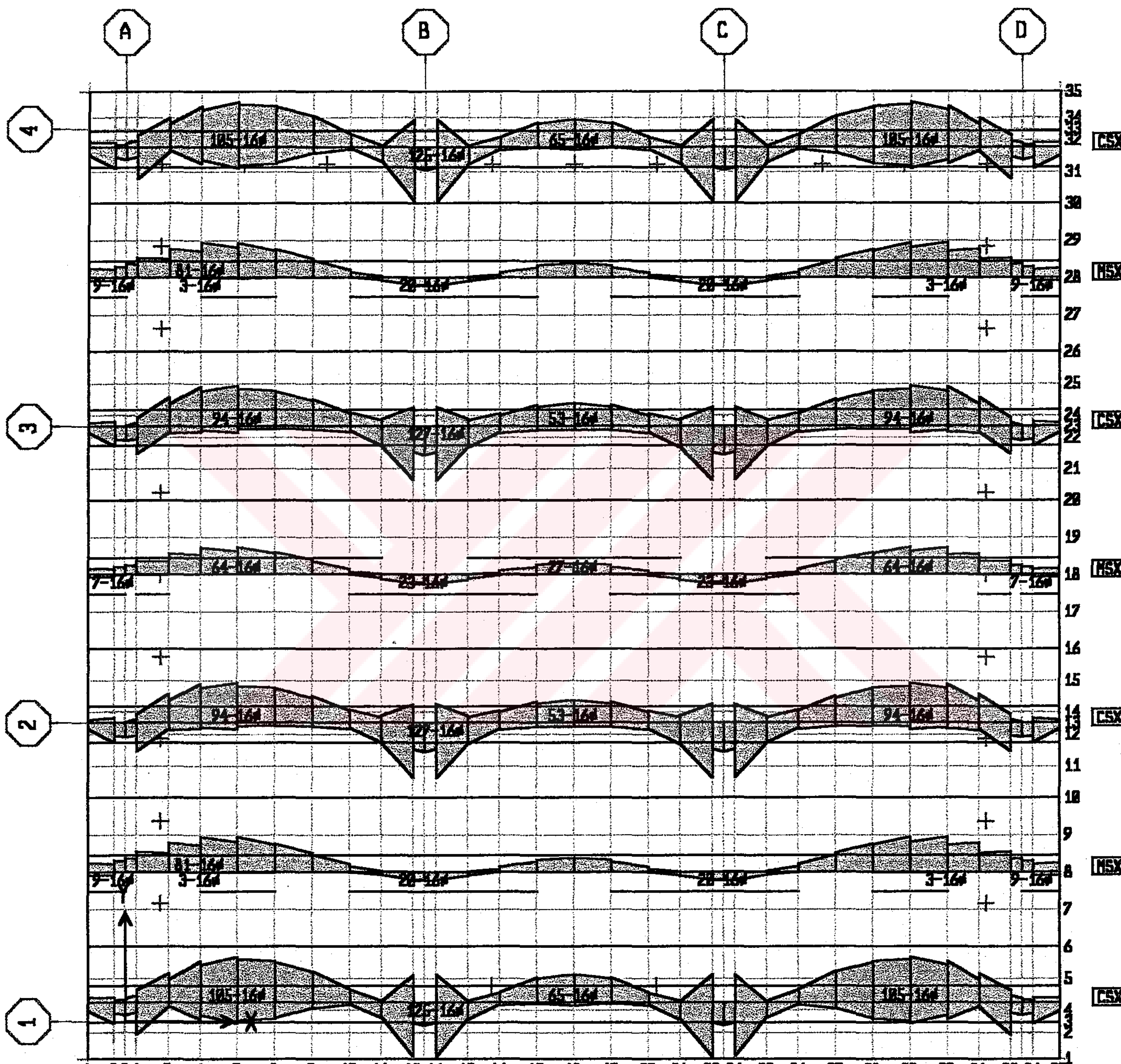
SAFE

D=1m Load=4000kN ks=375000kN/m3

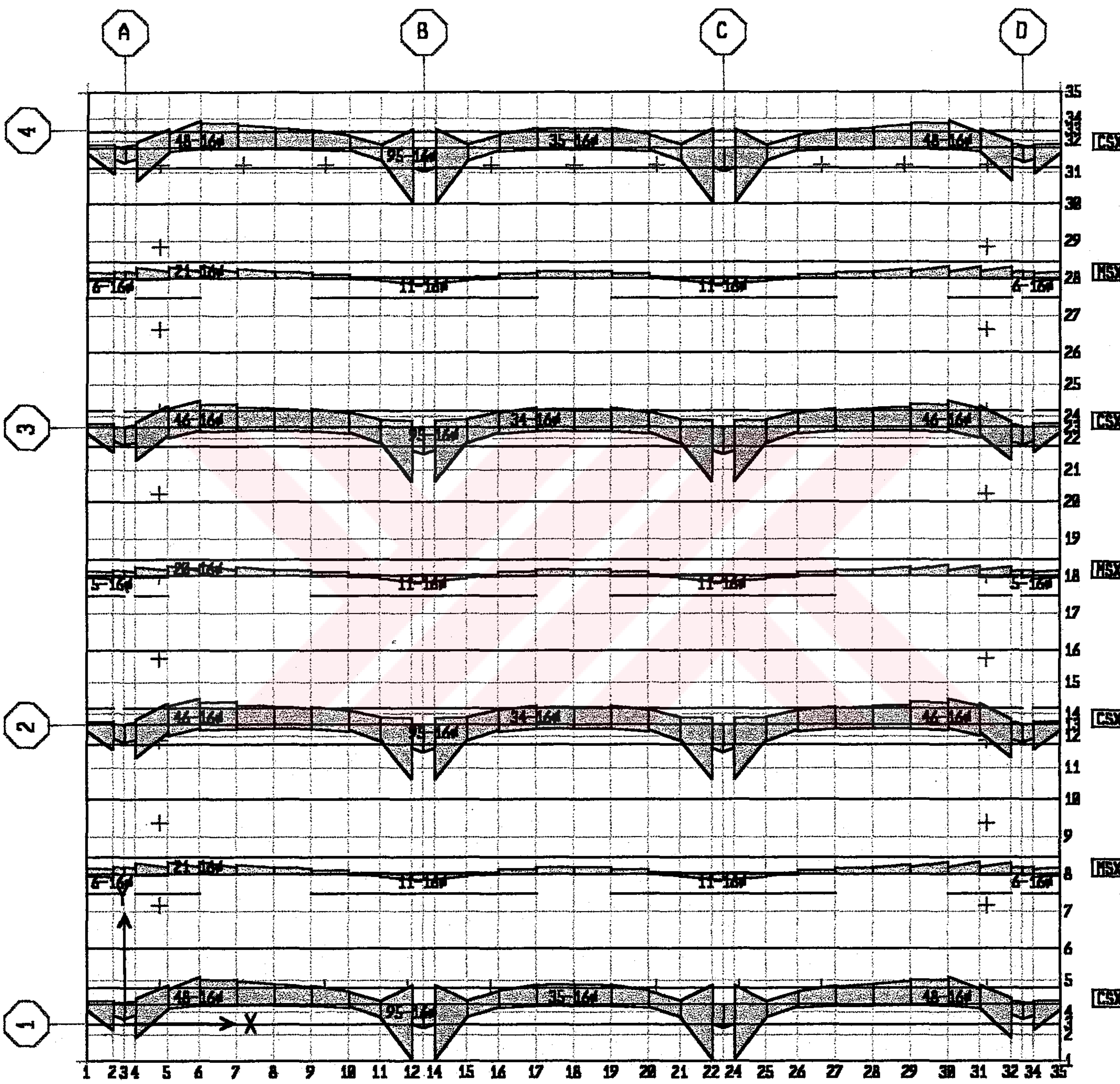


SAFE v7.01 - File: Mat Foundation - Þubat 2,2005 6:08 - Scale: Fit to Page

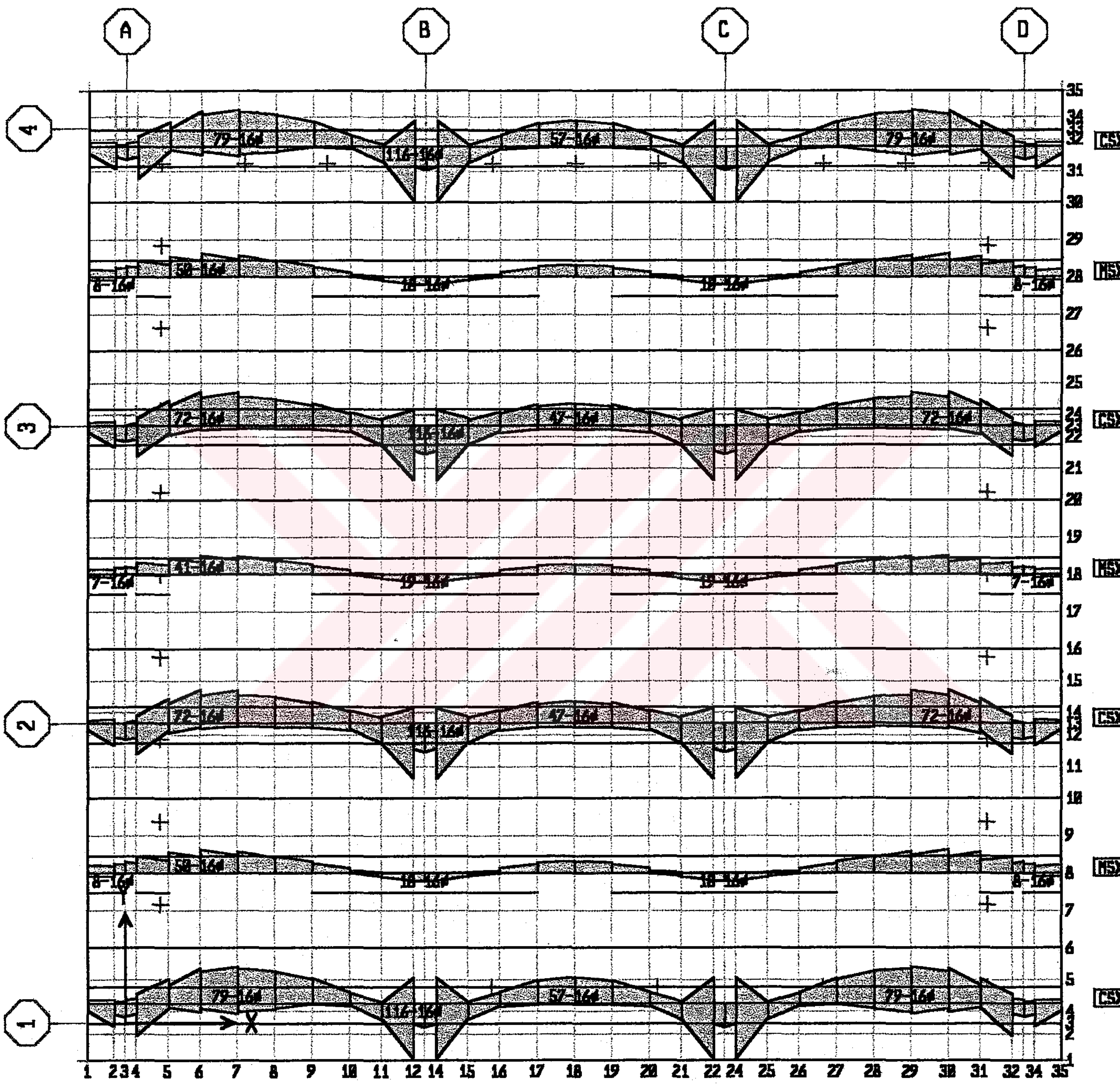
X-Strip Reinforcement (Sq-cm) - KN-m Units

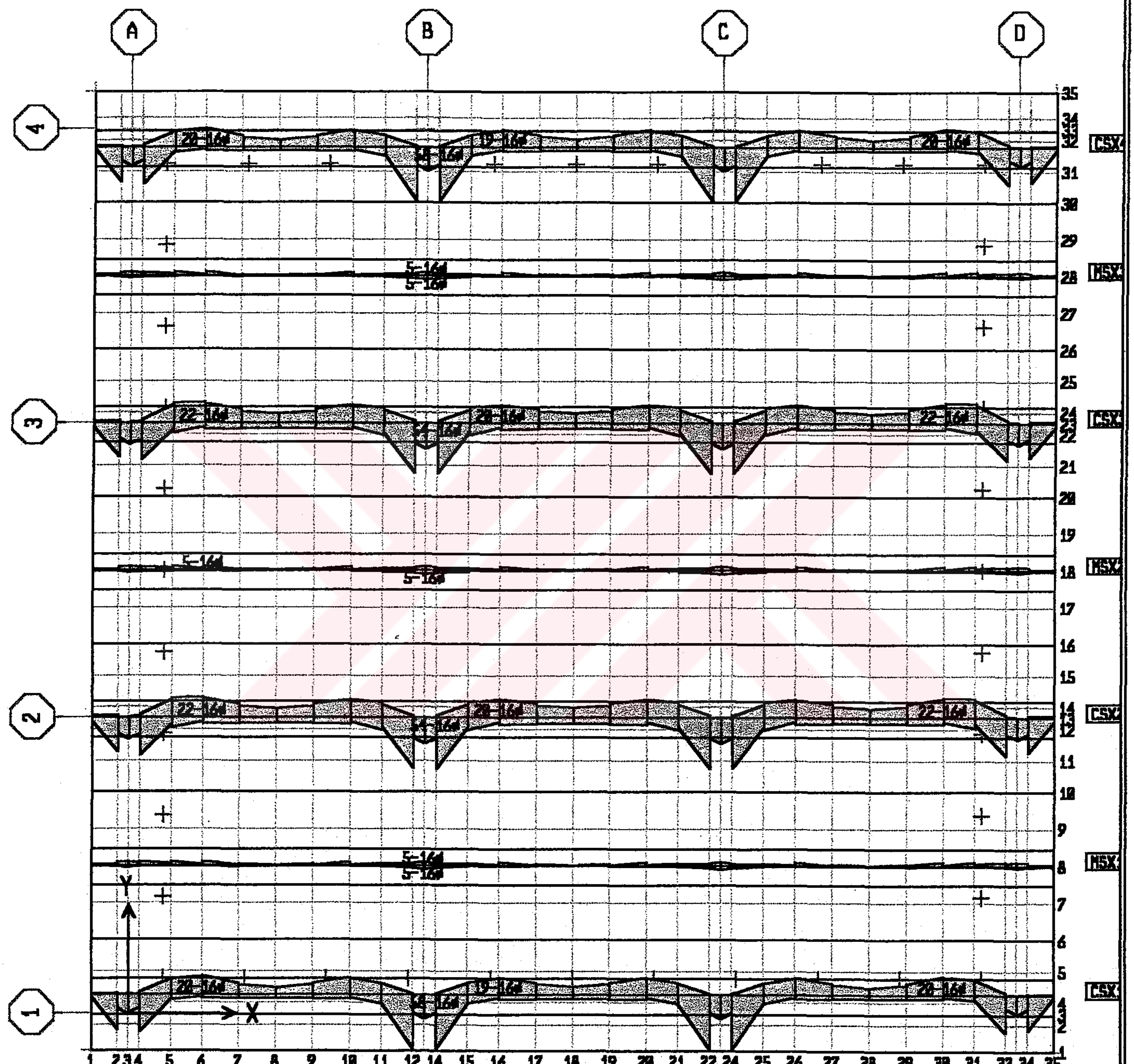


SAFE v7.01 - File: t0,3m - Þubat 2,2005 6:12 - Scale: Fit to Page
X-Strip Reinforcement (Sq-cm) - KN-m Units



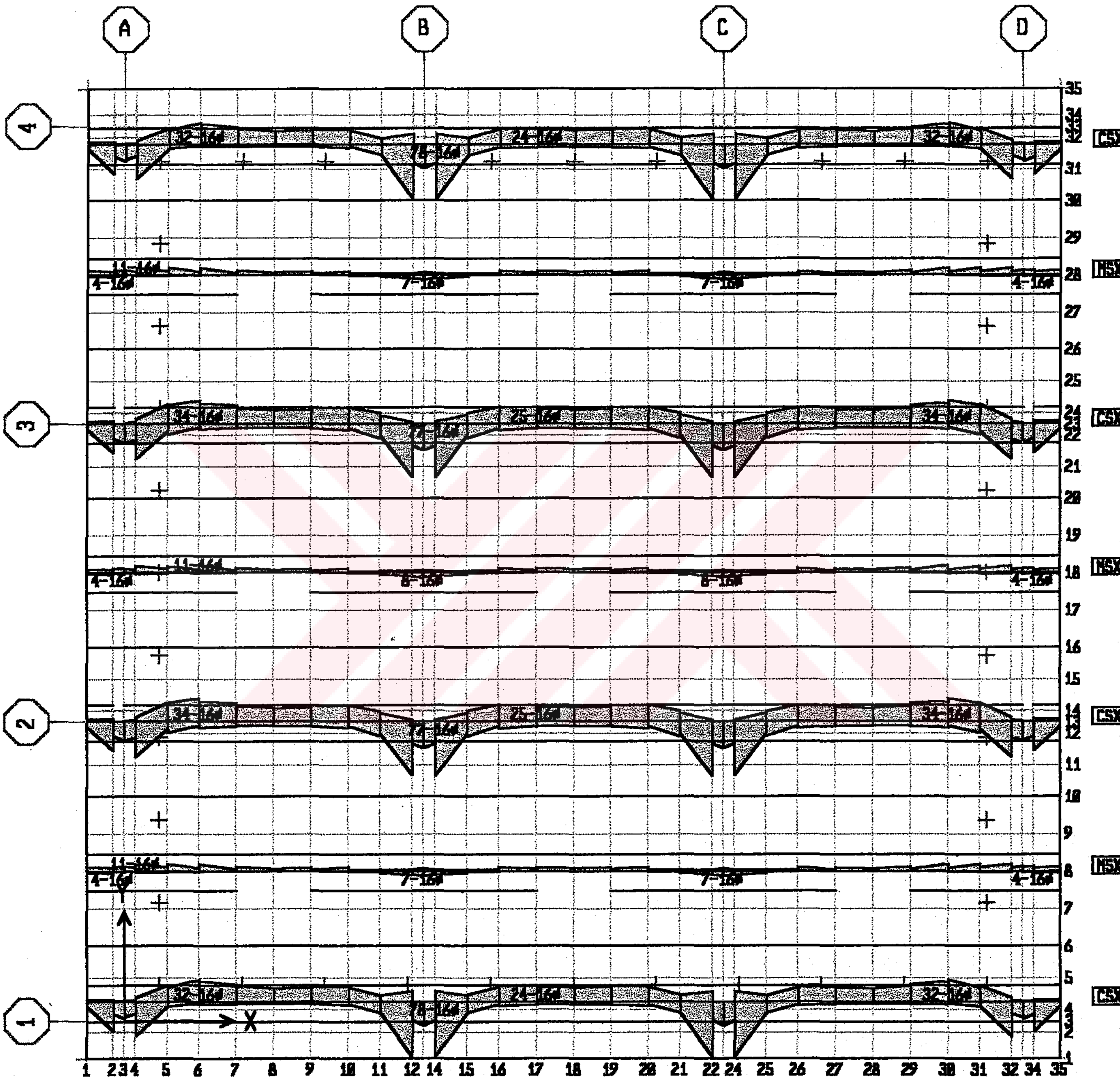
SAFE v7.01 - File: t0,3m - Þubat 2,2005 6:27 - Scale: Fit to Page
X-Strip Reinforcement (Sq-cm) - KN-m Units


A
B
C
D
4
3
2
1
CSX
MSX
79-16ø
116-16ø
57-16ø
58-16ø
8-16ø
18-16ø
72-16ø
47-16ø
41-16ø
7-16ø
19-16ø
X
Y

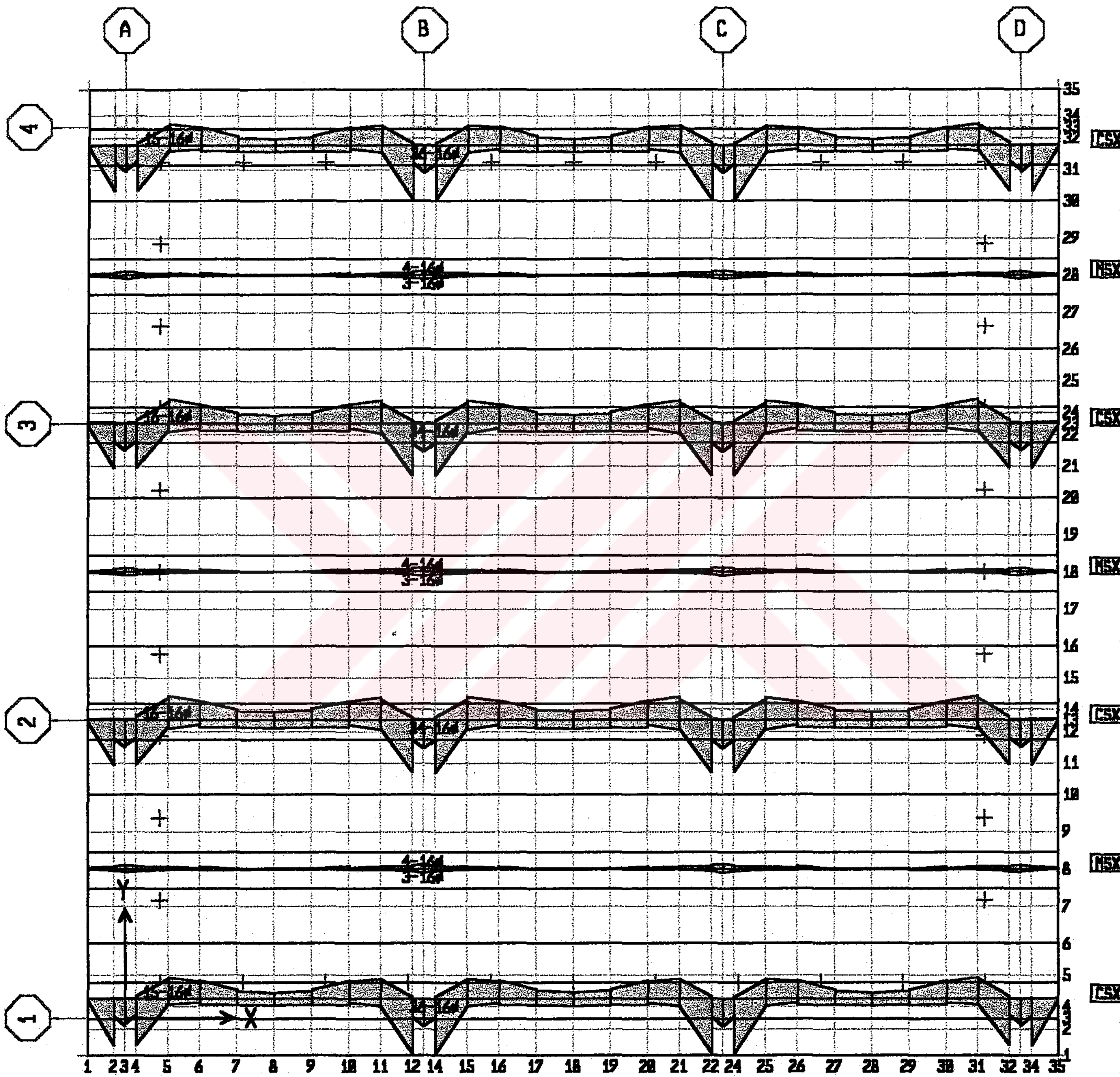


SAFE v7.01 - File: t0,3m - Þubat 2,2005 6:17 - Scale: Fit to Page
X-Strip Reinforcement (Sq-cm) - KN-m Units

D=0.3m Load=3000kN ks=50000kN/m3



SAFE v7.01 - File: t0,3m - Þubat 2,2005 6:36 - Scale: Fit to Page
X-Strip Reinforcement (Sq-cm) - KN-m Units


A
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C
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1
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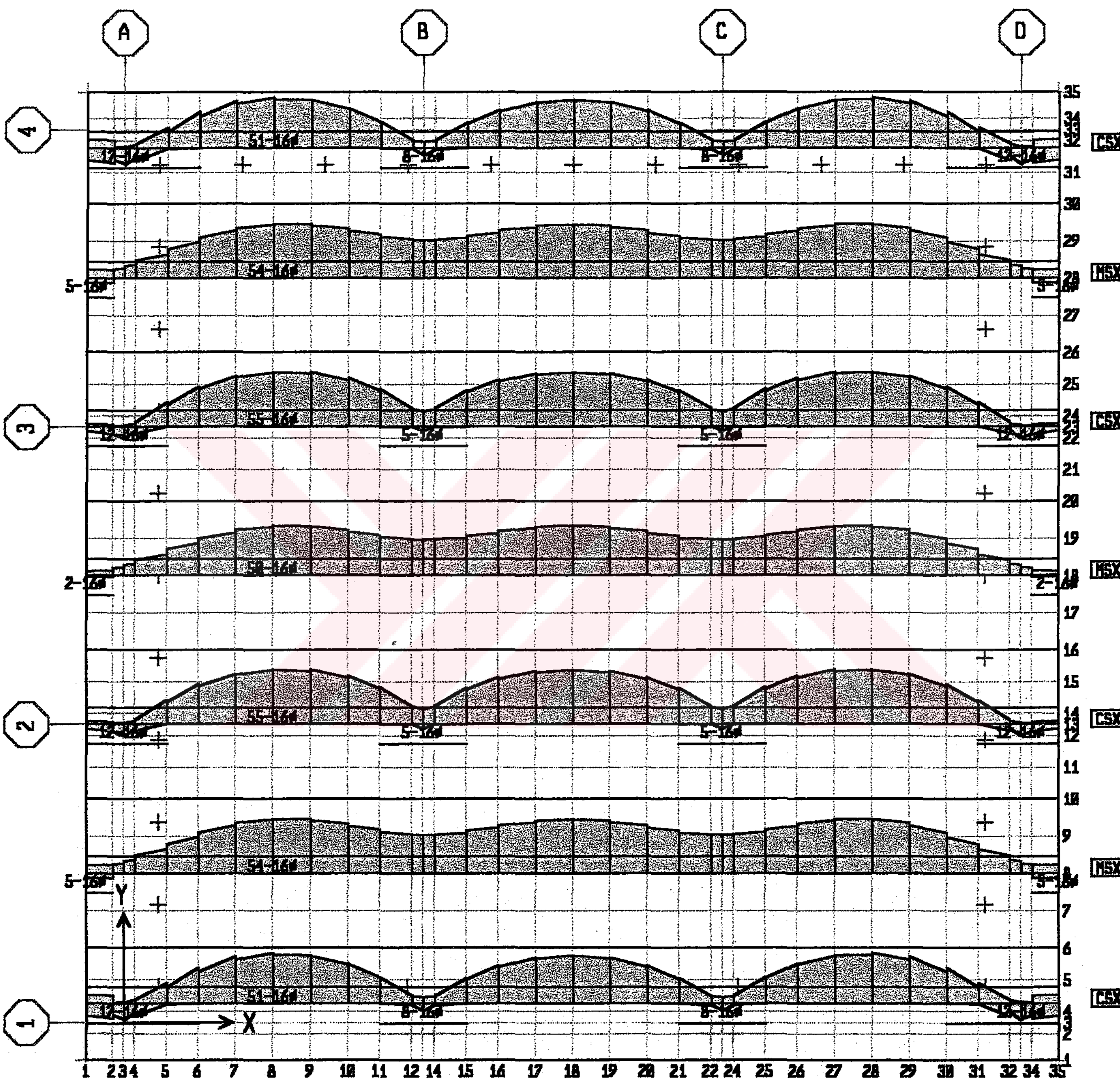
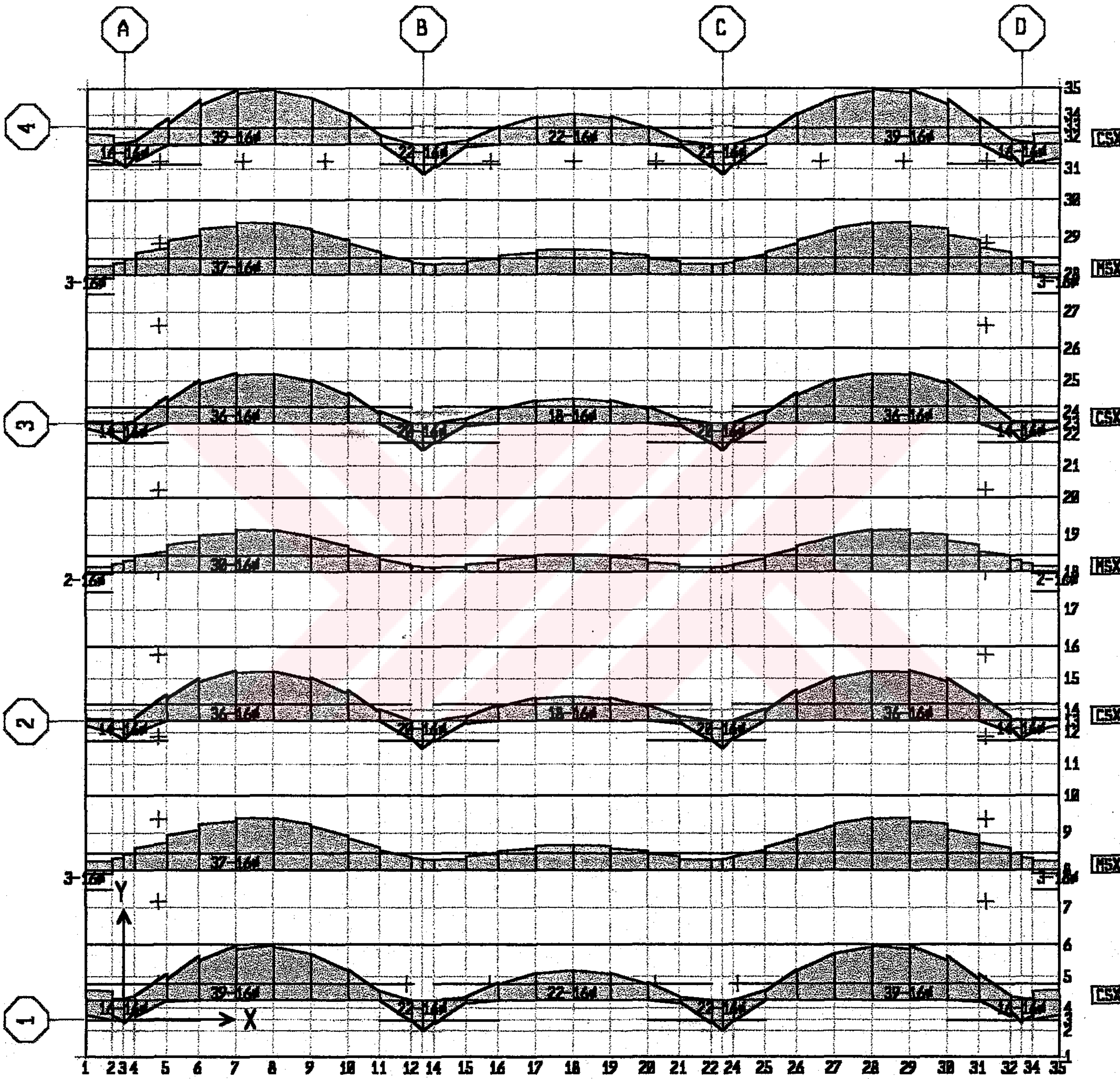
MSX
15-16ø
14-16ø
4-16ø
3-16ø
X
Y

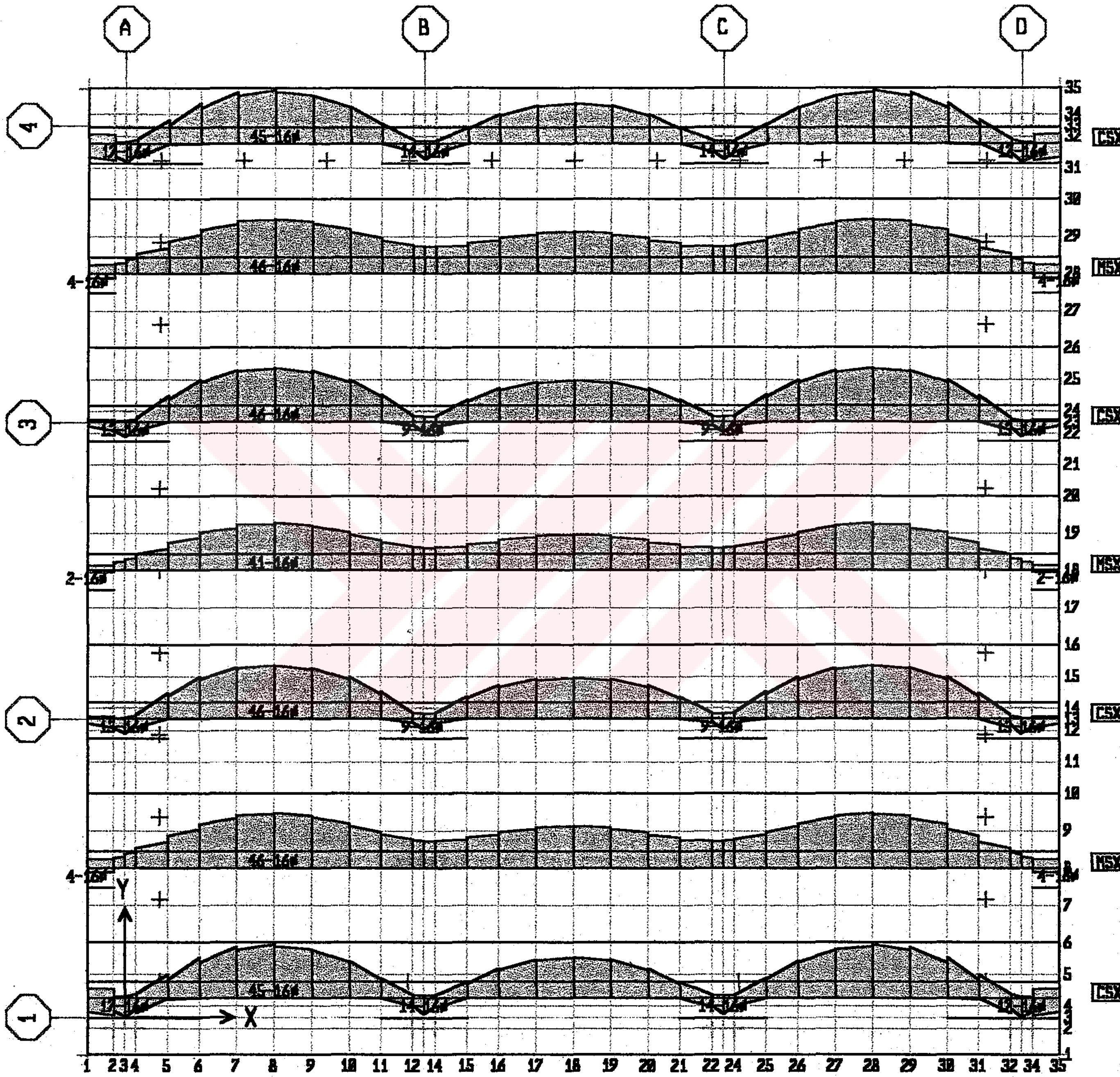


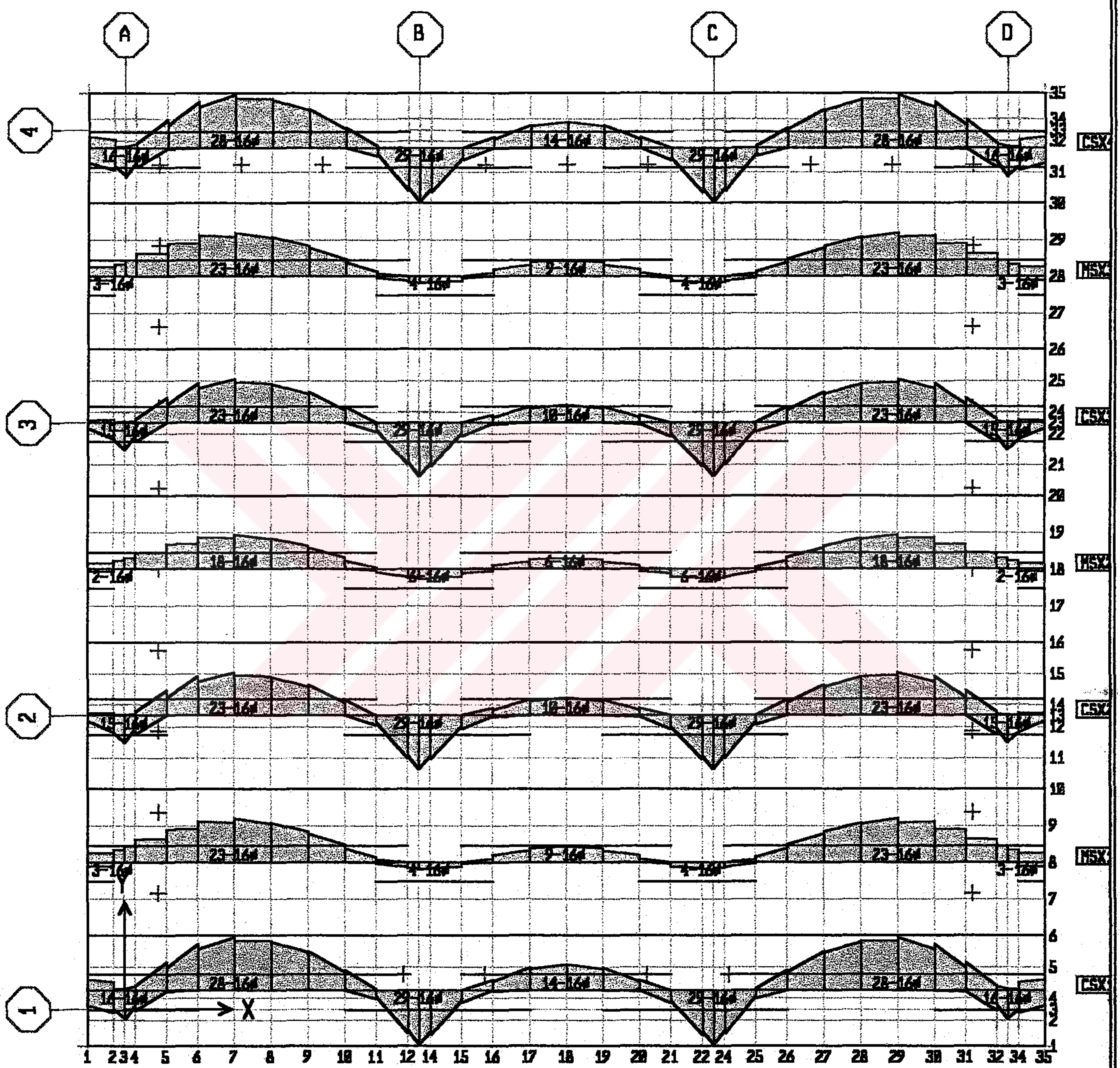
SAFE v7.01 - File: t1.2m - Þubat 2,2005 9:58 - Scale: Fit to Page
X-Strip Reinforcement (Sq-cm) - KN-m Units



FE v7.01 - File: t1.2m - Þubat 2,2005 9:56 - Scale: Fit to Page
Strip Reinforcement (Sq-cm) - KN-m Units

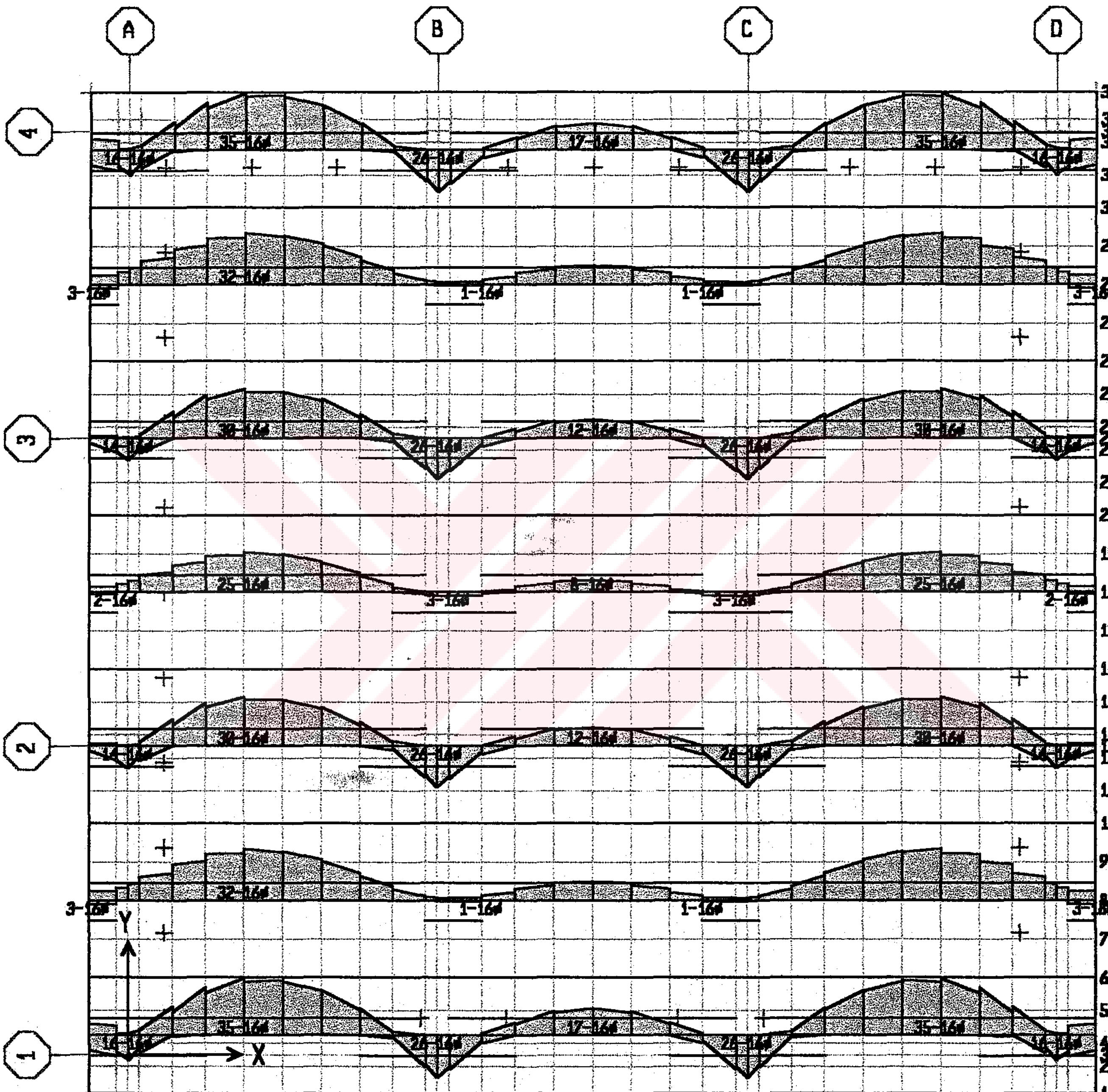
Mat Foundation 26x26m
D=1.2m Load=3000kN ks=9600kN/m3





SAFE v7.01 - File: t1.2m - Þubat 2,2005 10:05 - Scale: Fit to Page
-Strip Reinforcement (Sq-cm) - KN-m Units

D=1.2m Load=3000kN ks=50000kN/m3



SAFE v7.01 - File: t1.2m - Þubat 2,2005 10:07 - Scale: Fit to Page
Strip Reinforcement (Sq-cm) - KN-m Units