DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL APPLIED SCIENCES

PILED RAFT APPLICATIONS

by
Kubilay ÖZTÜRK

September, 2008
İZMİR

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Ph.D. THESIS EXAMINATION RESULT FORM

We have read the thesis entitled **“PILED RAFT APPLICATIONS”** completed by **KUBİLAY ÖZTÜRK** under supervision of **PROF. DR. ARİF ŞENGÜN KAYALAR** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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PILED RAFT APPLICATIONS

ABSTRACT

The choice of the foundation type depends on some factors such as intended use of the structure, structural loads, geotechnical and environmental circumstances of the construction site and some other factors. In most case shallow or raft foundations may provide enough safety factors under service loads. Piles are used when design criteria's of shallow or raft foundations are over passed. The conventional design of piled foundations neglects the contribution of the raft, although the raft contributes more or less to the bearing capacity and the settlement. The foundation system of assuming that piles and the raft both carrying the structural loads together is called Piled Raft Foundations.

In this study it is aimed to give a general knowledge about piled raft foundations, and their design criteria's. The hand calculation method and a worked example given by Poulos (2000) are studied here. The same foundation model was analyzed with a finite element program (PLAXIS 3D 1.1) and the results were compared. In the second part of the thesis, the idea of using piled raft foundations in İzmir is assessed. Through this aim, a well known area in Mavişehir region for a 16 stories building model is handled. The raft foundation, the conventional piled foundation and piled raft foundation analyses are performed and results are compared.

Keywords: piled rafts, pile, raft, stiffness, settlement, finite element

KAZIKLI RADYE UYGULAMALARI

ÖZET

Temel tipinin seçimi yapı kullanım amacı, yapısal yükler, inşaat sahasına ait geoteknik ve çevresel etkenler gibi birtakım faktörlere bağlıdır. Birçok durumda yüzeysel veya radye temeller servis yükleri altında yeterli güvenlik katsayısını sağlamaktadır. Kazıklar, yüzeysel veya radye temellerin dizayn kriterlerinin aşıldığı durumlarda kullanılır. Gerçekte kazıklı temel sistemlerinde radye taşıma gücü ve oturmaya karşı az ya da çok katkı yapmaktadır. Ancak geleneksel kazıklı temel tasarımı bu katkıyı göz önünde bulundurmaz. Yükün radye ve kazıklar tarafından beraber taşındığı kabulüne dayanan temel sistemine "kazıklı radye temel" adı verilir.

Bu çalışmada amaç "kazıklı radye" temeller ile ilgili genel bir bilgi vermek ve tasarım kıstaslarını açıklamaktır. Poulos(2000) tarafından önerilen bir elle hesap yöntemi ile sayısal uygulaması üzerinde çalışılmıştır. Örnekte ele alınan temel ve zemin modeli üç boyutlu bir sonlu elemanlar programı (PLAXIS 3D 1.1) ile analiz edilip ve sonuçlar karşılaştırıldı. Tezin ikinci bölümünde kazıklı radye temellerin İzmir' de uygulanması değerlendirilmiştir. 16 katlı bir yapı modeli Mavişehir bölgesinde iyi bilinen bir saha için ele alınmıştır. Radye temel, geleneksel kazıklı temel ve kazıklı radye temel analizleri yapılmış ve sonuçlar karşılaştırılmıştır.

Anahtar sözcükler: kazıklı radye, kazık, radye, rijitlik, oturma, sonlu elemanlar

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CHAPTER ONE
INTRODUCTION

"Every piled foundation behaves like a piled raft, with the exception of those cases where there is no contact between the raft and the soil as in offshore structures" (Sanctis & Mandolini, 2006). Also in piled foundations it is possible that seperation between the raft and the soil in the case of soil profiles which are likely to undergo consolidation settlements due to external causes. So that the conventional pile design method is a realistic approach in such a problem. But in the case of soil profiles consisting of relatively stiff clays or dense sands the raft can provide a significant proportion of the required load capacity and stiffness (Poulos, 2000). It is then needed to be considering raft – soil interaction, because the structural loads are carried by both the raft and piles.

Piled raft foundations are complicated problems and have to be designed by using appropriate computer programs. Although it is necessary to use a computer program, a simple hand calculation method is needed to check if computer solutions are logical or not. In the second chapter, a two – stage process and a hand calculation method considered by Poulos (2000) in piled raft design is studied. The worked example given in the same article is explained step by step. A finite element analyze program PLAXIS 3D 1.1 was used to analyze the same model used in the hand calculation example and the hand calculation and the computer solutions are compared.

In the third chapter, the idea of using piled raft foundations in İzmir is assessed. Trough this aim a well known area in Mavişehir region is handled for a 16 story building. To form the idealized soil profile, all of the field and laboratory test data given in the soil investigation report prepared by Ege Temel Sondajcılık for the investigation site were studied. The hand calculations were performed for raft foundation, conventional piled foundation and piled raft foundation models and results were compared. After the hand calculations raft and piled raft foundation models were analyzed with PLAXIS 3D 1.1.

In the last chapter the results and the general evaluations on piled rafts and applications of piled rafts are given. The geotechnical data, idealized soil profile, analyzes, models and the results of analyzes are given in the appendices.

CHAPTER TWO
DESIGN PHLOSOPHIES OF PILED RAFTS

2.1 Piled Raft Behavior

The foundation engineering is a combination of two principles: soil mechanics and structural engineering. The importance of the interaction of soil–structure directly related to the structural loading and the foundation system. However the load sharing behavior of piled raft foundations depends on many variables. In the case that different foundation systems work together, the interaction between each other must be taken into account.

Randolph (1994) identified three different design philosophies for piled rafts:

1. *Conventional Approach*: foundation is designed as a pile group with a regular spacing of the piles over the complete foundation area. Piles carry the major part of the load while making the allowance of pile cap. The 60 – 70 % of the structural loads being carried by the piles.
2. *Creep Piling*: Each pile are designed to operate at a working load at which significant creep starts to occur at the pile soil interface, typically at about 70 – 80 % of its ultimate load capacity. Sufficient piles are included to reduce the net contact pressure of the soil.
3. *Differential settlement control*: the piles are located strategically in order to reduce the differential settlements, rather than to substantially reduce the overall average settlement.

Poulos (2000) suggests a more extreme version of creep piling, in which the full load capacity of the piles is utilized: that is, some or all of the piles operate at 100% of their ultimate load capacity. In this case although piles contribute to increasing the ultimate load capacity, they are used primarily as settlement reducers.

Poulos (2000) represented the load settlement behavior of the above given assumptions in a graph (Figure 2.1). Curve 0 represents the case that the foundation is the raft itself and it's clearly seen that the raft's settlement limits are over passed under design loads. Curve 1 represents the conventional design philosophy, for which the behavior of the pile – raft system is governed by the pile group behavior, and which may be largely linear at the design load. Curve 2 represents the case of creep piling, where the piles operate at a lower factor of safety. Both of curves are linear up to design load, although Curve 1 and Curve 2 provide the settlement criterion under design loads. Curve 3 (ra^2 with piles designed for full utilization of capacity) provides the settlement criterion with less piles. Piles are designed with the strategy of using as settlement reducer. The load settlement curve is non – linear at the design load but the safety factor of the foundation system is satisfied. Also piles work with full capacity under design load, and it is a more economical solution than Curve 1 and Curve 2.

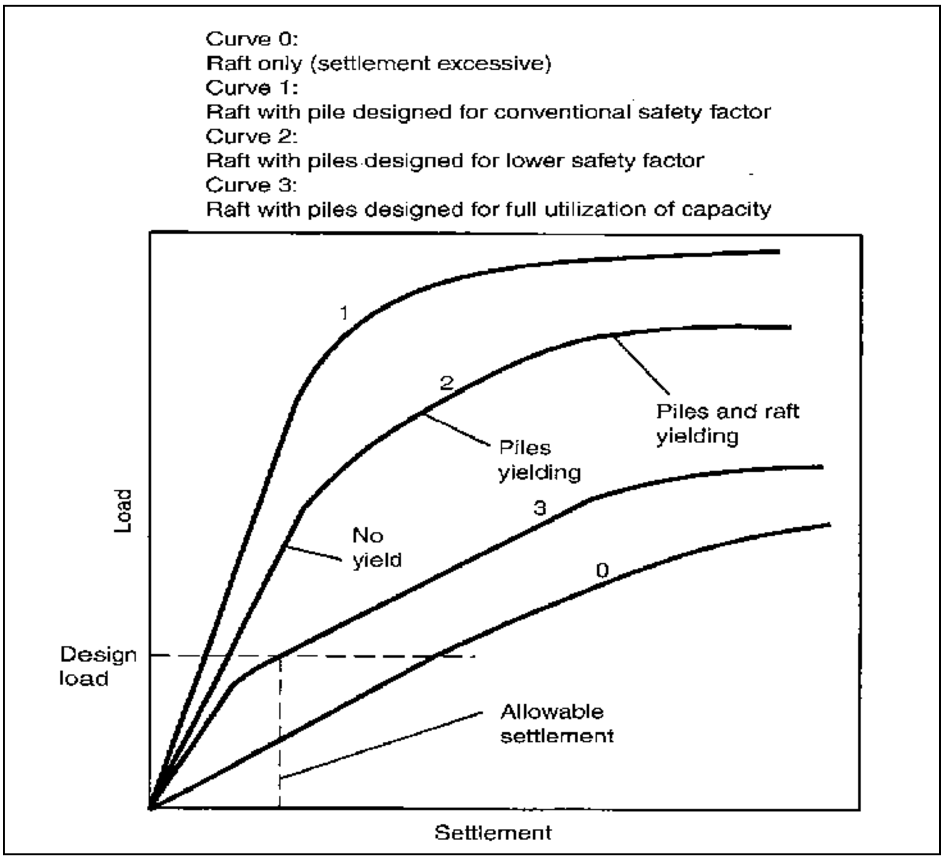

Figure 2.1 Load – settlement curves for piled rafts according to various design philosophies (Poulos, 2000).

There are also different categorization methodologies of piled rafts in the literature. Russo & Viggiani (1998) grouped piled rafts as *small and large piled rafts*. In the first group the ratio of the width of the raft (B_r) to the length of the piles is generally small then unity ($B_r /L < 1$). The problem of bearing capacity failure is of particular concern for the small piled rafts, and for the case of soft clay soils (Sanctis & Mandolini 2006). In large piled rafts the bearing capacity of the raft alone is satisfies the design criterion, while piles are designed for the settlement and differential settlement reducers.

Many researchers have obtained that the use of piled raft assumption is generally gives a considerable economy in the case that the raft satisfies the required bearing capacity, but settlement is over the allowable limits. In this case, because of the differential settlements, the additional forces act to the raft. Piles are here work as settlement and also differential settlement reducers, but Poulos (1991) has observed that raft can provide more or less the adequate load capacity in the case that soil profile consisting of relatively stiff clays or dense sands. Also gives some unfavorable circumstances for piled rafts. These are:

a. soil profiles containing soft clays near the surface;
b. soil profiles containing loose sands near the surface;
c. soil profiles which contain soft compressible layers at relatively shallow depths;
d. soil profiles which are likely to undergo consolidation settlements due to external causes
e. soil profiles which are likely to undergo swelling movements due to external causes.

In the case of soft clay or loose sand layers near the surface, the adequate load capacity and the stiffness may not be able to provide by the raft itself. When soil profiles containing soft compressible layers lying at relatively shallow depths; it reduces the contribution of the raft to the long – term settlement. Also consolidation settlement may cause to some spaces to occur or loose of contact between raft and the soil, thus increasing the load on piles. In the last case the swelling may result additional tensile forces on piles because of the swelling soil on the raft.

Poulos (2001) performed number of analysis for a piled raft model with following parameters to:

a. the number of piles
b. the nature of loading
c. raft thickness
d. applied load level

Some of the important results observed by Poulos (2001) are:

a. The maximum settlement decreases to a certain number of piles then becomes almost constant above this pile number. Similarly load carried by the piles increases with increasing pile numbers but becomes almost constant above a certain number of piles.

b. The differential settlement between the center and the corner piles does not change in a regular fashion with the number of piles. The smallest differential settlement occurred when piles were concentrated in the middle.

c. The smallest maximum bending moments are occurred in the case of minimum differential settlement (piles are concentrated in the middle). The maximum bending moments for concentrated loadings are substantially greater than for uniform loading.

d. The maximum settlement and the percentage of load carried by the piles are not very sensitive to raft thickness. It has little effect on load sharing or maximum settlements.

e. Increasing the raft thickness reduces the differential settlement, but generally increases the maximum bending moment.

Many researchers interested in the behavior of piled rafts and developed several methods. Poulos, Small, Ta, Shinha & Chen (1997) identified three broad classes of analysis method:

- Simplified calculation methods
- Approximate computer based methods
- More rigorous computer based methods

In this thesis the first and the third methods are studied.

2.2 A Hand Calculation Method

Simplified methods are based on the hand calculations and they are generally used for controlling the more rigorous computer based solutions if they are logical or not. Although there are many hand calculation methods in the literature, one of them suggested by (Poulos 2000) is studied and detailed here.

Preliminary design process of piled rafts can be grouped under four main topics (Figure – 2.2). These design processes are described here using the worked example of Poulos (2000).

The piled raft system and loading conditions shown in Figure 2.3 is consisting of nine piles with a height of 15 m and a raft of 0.5 m tick. The soil stratum is a single clay layer with a depth of 25 m. It is needed to check the foundation system for the minimum design criteria's of:

a. overall factor of safety of 2.5 against bearing capacity, overturning, and lateral failure for the ultimate load case;
b. Long – term average settlement of 50 mm and a maximum differential settlement not exceeding 10 mm.

Long term and short term loadings are given in the example. In short term loadings, in clayey layers, because of the stress increment, the excess pore water pressure generates. This is an undrained loading problem. Elasticity modulus for undrained conditions is bigger than for the drained conditions, because the compressibility of water takes place and it is very small. Inversely for the long term loadings, the effective parameters have to be used. The excess pore water pressure decreases with time, and thus an effective stress increment generates on soil particles. During this process the compressibility of soil occurs.

2.2.1 Estimation of ultimate geotechnical capacity

The estimation of ultimate geotechnical capacity of piled foundation can be obtained for three loading cases (Figure 2.4).

2.2.1.1 Vertical loading

Geotechnical capacity of the piled raft foundation under vertical loading is estimated as:

a. The sum of the ultimate capacities of the raft plus all the piles in the system;

b. The ultimate capacity of a block containing the piles and the raft, plus that of the portion of the raft outside the periphery of the pile group.

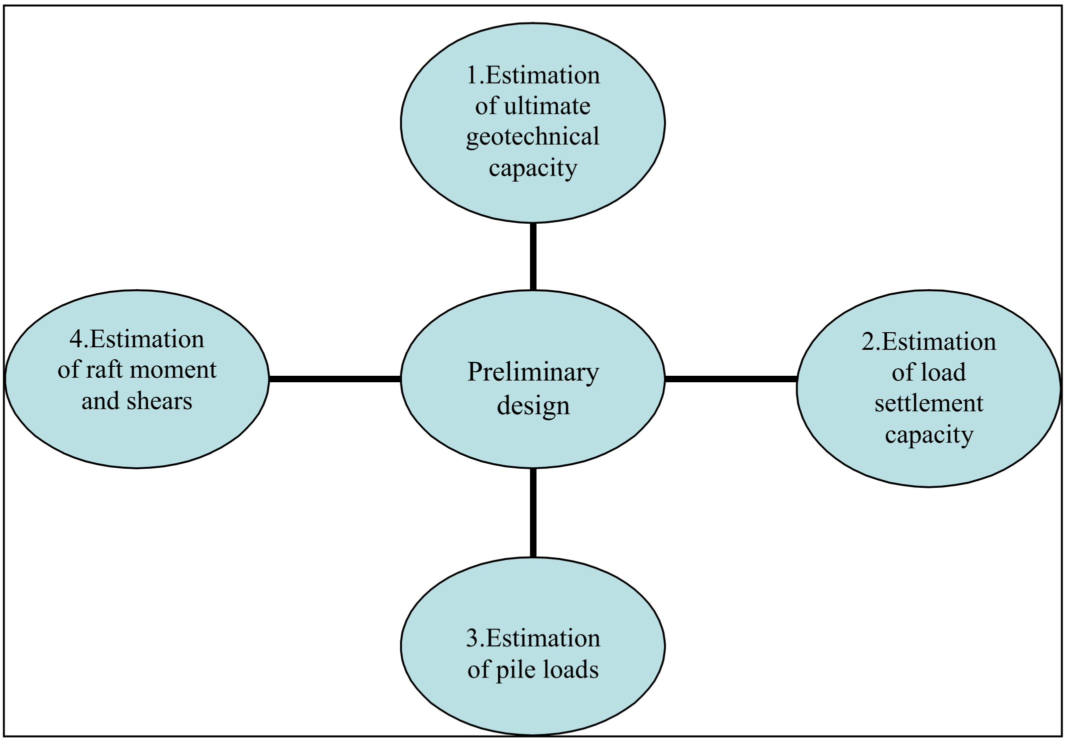

Figure 2.2 Preliminary design processes.

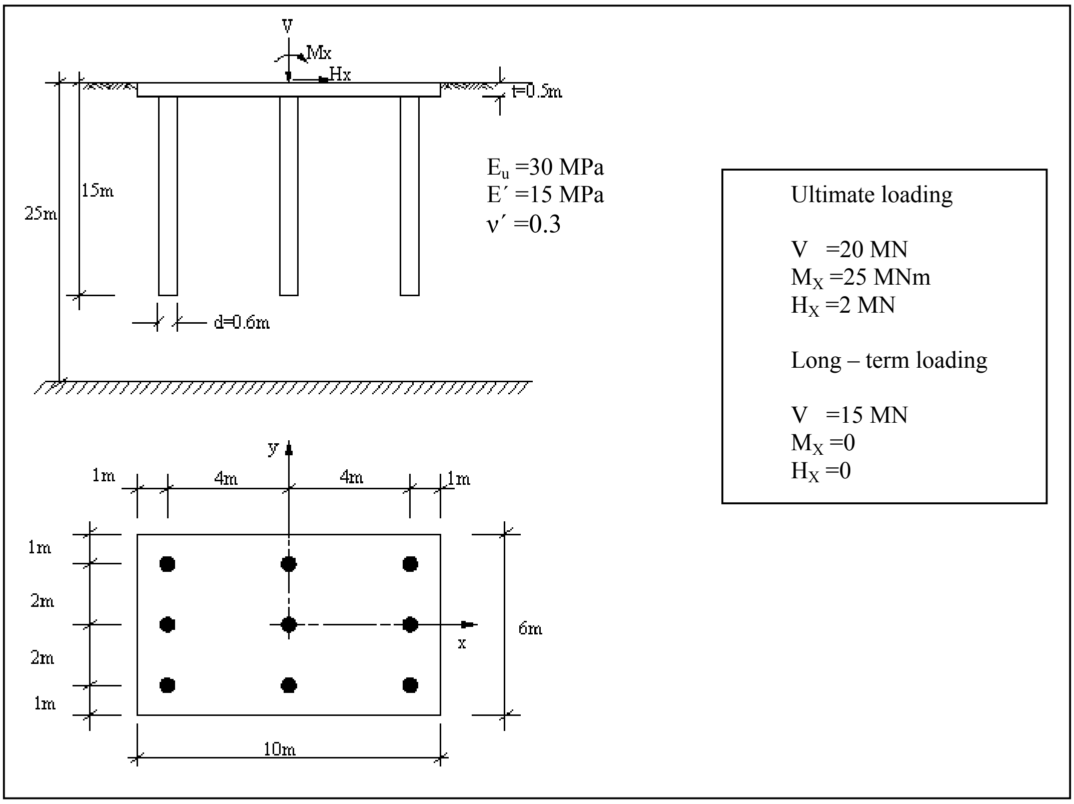

Figure 2.3 Piled raft foundation model used in the worked example.

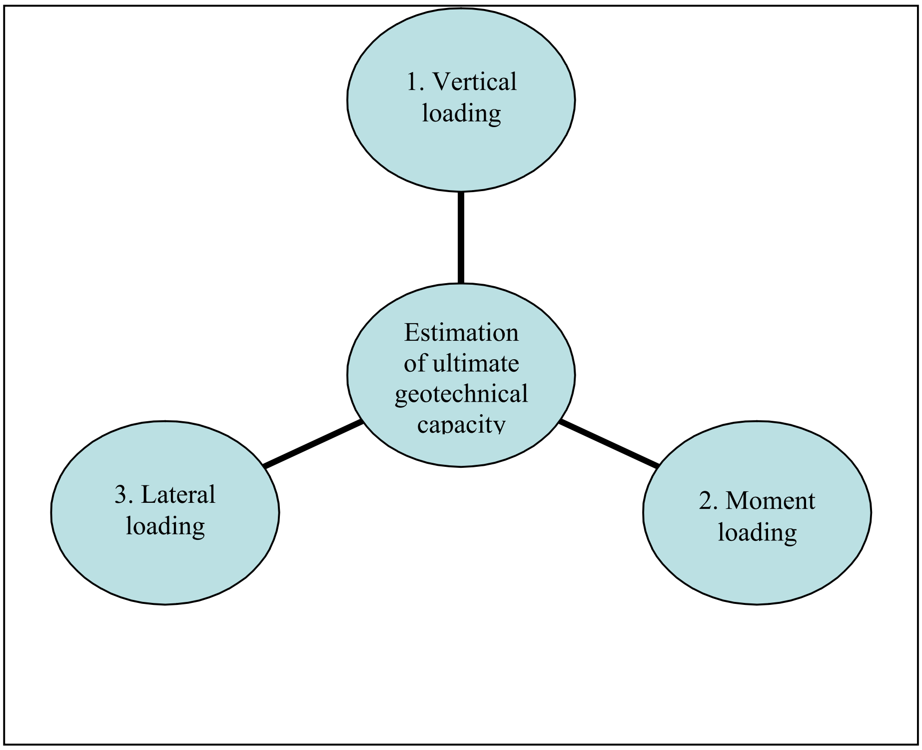

Figure 2.4 Estimation of ultimate geotechnical capacity process

The average ultimate shaft frictions are given as 60 kPa in compression and 42kPa in tension and are assumed constant with depth. The ultimate end bearing capacity is 900 kPa. The axial capacity for single pile in compression is:

$$q_{pc} = A_{sp} \times f_{sc} + A_{bp} \times q_{cp}$$

Where A_{sp} : pile shaft area

f_{sc} : pile shaft friction in compression

A_{bp} : pile tip area

q_c : pile end bearing capacity

$$q_{pc} = (0.6 \times \pi \times 15) \times 0.06 + \left(\frac{0.6 \times \pi}{4}\right) \times 0.9$$

$$q_{pc} = 1.95MN$$

$$Q_{pc} = n \times q_{pc}$$

Where Q_{pt} : the total axial pile capacity under compression loads

n : number of piles

$$Q_{pc} = 9 \times 1.95$$

$$= 17.55MN$$

The axial capacity for single pile in tension is:

$$q_{pt} = A_{sp} \times f_{st}$$

Where A_{sp} : pile shaft area

f_{sc} : pile shaft friction in tension

$$q_{pt} = (0.6 \times \pi \times 15) \times 0.042$$

$$= 1.20MN$$

For the raft it is assumed that the ultimate bearing capacity below raft is:

$$P_{ur} = 6 \times c_u$$

Where c_u : undrained shear strength

$$P_{ur} = 0.6MPa$$

while the undrained shear strength of the soil c_u = 0.1.

The total bearing capacity of the raft is therefore:

$$Q_r = A \times P_{ur}$$

Where A : the area of the raft surface

$$Q_r = (10 \times 6) \times 0.6$$
$$= 36MN$$

If the raft and the pile capacities are added, the total capacity of the foundation in compression is:

$$Q_{pr} = Q_r + Q_{pc}$$
$$= 36 + 17.55$$
$$= 53.55MN$$

The bearing capacity of the block containing the raft and the piles must now be considered. The outer dimensions of the pile group are 4.6x8.6m. The block capacity is:

$$\begin{aligned} \textit{The block capacity} &= \textit{shaft friction} + \textit{the end axial capacity} + \\ &\quad \textit{the bearing capacity of the rest of the raft} \\ &= [2 \times (8.6 + 4.6) \times 0.1 \times 15] + [8.6 \times 4.6 \times 0.9] + \\ &\quad [(10 \times 6 - 8.6 \times 4.6) \times 0.6] \\ &= 39.6 + 35.6 + 12.6 \\ &= 87.46MN \end{aligned}$$

This exceeds the sum of the raft and the pile capacities, and the design value of the ultimate capacity of the foundation is 53.55MN. The corresponding factor of safety is:

$$F = \frac{53.55}{V}$$
$$= \frac{53.55}{20}$$
$$= 2.67, \text{which satisfies the design criterion}$$

2.2.1.2 Moment capacity

The ultimate moment capacity of a piled raft can be estimated approximately as the lesser of:

a) The ultimate moment capacity of the raft (M_{ur}) and the individual piles (M_{up})

b) The ultimate moment capacity of a block containing the piles, raft and the soil (M_{ub}) (Poulos, 2000).

a) If we are working on a uniform loaded rigid plate to obtain the maximum ultimate moment sustained by the soil, rotation center will be the center of the plate (Figure 2.5). In this case while the half of the raft is subjected to tensile forces, the other half will be subjected to compressive forces.

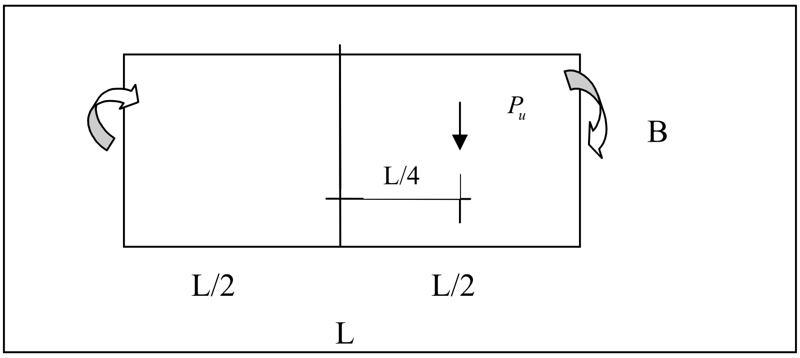

Figure 2.5 Moment loading of a rigid plate

Then the moment is:

$$M_m = \frac{P_{ur} B L^2}{8} \tag{2.1}$$

$$= \frac{0.6 \times 6 \times 10^2}{8}$$

$$= 45 MNm$$

The ultimate moment capacity of the raft is:

$$M_{ur} = M_m \times \frac{27}{4}\frac{V}{V_u}\left[1 - \left(\frac{V}{V_u}\right)^{\frac{1}{2}}\right] \tag{2.2}$$

Where M_{ur} : the ultimate moment capacity of the raft

M_m : maximum possible moment that soil can support

V : applied vertical load

V_u : ultimate centric load

$$M_{ur} = 45 \times \frac{27}{4} \frac{20}{53.55} \left[1 - \left(\frac{20}{53.55} \right)^{\frac{1}{2}} \right]$$

$$= 44.1 MNm$$

The contribution of piles to the moment capacity is represented as:

$$M_{up} \approx \sum_{i=1}^{n_p} P_{uui} \left| x_i \right| \tag{2.3}$$

Where P_{uui}: ultimate uplift capacity of typical pile

$|x_i|$: absolute distance of pile i from center of gravity of group

n_p : number of piles

$$M_{up} = 1.20 \times (3 \times 4 + 3 \times 4 + 3 \times 0)$$

$$= 28.8 MNm$$

$$M_T = M_{ur} + M_{up}$$

$$= 44.1 + 28.8$$

$$= 72.9 MNm$$

b) The moment capacity of the block is given by Poulos and Davis (1980) as:

$$\mathrm{M}_{\mathrm{uB}} = \alpha_{\mathrm{B}}\ \mathrm{p}_{\mathrm{u}}\ \mathrm{B}_{\mathrm{B}}\ \mathrm{D}_{\mathrm{B}}^{\ 2} \tag{2.4}$$

Where B_{B} = width of bloc perpendicular to direction of loading

D_B = depth of block

P_u = average ultimate lateral resistance of soil along block

α_{B} = factor depending on distribution of ultimate lateral pressure with depth

- 0.25 for constant p_u with depth
- 0.2 for linearly increasing p_u with depth from zero at the surface

The average ultimate lateral resistance of soil along block P_u is:

$$P_u = N_c \times c_u \tag{2.5}$$

Where N_c: a lateral capacity factor

c_u: undrained shear strength.

For the block, the length is 2.5 times the width, so that the average ultimate lateral pressure along the block, p_u, is approximately:

$$P_u = 4.5 \times 0.1$$
$$= 0.45\text{MPa}$$

$$M_{uB} = 0.25 \times 0.45 \times 6 \times 15^2$$
$$= 151.9\text{MNm} > 72.9\text{MNm}$$

Therefore the factor of safety for moment loading is 72.9/25=2.92, which also satisfies the design criterion.

2.2.1.3. Lateral load capacity

The lateral load capacity is computed using the method given by Broms (1964) assuming that the pile heads considered as fixed. The lateral load capacity is calculated with two assumptions; short pile and long pile. The differences between short – piles and long – piles are given in Table 2.1.

Table 2.1 Failure Modes of Vertical Piles under Lateral Loads (Broms (1964a))

Pile type	Soil modulus	
	Linearly increasing	constant
Short (rigid) piles	$L \leq 2T$	$L \leq 2R$
Long (flexible) piles	$L \geq 4T$	$L \geq 3.5R$

- For constant soil modulus with depth (e.g. stiff over consolidated clay), pile stiffness factor is:

$$R = \sqrt[4]{\frac{E_p I_p}{k_h}} \tag{2.6}$$

Where R : pile stiffness factor for constant soil modules with depth (in units of length)

$E_p I_p$: bending stiffness of the pile

D : width of the pile

kh : coefficient of horizontal subgrade reaction

- For soil modulus increases linearly with depth (e.g. normally consolidated clay & granular soils), pile stiffness factor:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \tag{2.7}$$

Where T : pile stiffness factor for linearly increasing soil modules with depth (in units of length)

$E_p I_p$: bending stiffness of the pile

b : width of the pile

N_h : horizontal subgrade reaction constant

a) According to the short pile failure the lateral resistance of the soil up to 1.5b depth is assumed to be zero in this assumption and beneath this level the lateral resistance of the soil is considered to be uniform and is 9cb (Figure 2.6).

$$\begin{aligned} P &= (L - 1.5b)9cb \\ &= (15 - 1.5 \times 0.6) \times 9 \times 0.1 \times 0.6 \\ &= 7.6MN \end{aligned}$$

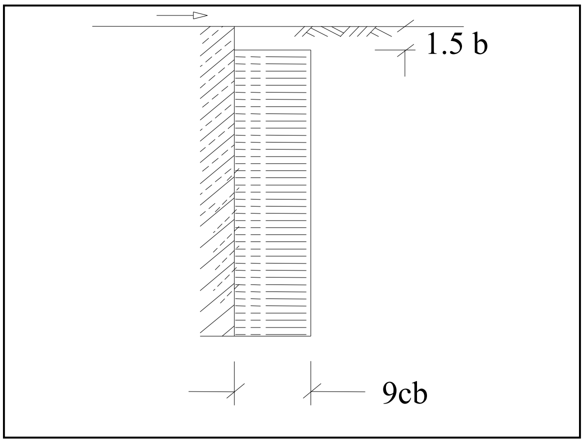

Figure 2.6 The acceptable ultimate soil stress

b) For long-pile failure, the moment capacity of a single pile has been calculated according to the Turkish regulations. 10ϕ18 ST III steel, and C30 concrete is used in calculations and the moment capacity is obtained as 0.43MN. The section of the calculation model is shown in Figure 2.7. Poulos (2000) has calculated the yield moment of the pile itself to be 0.45 MN.

Figure 2.8 gives the relationship between P_{ult} and M_y. $\frac{M_y}{cb^2} = \frac{0.45}{0.1 \times 0.6^2} = 20.8$

When we intersect this with the curve for the fixed head line, the corresponding point on the y axis is 17. P_{ult} = 17 x 0.1 x 0.6^2 = 0.61 MN. For nine piles, the total lateral load capacity is 5.49 MN. This value is found to be less than the corresponding value for the block. Thus, the factor of safety against lateral failure is 5.49/2.0=2.74, which satisfies the design criterion.

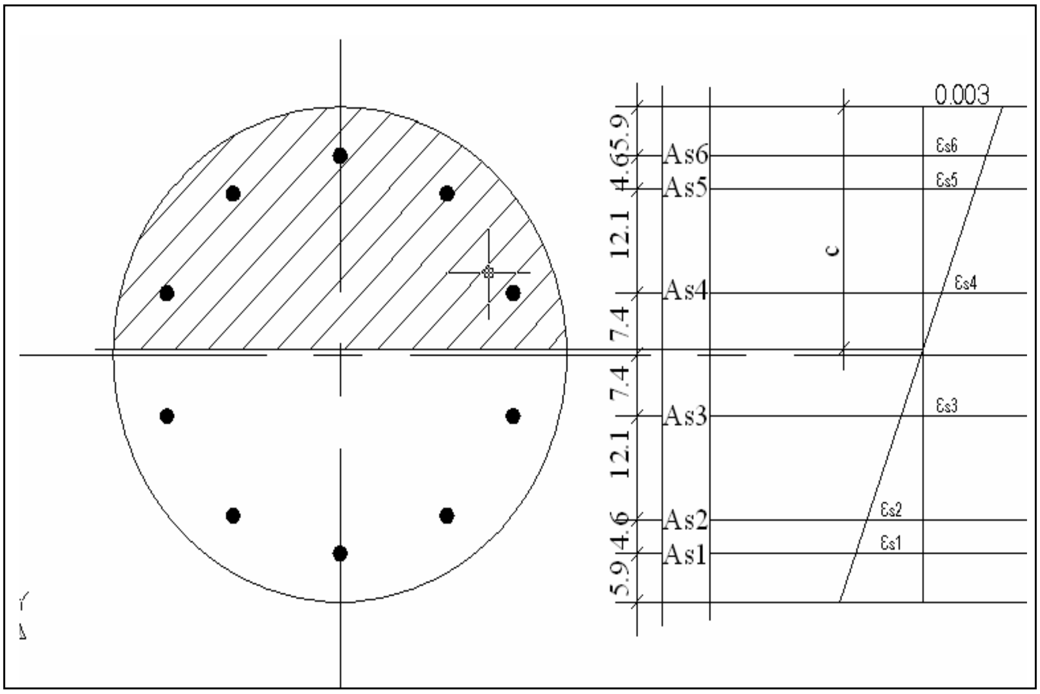

Figure 2.7 The section of the pile model.

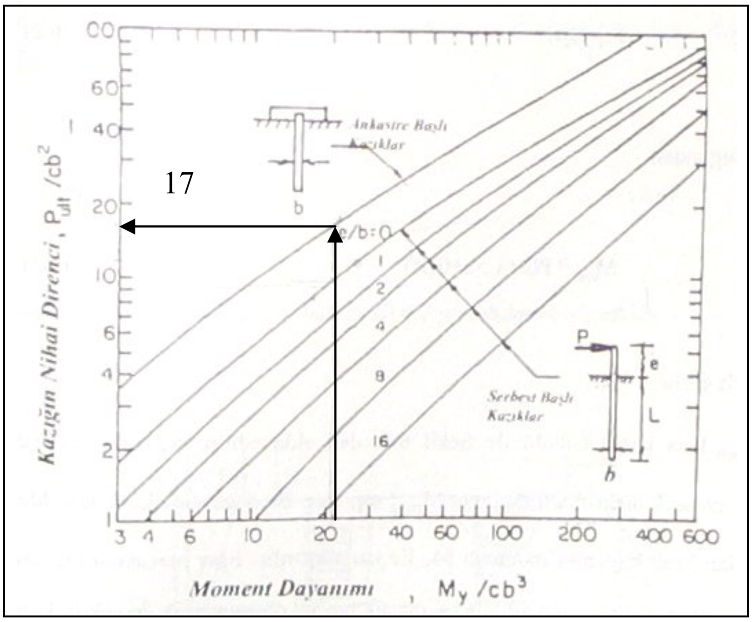

Figure 2.8. The moment capacity (Birand, 2001)

2.2.2 Estimation of load settlement behavior

The following aspects are included in the below formulations about settlement calculations.

- Estimation of load sharing between the raft and the piles, using the approximate solution of Randolph (1994).
- Hyperbolic load deflection relationships for the piles and for the raft, thus providing a more realistic overall load-settlement response for the piled raft system than the original tri-linear approach of Poulos & Davis (1980)

The piled – raft settlement relationship is shown in Figure 2.9. The point A represents the point at which the pile capacity is fully mobilized, when the total vertical applied load is V_A. Up to that point, piles and the raft share the load. The settlement S is:

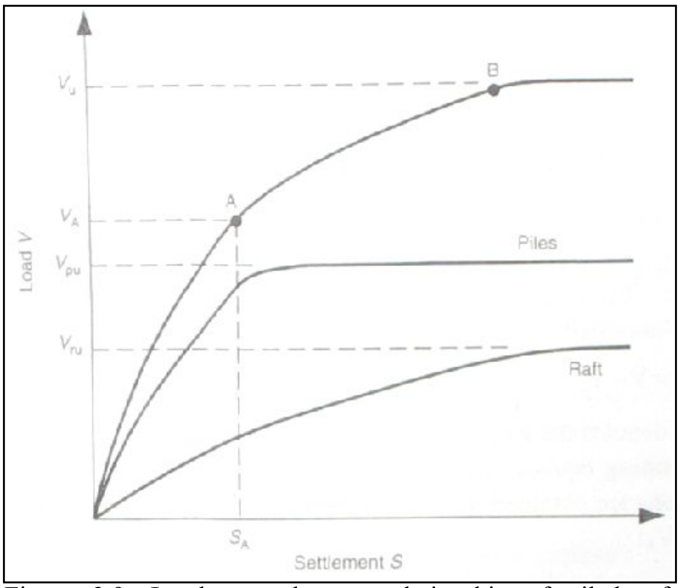

Figure 2.9. Load – settlement relationship of piled raft foundations (Poulos,2000)

$$S = \frac{V}{K_{pr}} \tag{2.8}$$

Where V = verticle applied load

K_{pr} = axial stiffness of piled raft system

Beyond point A, additional load must be carried by the raft.

$$S = \frac{V_A}{K_{pr}} + \frac{V - V_A}{K_r} \tag{2.9}$$

Where V_A = applied load at which pile capacity is mobilized

K_r = axial stiffness of raft

$$V_A = \frac{V_{pu}}{\beta_p} \tag{2.10}$$

Where V_{pu} = ultimate capacity of piles (single pile or block failure whichever is less)

β_p = proportion of load carried by piles

$$K_{pr} = XK_p \tag{2.11}$$

Where K_p : stiffness of pile group alone

X is defined with below equation:

$$\mathrm{X} \approx \frac{1\text{-}0.6\left(\mathrm{K_r}/K_p\right)}{1\text{-}0.64\left(\mathrm{K_r}/K_p\right)} \tag{2.12}$$

K_p denotes the stiffness of pile group alone and, for fairly large numbers of piles.

The average axial stiffness of the raft can be estimated from the elastic solutions reproduced by Poulos & Davis (1974). Stiffness is the force for unit displacement. Figure 2.10 gives a relationship between the raft geometry and the settlement by poisson ratio. The curves are for circular rafts so that, an equivalent circular raft with the same area is used. The radius a = 4.37m. “h” is the depth of bedrock. The settlement of the raft is:

$$\rho_z = \frac{P_{av}a}{E} I_p \tag{2.13}$$

Where I_p : Influence factor and can be obtained by using Figure 2.10

P_{av} : stress distribution

E : elasticity modules of the soil profile

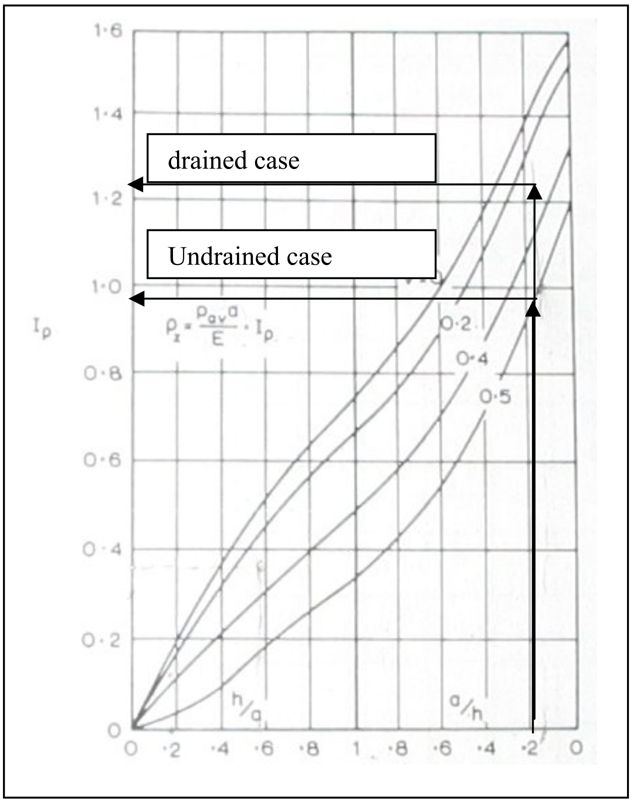

Figure 2.10. Influence factor for vertical displacement for rigid circle (Poulos&Davis 1974).

the axial stiffness of the raft for the effective (long term analysis) case ($\upsilon = 0.3$)

$$P_{av} = \frac{P}{\pi a^2} = \frac{15}{\pi \times 4.37^2} = 0.25$$

$$\frac{a}{h} = \frac{4.37}{25} = 0.19\text{........from Figure 2.10 } for\ \upsilon = 0.3\text{........} I_p = 1.22$$

$$\rho_z = \frac{0.25 \times 4.37}{15} \times 1.22$$

$$= 0.0888m$$

therefore the axial stiffness of the raft K_r is :

$$K_r = \frac{V}{\rho_z}$$

Where V : verticle load

$$K_r = \frac{15}{0.0888} = 169 MN/m$$

the axial stiffness of the raft for the undrained (short term analysis) case ($\upsilon = 0.5$)

$$P_{av} = \frac{P}{\pi a^2} = \frac{20}{\pi \times 4.37^2} = 0.333$$

$$\frac{a}{h} = \frac{4.37}{25} = 0.19 \text{........from Figure 2.10 } \textit{for } \upsilon = 0.5 \text{........} I_p = 0.98$$

$$\rho_z = \frac{0.25 \times 4.37}{20} \times 0.98$$
$$= 0.0476m$$

therefore the axial stiffness of the raft K_r is :

$$K_r = \frac{20}{0.04855} = 420 MN/m$$

Axial stiffness of piles is presented by Randolph and Wroth (1998):

$$k_p = \frac{P_t}{w_t} = \frac{2\pi}{\zeta} l G_1 \rho \frac{\tanh(\mu l)}{\mu l} \tag{2.14}$$

Where l : pile length

μ : a coefficient of the solution

G : Shear modulus

ρ : the ratio of the shear modulus at the pile mid - depth to that at the base, but 1 for constant G .

ζ : a factor of the distance that the shear stress influence diameter.

$$\zeta = \ln(r_m/r_0) \tag{2.15}$$

Where r_m : the distance at which the shear stres becomes negligible.

r_0 : pile radius

$$r_m = 2.5 l (1-\upsilon) \tag{2.16}$$

$$\mu l = \frac{l}{r_0}\sqrt{\frac{2}{\zeta\lambda}} \tag{2.17}$$

$$\lambda = \frac{E_p}{G} \tag{2.18}$$

$$G = \frac{E}{2(1+\upsilon)} \tag{2.19}$$

<u>the axial stiffness of single pile for the effective (long term analysis) case ($\upsilon = 0.3$)</u>

$$G = \frac{15}{2(1+0.3)} = 5.77MPa$$

$$\rho = 1$$

$$\lambda = \frac{30250}{5.77} \text{.........................} E = 30250MPa \text{ for cocrete class } C25 \text{ (Ersoy,1985)}$$

$$= 5242$$

$$r_m = 2.5 \times 15 \times (1-0.3)$$

$$= 26.25$$

$$r_0 = 0.3$$

$$\zeta = \ln(26.25/0.3)$$

$$= 4.47$$

$$\mu l = \frac{15}{0.3}\sqrt{\frac{2}{4.47 \times 5242}}$$

$$= 0.46$$

From Figure 2.11 $\dfrac{\tanh(\mu l)}{\mu l}$ is obtained as 0.93

$$K_p = \left(\frac{2 \times \pi}{4.77}\right) \times 15 \times 5.77 \times 1 \times 0.93$$

$$= 106$$

<u>the axial stiffness of single pile for the undrained (short term analysis) case ($\upsilon = 0.5$)</u>

$$G = \frac{30}{2(1+0.5)} = 10MPa$$

$$\rho = 1$$

$$\lambda = \frac{30250}{10} \text{.........................} E = 30250MPa \text{ for } C25 \text{ class concrete (Ersoy,1985)}$$

$$= 3025$$

$$r_m = 2.5 \times 15 \times (1-0.5)$$

$$= 18.75$$

$$r_0 = 0.3$$

$$\zeta = \ln(18.75/0.3)$$

$$= 4.13$$

$$\mu l = \frac{15}{0.3}\sqrt{\frac{2}{4.13 \times 3025}}$$
$$= 0.63$$

From Figure 2.11 $\frac{\tanh(\mu l)}{\mu l}$ is obtained as 0.90

$$K_p = \left(\frac{2 \times \pi}{4.13}\right) \times 15 \times 10 \times 1 \times 0.89$$
$$= 203 MN/m$$

Poulos (2000) gives single pile stiffness values of 122 MN/m and 217 MN/m for the drained and undrained cases, respectively. Assuming that the group factor is approximated as $\sqrt{n_p}$ (where n_p is the number of piles), the following initial pile group stiffness are obtained.

- undrained case; K_{pi} =651MN/m
- drained case ; K_{pi} =366MN/m

For undrined case $\quad X = \frac{1-0.6(420/651)}{1-0.64(420/651)} = 1.044$

K_{ue} $= 1.044 \times 651$ = 680 MN/m

For drined case $\quad X = \frac{1-0.6(169/366)}{1-0.64(169/366)} = 1.026$

K_{ue} =1.026x366 = 375 MN/m

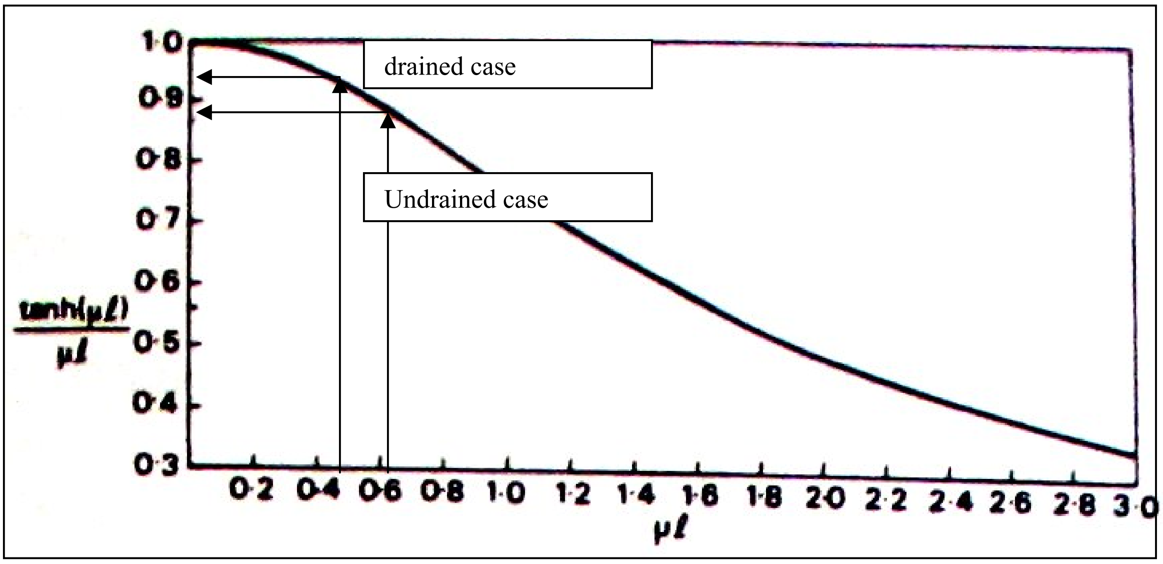

Figure 2.11. The variation of tanh(μ l)/(μ l) with μ l (Randolph&Wroth, 1978)

Proportion of load carried by the piles

$$\beta_p = 1/(1+\alpha) \tag{2.20}$$

$$\alpha = \frac{0.2}{1-0.8\left(K_r/K_p\right)}\left(\frac{K_r}{K_p}\right) \tag{2.21}$$

$\underline{\text{For undrained conditions:}}$

$$\alpha = \frac{0.2}{1-0.8(420/651)}\left(\frac{420}{651}\right)$$
$$= 0.267$$
$$\beta_p = \frac{1}{1+0.267}$$
$$= 0.79$$

$\underline{\text{For drained conditions:}}$

$$\alpha = \frac{0.2}{1-0.8(169/366)}\left(\frac{169}{366}\right)$$
$$= 0.146$$
$$\beta_p = \frac{1}{1+0.146}$$
$$= 0.87$$

If it is assumed that the pile and raft load settlement relationships are hyperbolic, then the secant stiffness of the piles (K_p) and the raft (K_p) are expressed as:

$$K_p = K_{pi}\left(1 - R_{fp}V_p / V_{pu}\right) \tag{2.23}$$

$$K_r = K_{ri}\left(1 - R_{fr}V_r / V_{ru}\right) \tag{2.24}$$

Where K_{pi} = initial tangent stiffness of pile group

R_{fp} = hyperbolic factor for pile group

V_p = load carried by piles

V_{pu} = ultimate capacity of piles

K_{ri} = initial tangent stiffness of raft

R_{fr} = hyperbolic factor for raft

V_r = load carried by raft

V_{ru} = ultimate capacity of raft

The hyperbolic factor is not a well-defined parameter and is really a fitting parameter to the single pile load-settlement curve. The value depends on the founding conditions of a pile and many other geometric and soil parameters. A factor of about 0.75 or so, as this seems to fit quite a number of load tests. So that the hyperbolic factors R_{fr}=0.75 and R_{fp}=0.5 for each applied load, β_p and X from the previous load are used, starting with the initial values first. The calculation of load settlement curve for piled raft foundation in worked example is given in Table 2.3.

Table 2.3. Calculation of load settlement curve for piled raft foundation in worked example (undrained case)

Vru (MN)	36		Vpu (MN)	17.55						
V	Kr (MN/m)	Kp (MN/m)	X	β_p	Vp (MN)	Vr (MN)	V_A (MN)	Kpr (MN/m)	S (mm)	V>VA
0	420.0	651.0	1.044	0.789	0	0	22.2	679.6	0.0	NO
5	410.8	577.8	1.044	0.789	3.95	1.05	22.2	603.2	8.3	NO
10	398.3	511.5	1.052	0.752	7.52	2.48	23.3	538.2	18.6	NO
15	381.6	454.1	1.062	0.708	10.62	4.38	24.8	482.3	31.1	NO
20	360.7	405.8	1.073	0.661	13.22	6.78	26.6	435.3	45.9	NO
25	336.7	363.9	1.082	0.619	15.48	9.52	28.3	393.9	63.5	NO
30	310.8	326.1	1.091	0.584	17.52	12.48	28.3	355.6	84.4	YES
35	267.1	326.1	-	-	17.52	17.48	28.3	355.6	104.7	YES
40	223.3	326.1	-	-	17.52	22.48	28.3	355.6	132.0	YES
45	179.6	326.1	-	-	17.52	27.48	28.3	355.6	172.6	YES
50	135.8	326.1	-	-	17.52	32.48	28.3	355.6	239.4	YES
52	118.3	326.1	-	-	17.52	34.48	28.3	355.6	279.9	YES

At long term design load of 15MN, the calculated immediate settlement is 31mm.

The final consolidation settlement (S_{CF}) is computed as the difference between the total final and immediate settlements from purely elastic analysis by Poulos (2000).

$$S_{cf} = V\left(\frac{1}{K'_e} - \frac{1}{K_{ue}}\right) \tag{2.25}$$
$$= 15\left(\frac{1}{375} - \frac{1}{680}\right)$$
$$= 17.9mm$$

The total final settlement is 0.0311 + 0.0179 =0.0490m (49mm)<50mm satisfies the design criterion.

2.2.3 *Differential settlement*

The simplified method given by Horikhoshi & Randolph (1997) is used here. The assumption is made that the vertical load is uniformly distributed. The soil raft stiffness is:

$$K_{rs} = 5.57\frac{E_r}{E_s}\frac{(1-v_s^2)}{(1-v_r^2)}\left(\frac{B}{L}\right)^{1/2}\left(\frac{t}{L}\right)^3 \tag{2.26}$$
$$= 5.57\frac{30000}{15}\frac{(1-0.3^2)}{(1-0.2^2)}\left(\frac{6}{10}\right)^{1/2}\left(\frac{0.5}{10}\right)^3$$
$$= 1.022$$

From the above reference, the ratio of the maximum differential settlement to the average settlement is 0.22 (Figure 2.12). Assuming that this ratio applies also to the piled raft, the maximum long term differential settlement (center to corner) is 0.22x0.049= 0.011m. This exceeds the specified value of 10mm, and it is found that the raft thickness needs to be increase slightly to 0.52m.

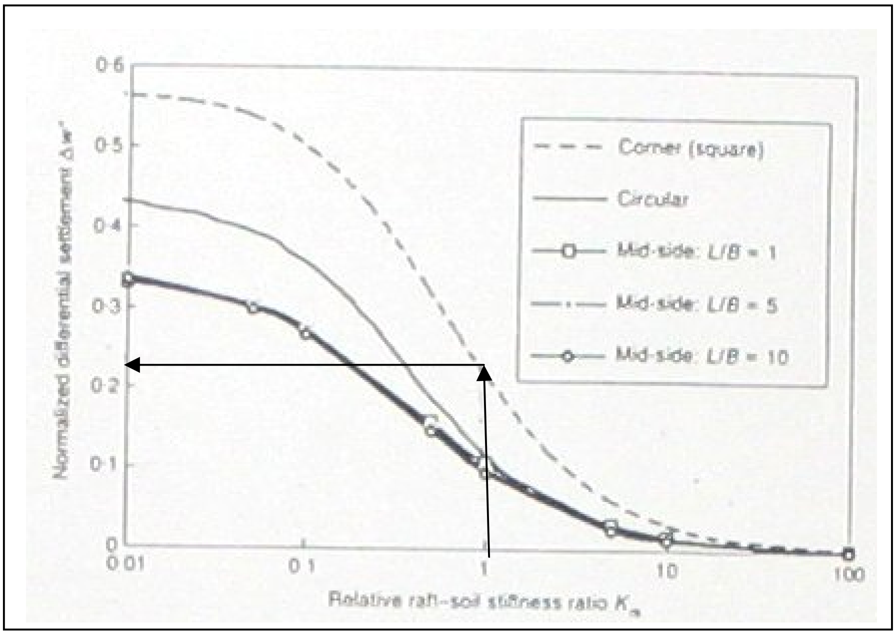

Figure 2.12 Variation of normilized differential settlement with raft - soil stiffness ratio K_{rs} (Horikoshi & Randolph, 1997)

2.2.4 Estimation of pile loads

A first estimate of the axial forces in the piles can be made using an adaptation of rivet group approach. If the piles carry a portion of β_p of the total vertical load, then the axial force "P_i" in any pile "i" in the foundation system can be estimated from:

$$P_i = \frac{V\beta_P}{n_p} + \frac{M_x^{*} x_i}{I_y} + \frac{M_y^{*} y_i}{I_x} \qquad (2.27)$$

$M_y^* = 0$..There is no M_y moment in this direction.

$M_x^* = M_x$..The piles are symmetric.

$$P_{\max,\min} = 20 \times \frac{0.661}{9} \pm 25 \times \frac{4}{96}$$

$$= 1.47 \pm 1.04$$

$$P_{\max} = 2.51 MN$$

$$P_{\min} = 0.43 MN$$

The ultimate capacity of a single pile is 1.95MN and because of this the capacity of the outer piles are fully utilized. Piles must be structurally designed to carry the maximum load.

2.2.5 Estimation of raft bending moments and shears

In the last part of the piled raft design it is needed to estimate the raft bending moments. Poulos (2000) uses the simple static for calculations. Loadings are assumed to be uniform loading and the long term case is assumed. The applied load is V = 15MN, the foundation area is A = 60m^2. The stress distribution on the raft is q_a=15 / 60 = 0.250 MPa. Proportion of the load carried by raft is β_r=0.13 and average contact pressure of the raft is q_r = 0.250 x 0.13 = 0.0325MPa. Therefore the net stress distribution is q_{net} = 0.250 – 0.0325 = 0.2175MPa. In Figure 2.13 P1 and P2 represent piles carrying the same loads. Raft bending moments are calculated for both x and y directions by dividing raft in strips. Calculations of bending moments are given in Figure 2.14 and to be 0.326 MNm/m in x direction and zero in y direction. Maximum negative bending moments are – 0.109MNm/m in both directions.

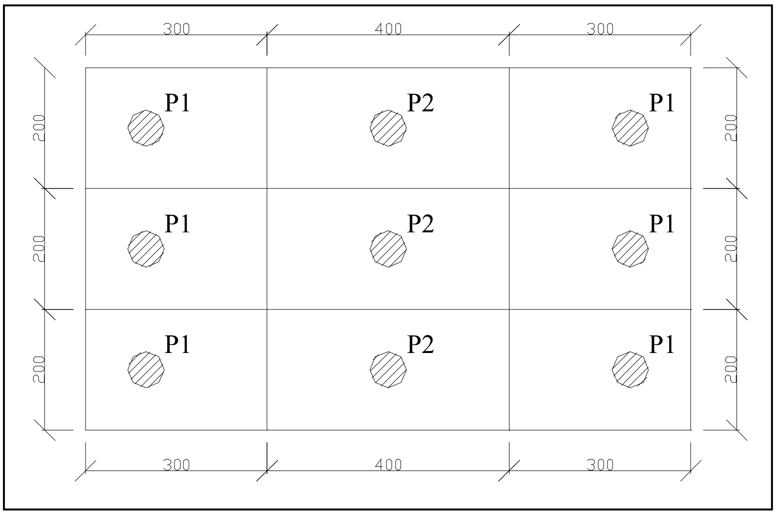

Figure 2.13 Load distributions on each pile

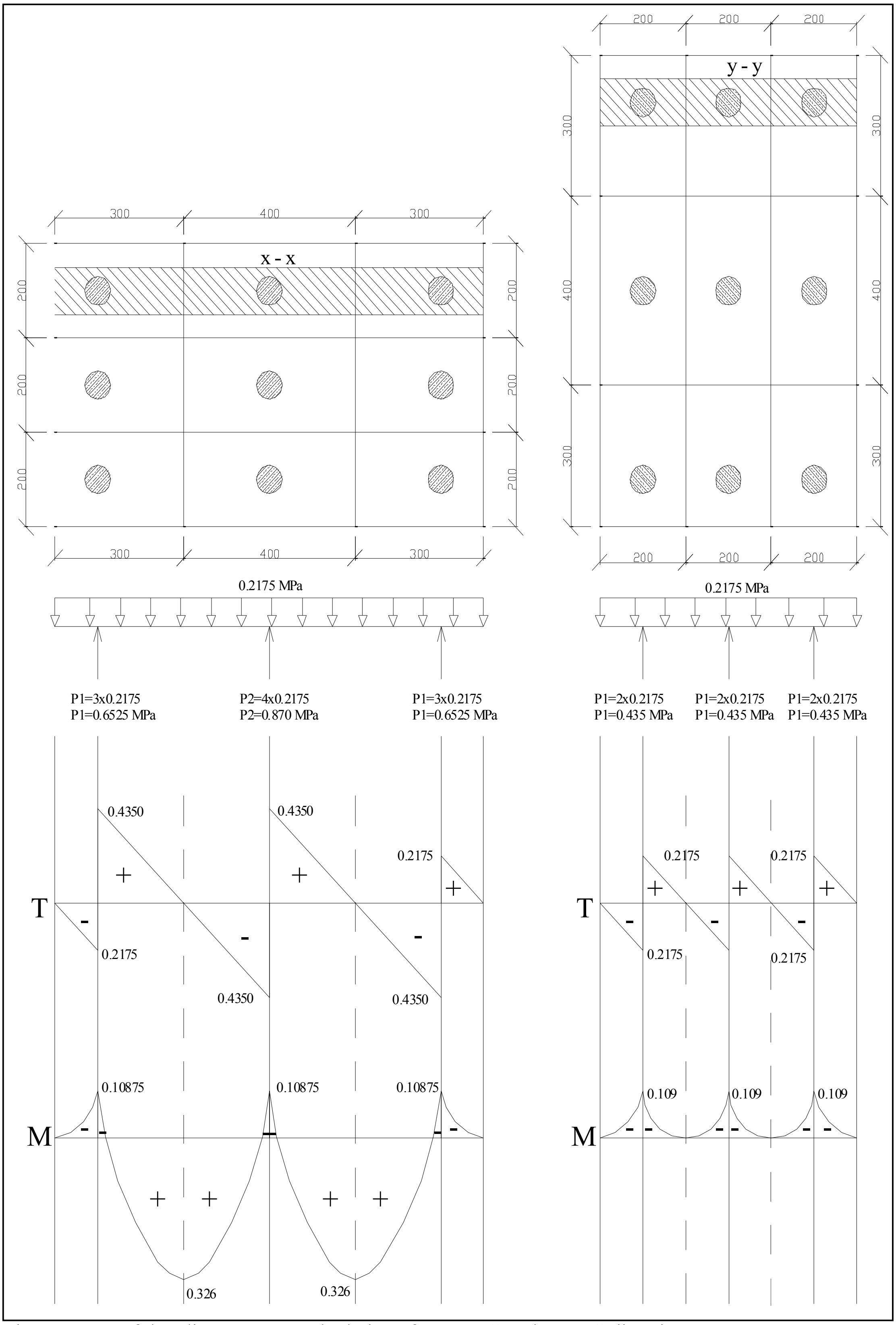

Figure 2.14 Raft bending moment calculations for "x – x" and "y – y" directions.

2.3 Finite Element Analysis

It has mentioned that piled raft foundations are complicated problems and have to be designed by using appropriate computer programs. A tree dimensional finite element computer program Plaxis 3D Foundation Version 1.1 is used for the computer based analysis. Analyses are generated by using the values given in Table 2.4, and the raft thickness is one meter. The rest of the finite element model is same as the worked example. The stage construction is used to define construction method.

Table 2.4 Soil Properties

E	υ	c	Ø	γ_{unsat}	γ_{sat}
(kN/m^2)	-	(kN/m^2)	(degree)	(kN/m^3)	(kN/m^3)
15000	0.3	4	30	18	20

2.3.1 Evaluations of the output files

The three dimensional soil and the piled raft foundation is modeled and the finite element analysis is performed and the maximum settlement is observed as 38.65 mm. The three dimensional deformed mash is shown in Figure 2.15. The settlement of the top surface is given in Figure 2.16 by means of shadings and the crossection is given in Figure 2.17. The raft settlements and bending moments are given in Figure 2.18 – 2.19, respectively. The pile loads are given in Figure 2.20.

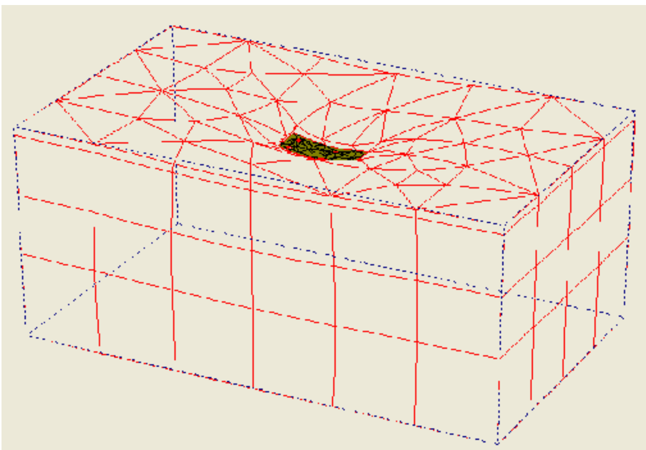

Figure 2.15. The three dimensional deformed mash .

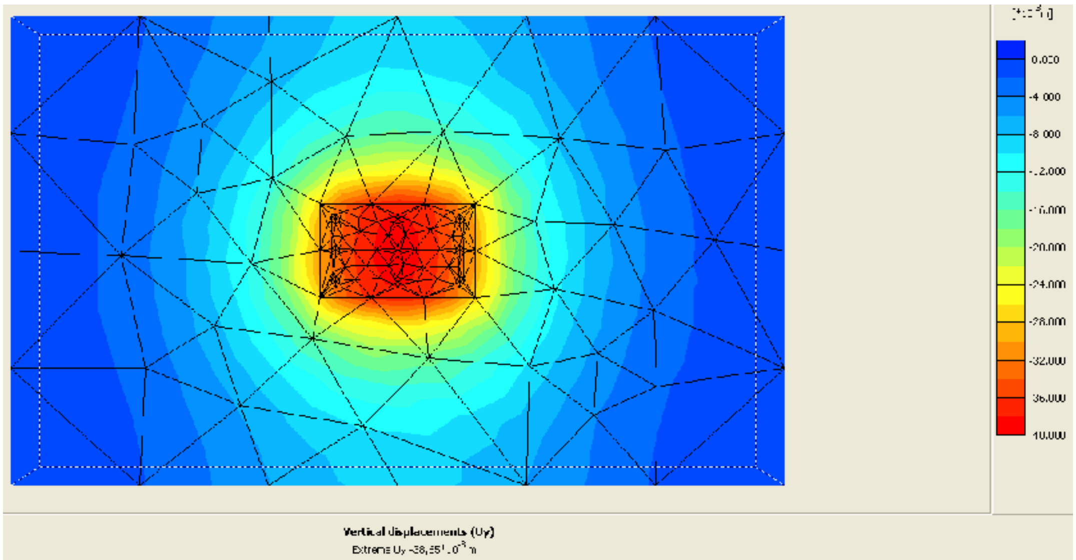

Figure 2.16 The settlement surface of the top surface.

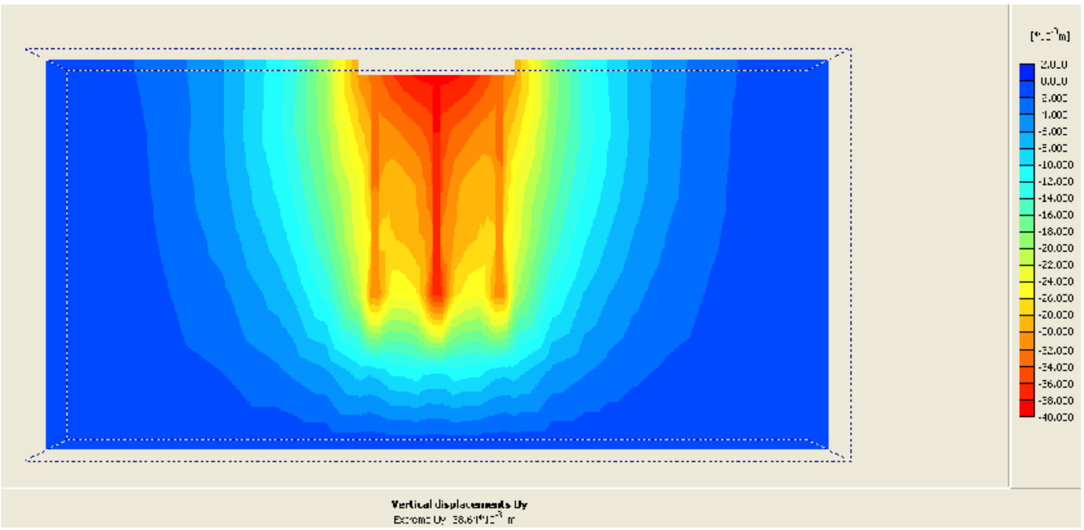

Figure 2.17 The vertical settlements on the crossection

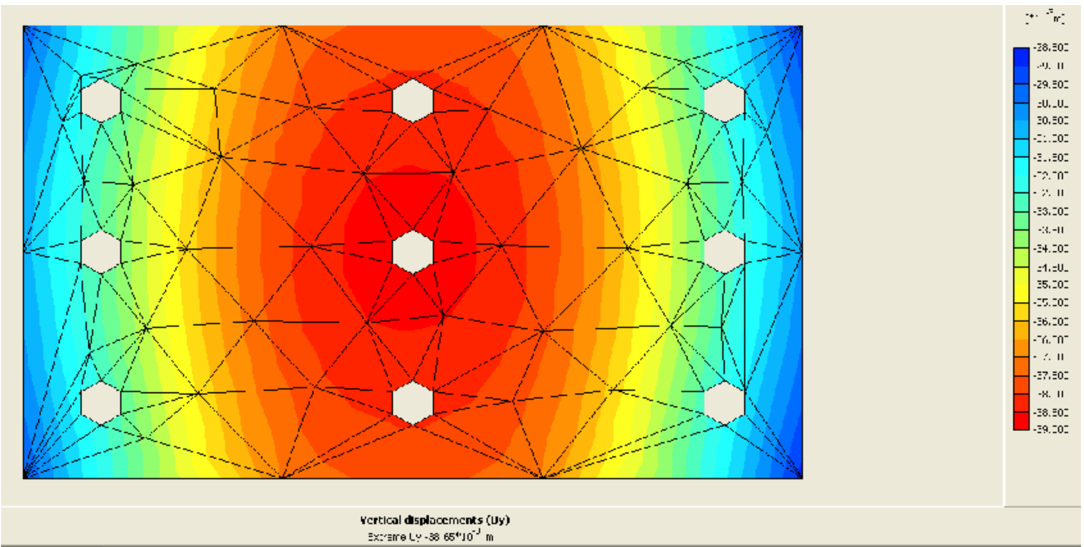

Figure 2.18 Raft settlements.

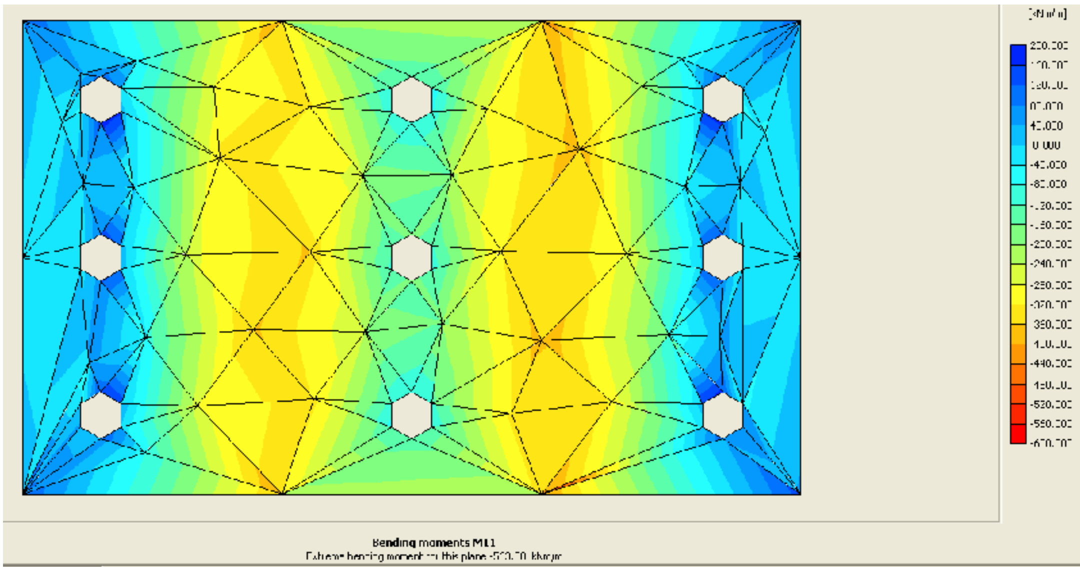

Figure 2.19 Raft bending moments.

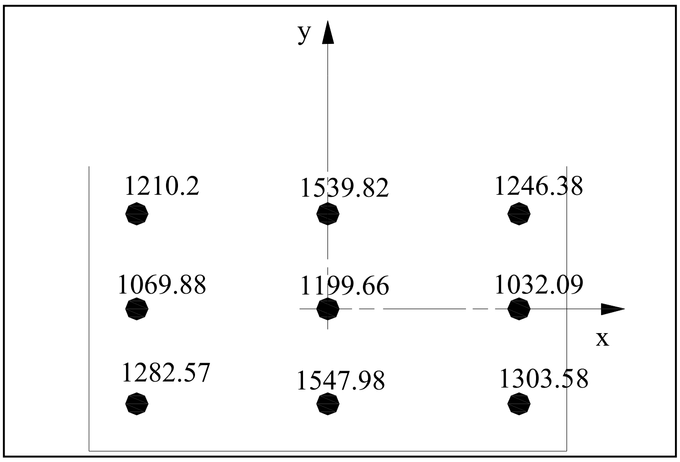

Figure 2.20 Pile loads

The total load carried by piles is 11.43MN and the proportion of load carried by piles β_P is 0.76. By using the hand calculation method it was calculated as 0.87. One of the reasons of the difference between two methods is the possible differences on estimating the soil properties. Although there is 9% percent of difference between two calculation methods, hand calculation method is logical for preliminary design.

CHAPTER THREE
THE PILED RAFT APPLICATION IN MAVİŞEHİR – İZMİR

3.1 Investigation site

Mavişehir has been chosen as the location for a piled raft application in İzmir. Mavişehir is at the North cost of İzmir bay. It is an old marshy area and it was used as garbage dump. But in recent years this region has became an important center of luxurious tall residences and shopping malls. Application site location is shown in Figure 3.1.

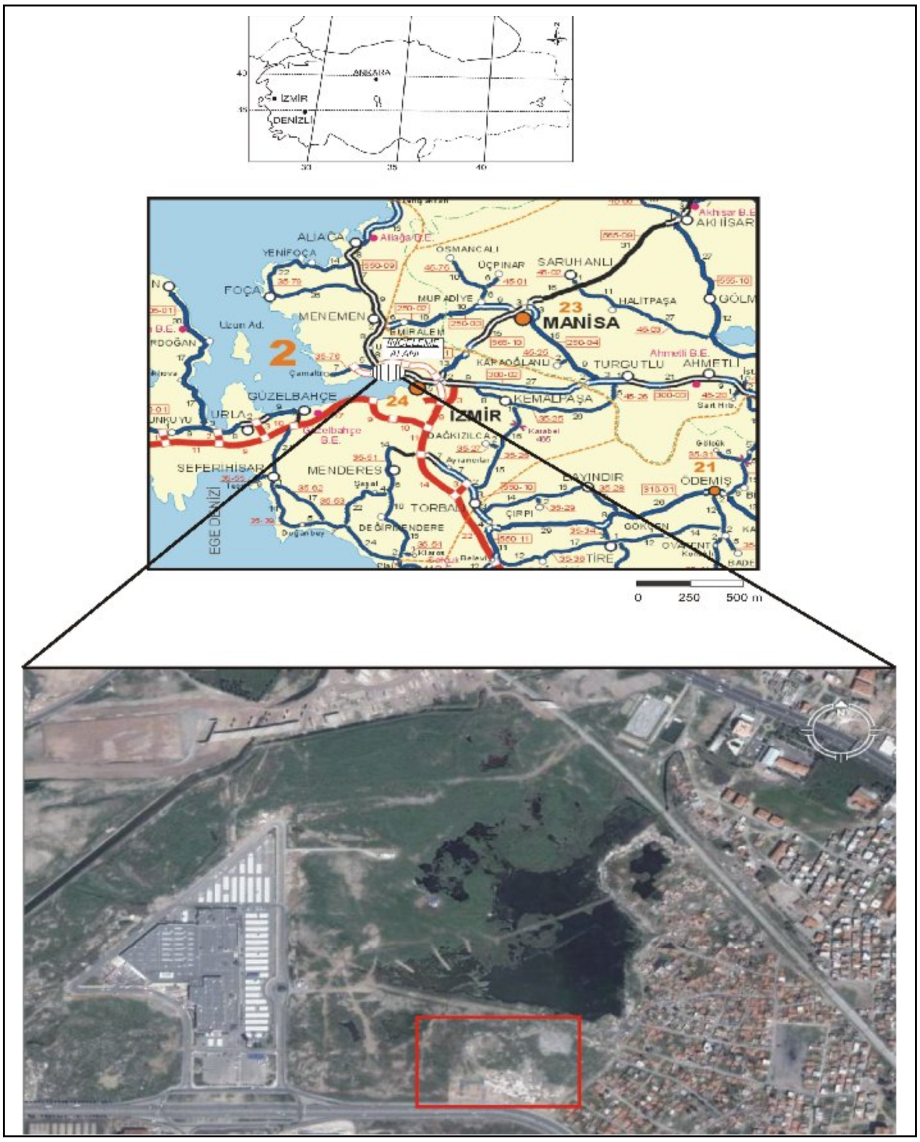

Figure 3.1 Investigation site locations

3.2 Soil model

A soil investigation report was prepa29 n April 2007 by Ege Temel Sondajcılık San. ve Tic. Ltd. Şti. for the investigation site. All of the fallowing analyses and evaluations are based on the data given in this report. In the first step of field studies approximately 40m depth 23 boreholes were opened. In all boreholes SPT (standard penetration test) was performed and SPT, core and UD samples were collected for laboratory tests. Collected samples were used for soil classifications. UD samples were also used for consolidation tests and UU triaxial tests. Furthermore, 9 CPT (cone penetration test) were performed as part of the soil investigation studies.

After the preparation of the report 3 additional boreholes were added. Two of the boreholes were 60m depth and one was 120m depth. Again SPT's were done for these boreholes, but in the last borehole SPT tests were performed up to 80m depth. Soil data has been used to model a representative soil profile of the investigation site. The application plan of the in-situ tests related structures are given in Appendix A.

3.2.1 SPT (Standard Penetration Test)

The SPT is one of the most popular test methods of site investigations in Turkey as in most countries. SPT is used to estimate the relative density, strength and the liquefaction potential of granular soils and consistency limits and strength of cohesive soils. Several factors are used for the correction of N_{30}. The corrected SPT , N'_{60} and correction factors are:

$N'_{60} = N_{30} C_N C_E C_B C_R C_S$

C_N : Overburden pressure correction factor............only for granular soils

$C_N = 2.2 / \left(1.2 + \sigma'_{vo} / P_a\right)$

C_E : Energy ratio correction factor

$C_E = \dfrac{ER}{60}$..........$ER \approx 45$.........$C_E = 0.75$

C_B : Borehole diameter correction factor

$C_B = 1$........$d \approx 76mm$

C_R : Rod length correction factor

$C_R = 0.8$...........$l < 3m$

$C_R = 0.85$.........$l \simeq 3 - 4m$

$C_R = 0.95$.........$l \simeq 6 - 10m$

$C_R = 1.00$.........$l \simeq 10 - ....$

C_S : Sampling method correction factor

$C_S = 1.2$.............Sampler without liners

SPT corrections are given in Appendix B. N_{30} values of all SPT tests are plotted in Figure 3.4.

While preparing the soil model all boreholes were studied. Drawings of crossections of boreholes side by side are given in Appendix C. Elevations of boreholes have been taken into account, but there was no readily available elevation measurements of boreholes so that the in – situ test locations were applied on the elevation plan (Appendix A). The elevation of a test location has been interpolated from the nearest elevations on the elevation plan. It is clearly seen that there is a great similarity of soil layers between boreholes and so that it is possible to define an idealized soil profile to represent the whole site. Average thicknesses of soil layers have been used to define the thickness of the soil layers in the idealized soil profile. The idealized soil profile is given in Appendix C and the soil layers are as given below:

00.00 – 05.00	FILL
05.00 – 09.00	SAND 1
09.00 – 19.00	CLAY
19.00 – 21.50	SAND 2
21.50 – 34.00	GRAVELLY CLAY 1

34.00 – 40.00	GRAVEL
40.00 – 60.00	GRAVELLY CLAY 2

In gravelly layer between the -34.00 and -40.00m elevations it is observed that most of SPT were failed. The results are over 50. Also there are gravel layers in some boreholes at these depths and so that this interval is thought as gravel. The ground water level (gwl) is at 3.50m.

The SPT N_{30} data's plotted in Figure 3.4 are again plotted near the idealized soil profile to see the general resistance of the soil layers (Appendix – C). The cloud shaped data follows soil layers and the degree of the resistance of soil layers can be seen.

3.2.2 CPT (Cone Penetration Test)

CPT is a quick and simple in-situ test method and is used in soft clays, soft silts, and in fine to medium sand deposits. Collecting continuous data during test is its best feature, but it doesn't allow collecting samples. CPT equipment may be used with a drilling machine to pass the stiff layers.

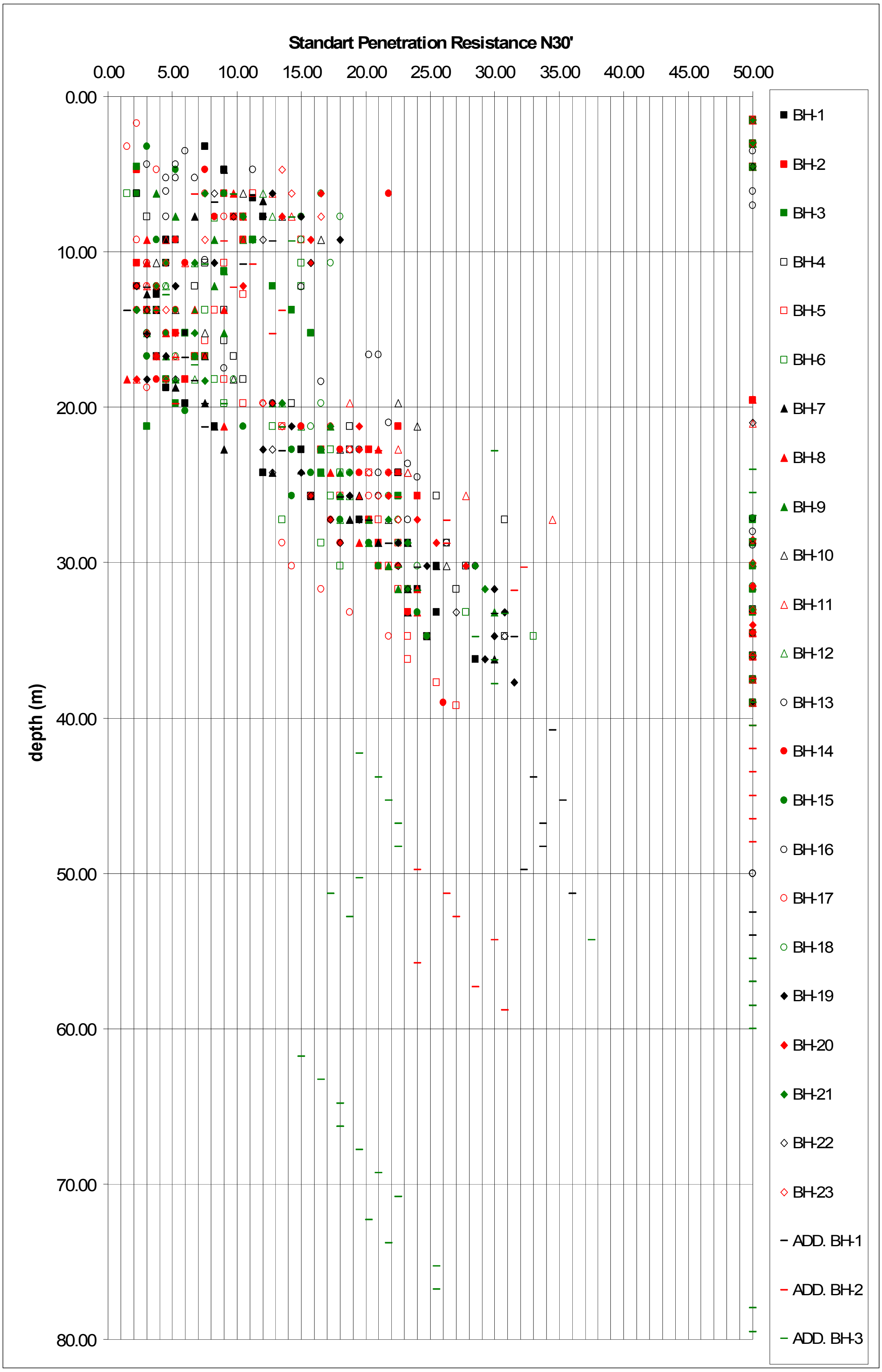

Figure 3.4 N_{30} values versus depth

In the investigation site it is observed that there is approximately five meter thick fill layer on the top surface. A drilling machine is used to pass this layer. 9 CPT were performed but only 7 of CPT could be used, because two of them were shallow. Other CPT's are up to 24m depth. CPT tip resistances are plotted together in Figure 3.5 with in-situ elevations. The soil profile from the CPT is seems to be similar to obtained from borings. Approximately first five meter of CPT is filling and about five to ten meters an obvious sand layer can be seen. The rest of the plotting shows a clay layer, although there are some small silt and sand layers. It is clearly seen that the tip resistance increases with depth. This is typicle behavior of NC clays. It is not possible to estimate the soil profile after depth of 23-24 meters. The average values of tip resistances (q_c) and skin resistances (q_s) of CPT are given in Table 3.1. High resistances of small silt and sand layers weren't taken into account while determining the average values of the clay layer.

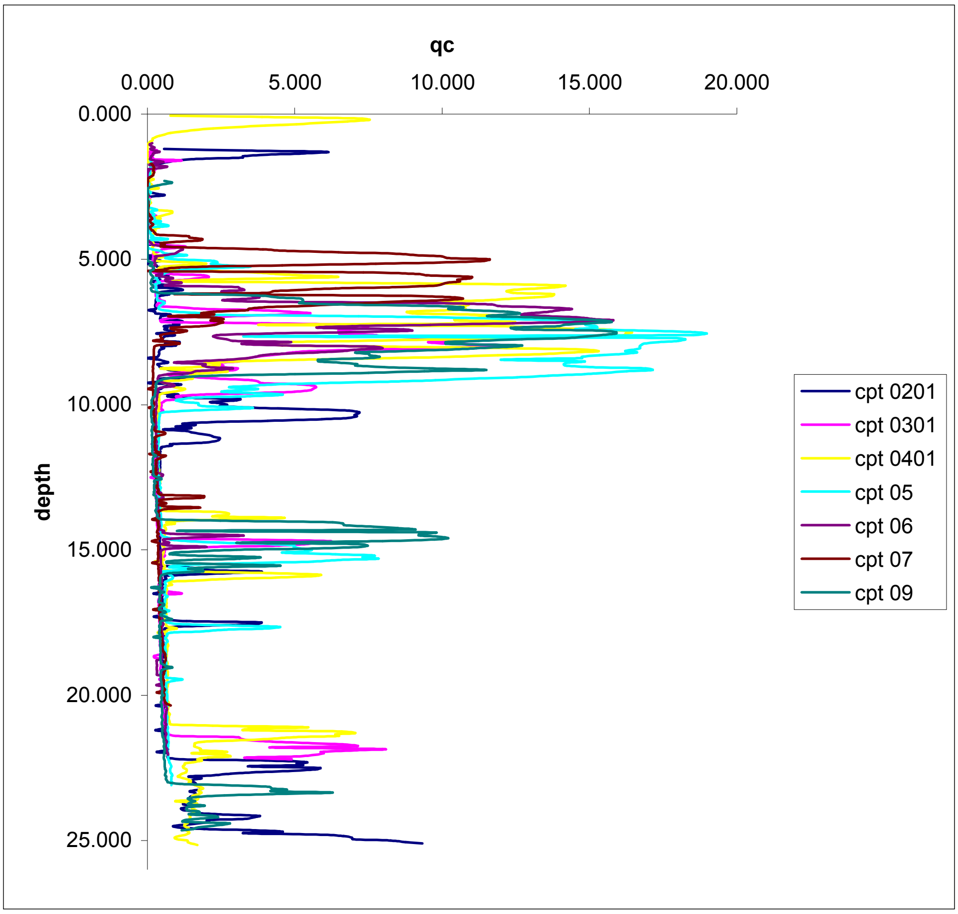

Figure 3.5 CPT tip resistances.

Table 3.1 Average CPT Parameters.

	depth (m)	q_s (kPa)	q_c (MPa)
sand	5.00 - 9.00	79.09	7.24
clay	9.00 - 19.00	6.66	0.46

3.2.3 FVST (Field Vane Shear Testing)

"The vane shear test (VST) is a substantially used method to estimate the in – situ undrained shear strength of very soft, sensitive, fine-grained soil deposits" (Bowles, 1996). During field studies vane shear test is performed with a tapper shaped vane equipment. Bowles, 1996 gives the undrained shear strength of tapper vane as:

$$s_{u,v} = \frac{0.3183T}{1.354d^3 + 0.354\left(d_1 d^2 - dd_1^2\right) + 0.2707d_1^3}$$

d : diameter of vane blades

d_1 : shaft diameter at vane

T : measured torque

The correction factor for undrained shear strength from vane test is given in Figure 3.6. Evaluation of vane test results are given in Appendix D. The average undrained shear strength of the clay layer (9.00-19.00m) has been found to be $s_{u,design}$= 23kPa.

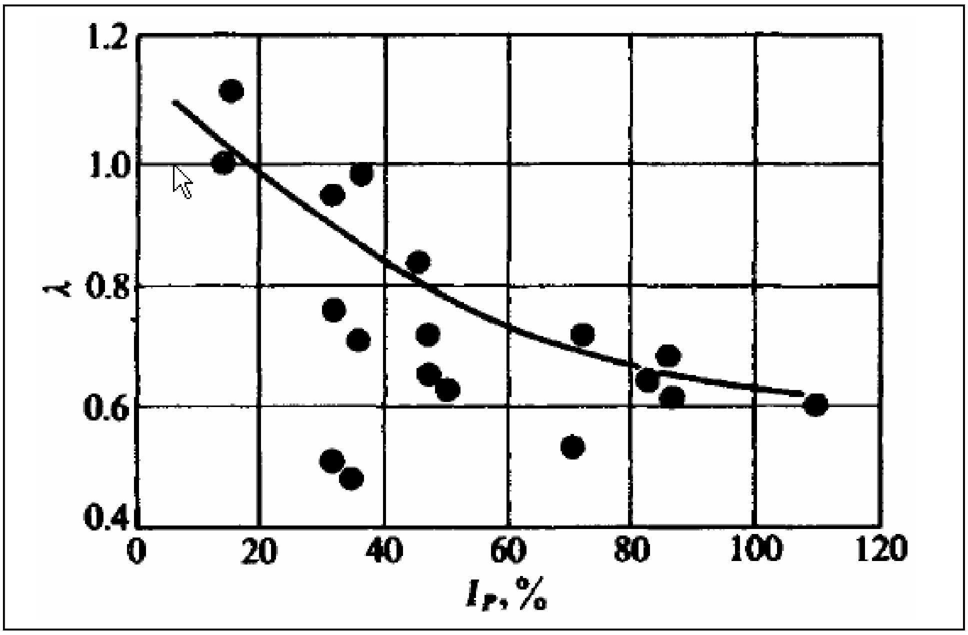

Figure 3.6 Bjerrum's correction factor λ for vane shear test (Bowles, 1996)

3.2.4 Laboratory tests

In field SPT, core and UD samples were collected for laboratory tests by Ege Temel Sondajcılık San. ve Tic. Ltd. Şti and laboratory tests were done by Ege Zemin Tic. ve Ltd. Şti. The laboratory has the certificates of TSE (The Institute of Turkish Standards) and The Ministry of Public Works and Settlement. Samples of the second step works were studied in the soil mechanics laboratory of Dokuz Eylül University. Soil classification tests were performed on the representative samples from SPT, core and UD samples. Also from the UD samples UU triaxial tests and consolidation tests were performed.

3.2.5 Soil Parameters

3.2.5.1 The Fill layer (0.00 – 5.00)

It has been mentioned that in recent years this region of the investigation site has became an important center of luxurious tall residences and shopping malls. Fill layer is an uncontrolled fill and consist of city garbage, excavation soils and wastes of nearby constructions. So it is not very well known that the consolidation settlement of clay layers has been completed or not because of this fill loading.

It is not possible to build any structure on this uncontrolled fill layer. The foundation depth of the subject matter building is approximately five meter and the building is going to stand on the sand layer.

3.2.5.2 The Sand layer (5.00 – 9.00)

The average thickness of the sand layer is four meter and the foundation of the structure lies on this layer. The average corrected SPT blow count of the sand layer is N'_{60} =12.5. The internal friction angle from SPT is:

$$\begin{aligned}\phi &= 0.36N'_{70} + 27 \ldots\ldots \mathit{Japanese\ Railway\ S}\tan \mathit{darts}\left(\mathit{Bowles}, 1996\right)\\ &= 0.36 \times 10.7 + 27\\ &= 30.9^\circ\end{aligned}$$

The internal friction angle from CPT is:

$$\begin{aligned}\phi &= 29 + \sqrt{q_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\left(\mathit{Bowles}, 1996\right)\\ &= 29 + \sqrt{7.24}\\ &= 31.7^\circ\end{aligned}$$

An average value for the internal friction angle has been selected as ϕ=32º.

The elasticity modulus, E_s, of the sand layer is estimated from the correlation based on the SPT test. For saturated sands E_s is:

$$\begin{aligned}E_s &= 250\left(N_{55} + 15\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\left(\mathit{Bowles}, 1996\right)\\ &= 250\left(13.6 + 15\right)\\ &= 7150\,kPa\end{aligned}$$

Although Bowles (1996) gives the information that this equation might give a value too small, this modulus value has been used for design for being on the safe side. The Poisson’s ratio υ is taken as 0.3 and the saturated unit weight of the sand is γ_d = 20 kN/m^3.

3.2.5.3 The Clay layer (9.00 – 19.00)

This layer of the investigation site mostly consists of CH clays. There are many different laboratory and in-situ tests performed on this layer. To estimate the undrained shear strength first the laboratory test were studied.

For saturated normally consolidated clays undrained shear strength can be obtained from unconfined compression test as:

$s_u = q_u / 2 \ (\phi = 0\, state)$.................(*Bowles*,1996)

The undrained shear strength, s_u, calculations of the unconfined compression tests are given in Table 3.2. The average undrained shear strength is s_u =12.5 kPa.

Table 3.2 Shear strength of clay layer from unconfined compression test

bore hole number	sample	USCS	q_u (kPa)	s_u (kPa)
BH-3	UD (18.00 - 18.45)	CH	29	15
BH-7	UD (12.00 - 12.50)	CH	24	12
BH-15	UD (19.50 - 20.00)	MH	22	11
BH-18	UD (17.10 - 17.60)	CH	32	16
BH-20	UD (15.50 - 16.00)	CH	28	14
BH-23	UD (13.00 - 13.50)	CH	13	7

Undrained shear strength of the clay layer from the UU triaxial tests are given in Table 3.3. The average undrained shear strength is c_u = 26.3 kPa.

Table 3.3 Shear strength of clay layer from UU three axial test

bore hole number	sample	c (kPa)
BH-1	UD (12.00 - 12.50)	20
BH-5	UD (15.50 - 16.00)	32
BH-9	UD (10.50 - 11.00)	22
BH-17	UD (18.00 - 18.50)	31

The average undrained shear strength from the vane shear test has been calculated in Section 3.2.3 and is $s_{u,design}$= 23kPa.

Also the undrained shear strength can be calculated from the CPT. The correlation is based on the tip resistance.

$$s_u = \frac{q_c - \sigma_v}{N_k} .... (Bowles, 1996)$$

q_c : tip resistance

σ_v : overburden pressure

N_k : cone factor

$$S_t \approx \frac{10}{f_r}$$

S_t : soil sensitivity

f_r : friction ratio (percentage)

$$f_r = \frac{q_s}{q_c} \times 100 = \frac{6.66}{460} \times 100 = 1.45 \Rightarrow S_t = 6.90$$

The Atterberg limits of the clay layer are obtained by averaging the Atterberg limits of UD samples given below:

$$BH01\left(UD\langle MH\rangle : 12^{.00} - 12^{.50}\right)......... wl = 58; wp = 31; Ip = 27$$

$$\left.\begin{array}{l} BH05\left(UD\langle CH\rangle : 15^{.50} - 15^{.95}\right)......... wl = 64; wp = 28; Ip = 36 \\ BH09\left(UD\langle CH\rangle : 10^{.50} - 11^{.00}\right)......... wl = 59; wp = 27; Ip = 32 \\ BH23\left(UD\langle CH\rangle : 13^{.00} - 13^{.50}\right)......... wl = 64; wp = 26; Ip = 38 \end{array}\right\} wl = 62; wp = 27; Ip = 35$$

N_k is 14.1 (Figure 3.7) and s_u is:

$$\sigma_v = \underbrace{3.5 \times 18 + 1.5 \times 20}_{fill} + \underbrace{4 \times 20}_{sand} + \underbrace{5 \times 17}_{clay}$$

$$= 258 kPa$$

$$s_u = \frac{460 - 258}{14.5}$$

$$= 14 kPa$$

All calculated undrained shear strength values with different test methods are given in Table 3.4. The smallest value is obtained from unconfined compression test, while UU triaxial test gives the largest value. The average value is 19kPa and this is used for design.

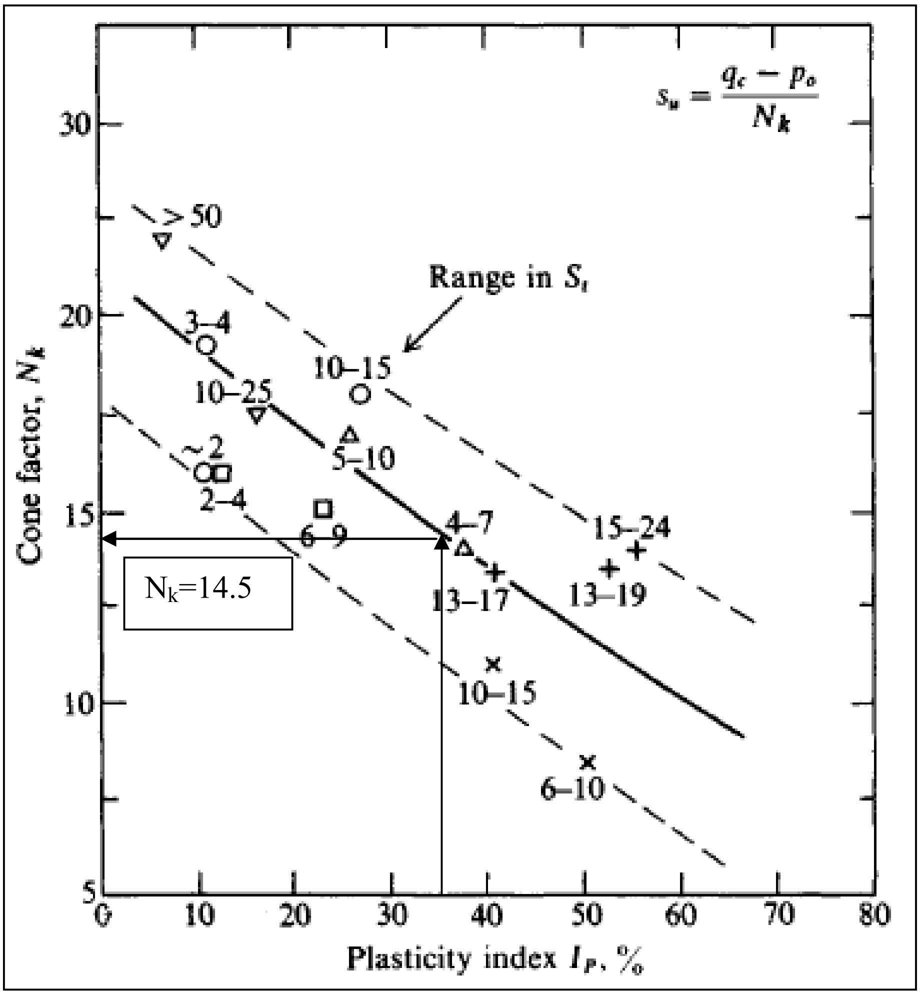

Figure 3.7 Cone factor N_k versus I_p plotted for several soils with range in sensitivity (Bowles, 1996)

Table 3.4 The undrained shear strength of the clay layer (9.00-19.00) obtained from different test methods

	Unconfined compression test	**UU triaxial test**	**FVST**	**CPT**
c_u (kPa)	12.5	26.3	23	14

The relation between elastic modulus (E_u) and undrained shear strength is expressed as (Das, 1997):

$$E_u = K_{cu} \times c_u$$

$$K_{cu} : factor\,relating\, E_u\, with\, c_u$$

K_{cu} can be obtained from Figure 3.8 the plasticity index is %35 for clay layer.

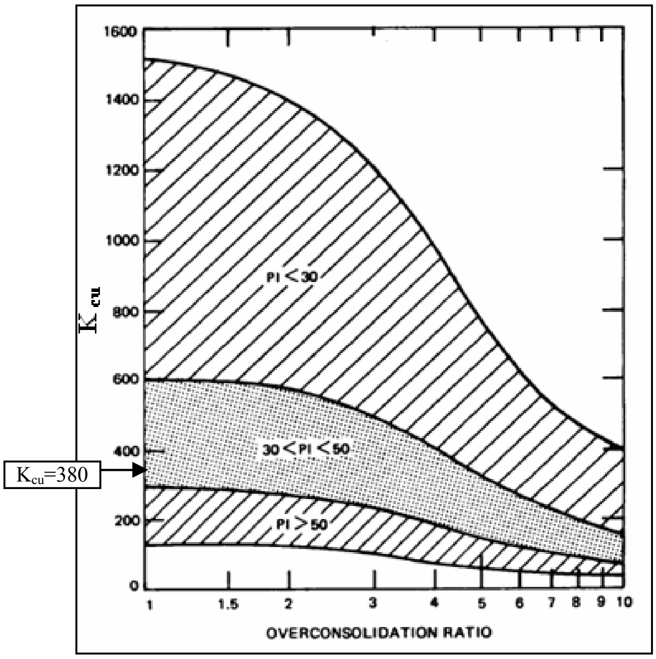

Figure 3.8 For estimating undrained modulus of clay (Das, 1997)

$$
\begin{aligned}
E_u &= 380 \times 19 \\
&= 7220 kPa
\end{aligned}
$$

The Poisson's ratio for the undrained case is υ_u=0.5. For most soils, the effective Poisson's ratio ranges between 0.12 and 0.35 (Wroth C.P. & Houldby G.T., 1985). The υ' is selected as 0.2. Wroth C.P. & Houldby G.T. (1985) suggests a relationship between drained and undrained elasticity modulus. The assumption is based on the idea of that shear modulus is same for both the undrained and effective cases. "For perfectly elastic soil, the value of the shear modulus is unaffected by the drainage condition, since the water within the soil skeleton has zero shear stiffness" (Kempfert H.G & Gebreselassie B., 2006).

$$
\begin{aligned}
&\frac{E_u}{2(1+\upsilon_u)} = G_u = G' = \frac{E'}{2(1+\upsilon')} \text{..........} \upsilon_u = 0.5\, then \\
&\frac{E_u}{E'} = \frac{3}{2(1+\upsilon')} \text{....................................} \upsilon' = 0.2 \\
&E' = \frac{E_u}{1.25} \\
&\quad = \frac{7220}{1.25} \\
&\quad = 5776 kPa
\end{aligned}
$$

The internal friction angle, Ø, is another important soil parameter for design. $Ø_u$ is thought to be zero, although in laboratory triaxial tests small values were obtained. The relationship between the effected Ø' and I_p is given in Figure 3.9. Ø' is obtained as 27°.

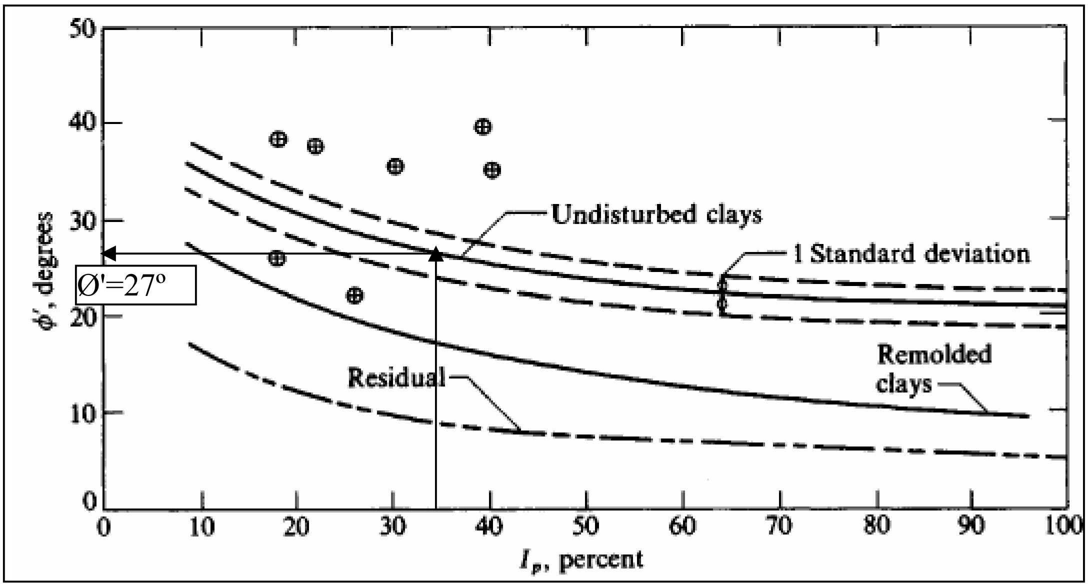

Figure 3.9 Correlation between Ø' and plasticity index I_p for normally consolidated (included marine) clays. Approximately 80 percent of data falls within one deviation. Only a few extreme values are shown (Bowles, 1996).

There are four consolidation test results given in the soil investigation report on clay layer. Test results are given in Table 3.5. It is thought that results of sample from BH-1 gives better representative parameters of the clay layer.

3.2.5.4 The silty sand layer (19.00 – 21.50)

The average thickness of the sand layer is 2.5 meter. The average corrected SPT blow count of the sand layer is N'_{60} =12. The internal friction angle from SPT is:

$$\phi = 0.36N'_{70} + 27 \ldots\ldots Japanese\,Railway\,S\tan darts\left(Bowles, 1996\right)$$
$$= 0.36 \times 10.3 + 27$$
$$= 31°$$

Table 3.5 The consolidation test results.

bore hole number	**sample**	**USCS**	**C_c**	**e_o**	**γ_n (kN/m^3)**
BH-1	UD (12.00 - 12.50)	MH	0.50	1.27	17
BH-5	UD (15.50 - 15.95)	CH	0.43	1.41	17
BH-9	UD (10.50 - 11.00)	CH	0.80	1.27	18
BH-23	UD (13.00 - 13.50)	CH	0.80	1.32	17

The elasticity modulus, E_s, of the sand layer is estimated from the correlation based on the SPT test. For silty sands E_s is:

$$\begin{aligned} E_s &= 300\left(N_{55}+6\right)\text{.........................}\left(Bowles, 1996\right) \\ &= 300\left(13.1+6\right) \\ &= 5730\,kPa \end{aligned}$$

The Poisson's ratio υ is assumed to be 0.3 and the saturated unit weight of the sand is γ_d = 20 kN/m^3.

3.2.5.5 Gravelly clay layer (21.50-34.00)

The average corrected SPT blow count is N'_{60} =24. The Atterberg limits, natural water content, fine content and the averages of these parameters are given in Table 3.6. It is thought that the average values represent the whole gravelly layers, but it is difficult to make decision if the soil behaves like gravel or clay. In Table 3.6 it is seen that average values of coarse content is 53%. The undrained shear strength is calculated from Scempton's correlation for fine materials.

$$\frac{c_u}{\sigma'_v} = 0.11 + 0.0037 \times I_p$$

$$\begin{aligned} \sigma'_v &= \underbrace{3.5\times18+1.5\times10}_{fill} + \underbrace{4\times10}_{sand} + \underbrace{10\times7}_{clay} + \underbrace{2.5\times10}_{silty\,sand} + \underbrace{6.25\times10}_{gravelly\,clay} \\ &= 78+40+70+25+62.5 \\ &= 275.5\,kPa \end{aligned}$$

$$\begin{aligned} c_u &= 275.5\left(0.11+0.0037\times15\right) \\ &= 46\,kPa \end{aligned}$$

The effective cohesion and effective internal friction angle values are assumed as $c' = 10kN/m^2$ and $Ø' = 30^o$ respectively.

Table 3.6 Average soil parameters of gravelly clay layer (21.50 – 34.00)

bore hole number	sample	USCS	w_L	w_p	w_n	I_p	-No 200	+No 10
BH-5	28.50 - 28.95	CL	38	21	24	17	68	2
BH-8	27.00 - 27.45	CL	37	21	17	16	52	9
BH-10	27.00 - 27.45	CL	37	21	24	16	50	6
BH-11	31.50 - 34.50	GC	24	18	15	6	17	57
BH-13	30.45 - 33.00	GC	37	21	20	16	46	29
BH-15	25.50 - 25.95	CL	44	22	27	22	61	2
BH-16	30.00 - 30.45	CL	39	21	25	18	64	4
BH-16	30.45 - 33.00	GC	24	17	19	7	20	43
BH-18	30.00 - 30.45	GC	29	17	10	12	29	33
BH-22	24.00 - 24.45	CL	46	22	27	24	64	4
	Average values		36	20	21	15	47	19

The Poisson's ratios for drained and undrained cases are thought to be as $\upsilon' = 0.3$ and $\upsilon = 0.4$ respectively. Bowles (1996) gives minimum elasticity modulus for sand and gravels as $50000\ kN/m^2$. So that elasticity module value of $E = 50000\ kN/m^2$ is used for design.

3.2.5.6 Gravel layer (34.00 – 40.00)

This layer lies between two gravelly clay layers. SPT's are failed and there is no any test performed in this layer except the soil classification tests. Bowles (1996) gives minimum and maximum elasticity modulus in the range of 50MPa – 200MPa and the value of 100MPa is selected for gravel layer. The Poisson's ratio is selected as 0.3 while cohession and friction angle values are assumed to be c=0 and $Ø=36^o$.

3.2.5.7 Gravely clay layer (40.00 – 60.00)

The average corrected SPT blow count $N'_{60} = 24$. This layer is modeled by using the second step in-situ tests. In the second step site studies there are only three boreholes opened and the samples were tested in the soil mechanic laboratory of Dokuz Eylül University, but no soil classification tests were performed for this layer.

So it is assumed that soil parameters of this layer are quite similar to the gravelly clay layer between 21.50 – 34.00 depths.

The Poisson's ratios for drained and undrained cases are thought to be as 0.3 and 0.4 respectively. The undrained elasticity modules E = 40000 kN/m^2 is used for design. The effective cohesion and effective internal friction angles are used as 5kN/m^2 and 29º respectively.

Idealized soil profile and representative soil parameters are given in Figure 3.10

3.3 Foundation Analyses

In the investigation site a building complex is planning to be built. Application plan of structures are given in Appendix A. In this section, a 1 basement story + 14 normal stories + 1 roof story building is studied. The plan and the cross section of the building are given in Appendix E. The foundation elevation is about -5.00m and the raft thickness is assumed to be 2m. The basement plan and the calculation model for the geotechnical design of the building are shown in Figure 3.11 and Figure 3.12 respectively.

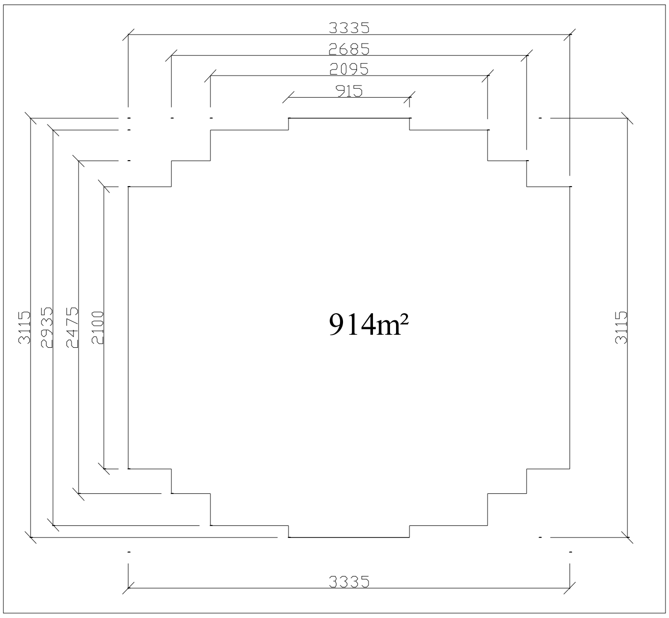

Figure 3.11 The basement plan of the building

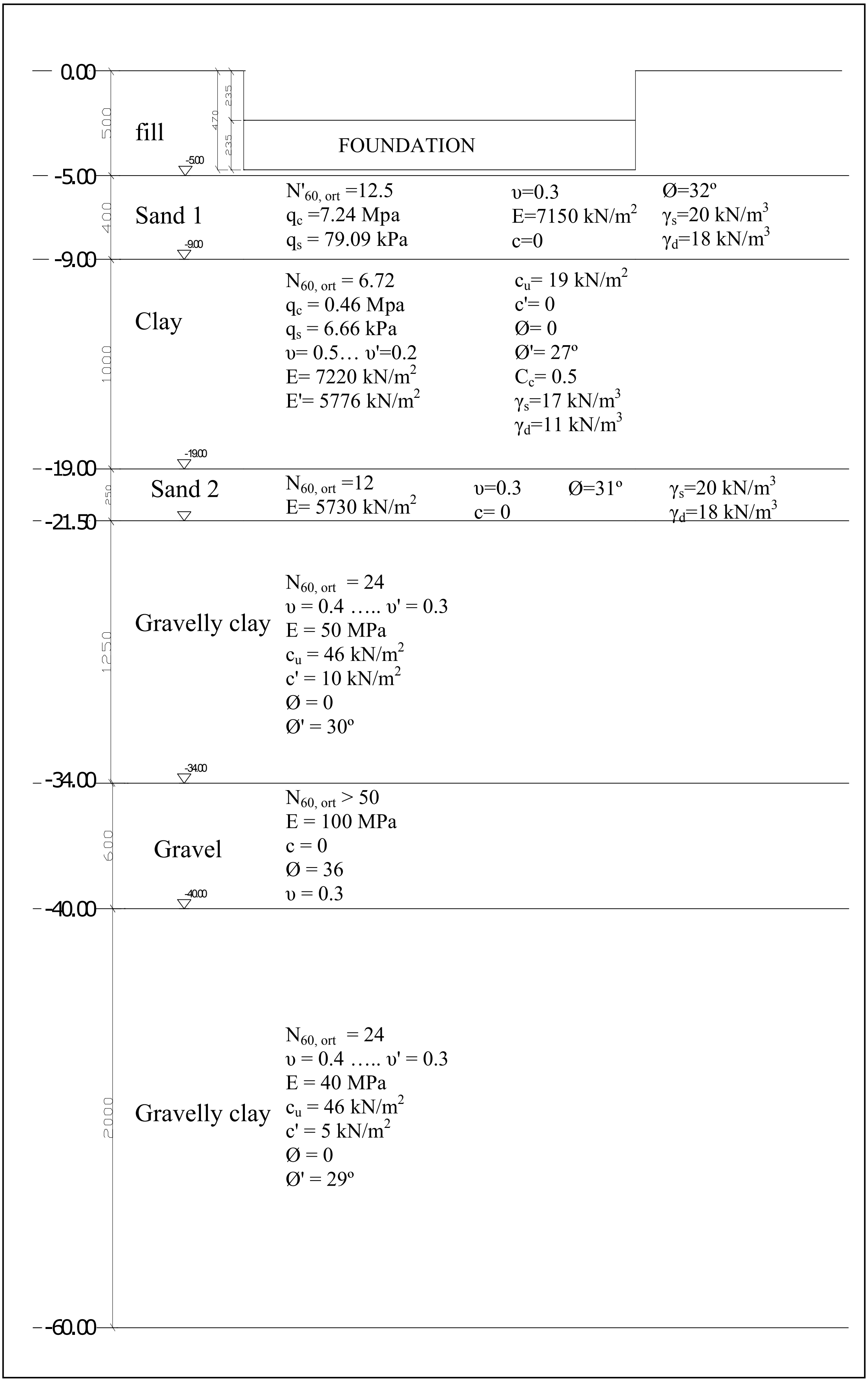

Figure 3.10 Idealized soil profile

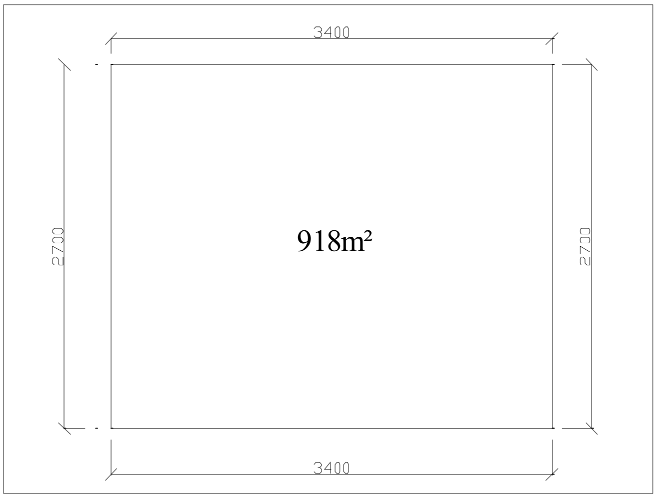

Figure 3.12 the calculation model for the geotechnical design of the building

3.3.1 Raft foundation analyses

The raft foundation is assumed two meter thick flat plate type mat foundation. Structural loads are G=13862t and Q= 4949t. The plan dimensions are B= 27m, L=34m and the foundation area is A= 918m^2 as given in Figure 3.12. The weight of excavated soil is:

$$W_{exc} = h \times A \times \gamma_s$$
$$= 4.7 \times 918 \times 2$$
$$= 8629t$$

The weight of the foundation is:

$$W_{foun} = h \times A \times \gamma_{concrete}$$
$$= 2.35 \times 918 \times 2.5$$
$$= 5393t$$

The net foundation contact pressure for bearing capacity is:

$$
\begin{aligned}
q_{net,bearing} &= \frac{G+Q+W_{foun}-W_{exc}}{A} \\
&= \frac{13862+4949+5393-8629}{918} \\
&= \frac{15575}{918} \\
&= 17t/m^2
\end{aligned}
$$

The net foundation contact pressure for settlement is:

$$
\begin{aligned}
q_{net,sett} &= \frac{G+Q/2+W_{foun}-W_{exc}}{A} \\
&= \frac{13862+2475+5393-8629}{918} \\
&= \frac{13101}{918} \\
&= 14.3t/m^2
\end{aligned}
$$

The bearing capacity of the raft is calculated according to both Meyerhof and Hansen's methods. The foundation stands on the sand layer. But there is a soft clay layer lies under the sand layer. So in the first case the bearing capacity is calculated for case of the soil profile consist of sand layer (Appendix F). The minimum bearing capacity is calculated as q_{sand}=208 t/m^2. In the second case the stress distribution and the raft dimensions are transfered to the clay layer with 63.5º assumption. In this case the imaginary foundation dimensions are B'=31m and L'=38m.

The minimum bearing capacity is calculated as q_{clay} = 7.8 t/m^2 for safety factor F=3. The contact pressure needed to cause 7.8 t/m^2 pressure on the clay layer is:

$$
\begin{aligned}
q &= \frac{7.8\times31\times38}{27\times34} \\
&= 10t/m^2 \text{................for the safety factor } F=3
\end{aligned}
$$

$q < q_{net,\ bearing}$, so the bearing capacity is over passed for this case. This is an assumption for the bearing capacity but its clear that the safety factor is too small. Deep foundation systems have to be used.

The raft foundation settlements analyses are also performed. The consolidation settlement is calculated for cohesive layers and the elastic settlement is calculated for granular layers. Raft settlements are:

Consolidation settlement: 73.2cm
Elastic settlement : 7.6 cm
Total settlement : 80.8cm

The settlement calculations and the stress increment model are given in Appendix-F.

The total settlement exceeds the allowable settlement of 10 cm. Deep foundation system s have to be used.

3.3.2 Piled foundation analyses

In Izmir region piled foundations are mostly designed according to the idea that piles carry the whole structural loads. Pile spacing is usually taken as three times the diameter of the pile. For this project pile diameter is selected as 1 meter with a length of 35 meters and pie spacing is 3 meters. Pile disposition and the crossection of the piled foundation are given in Appendix G.

3.3.2.1 Piled foundation bearing capacity

The piled foundation bearing capacity is calculated by using the α method. Pile skin friction is calculated as:

$$f_s = \alpha c + \sigma' K \tan\delta$$
$$K = \frac{K_a + F_w K_o + K_p}{2 + F_w}$$
$$K_a = \tan^2(45 - \phi/2)$$
$$K_p = \tan^2(45 + \phi/2)$$
$$K_o = 1 - \sin\phi$$

The adhesion factor α can be obtained from the Figure 3.13.

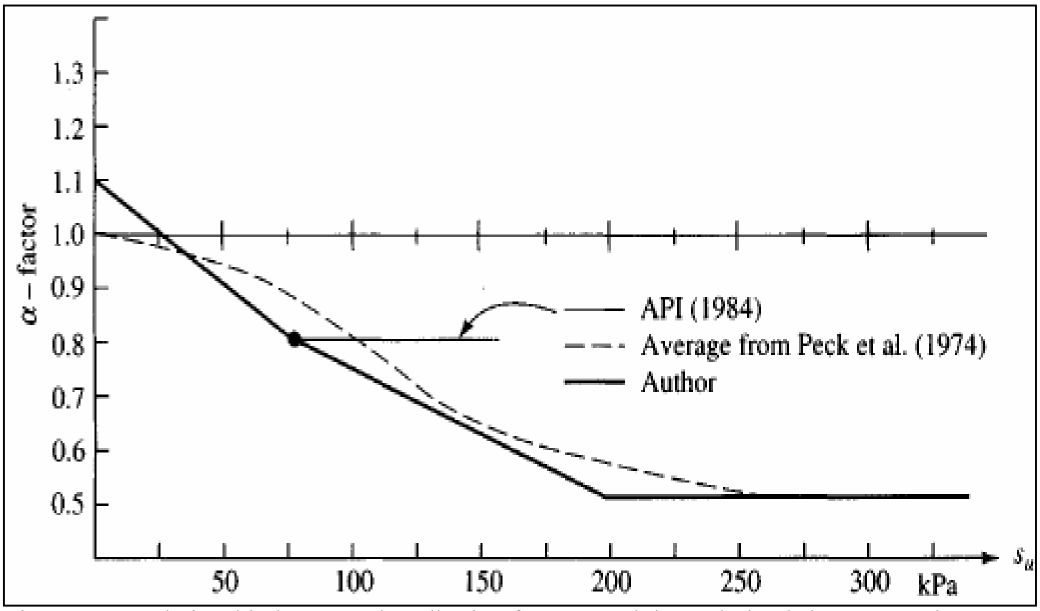

Figure 3.13 Relationship between the adhesion factor α and the undrained shear strength s_u.

Sand layer (5.00 – 9.00)

$$f_s = \sigma' K \tan\delta$$
$$\phi = 32°...............\delta = 0.6 \sim 0.8\phi' (Bowles, 1996).......\delta = 0.75\phi' \text{ is selected}$$
$$\delta = 23°$$
$$K_a = 0.307; K_p = 3.255; K_o = 0.470; F_w = 2$$
$$K = 1.13$$
$$\sigma' = 3.5\times18 + 1.5\times10 + 2\times10$$
$$= 98 kPa$$
$$f_s = 98\times1.13\times\tan 23$$
$$= 47 kPa$$

Clay layer (9.00 – 19.00)

$f_s = \alpha c_u$

$c_u = 19kPa .... \alpha = 1.05 \textit{ from Figure } 3.13.$

$\quad {}_s = 1.05 \times 19$

$\quad = 20kPa$

Sand layer (19.00 – 21.50)

$f_s = \sigma' K \tan\delta$

$\phi = 31° .... \delta = 0.75\phi' \textit{ is selected}$

$\delta = 23°$

$K_a = 0.324; K_p = 3.086; K_o = 0.489; F_w = 2$

$K = 1.10$

$\sigma' = 3.5 \times 18 + 1.5 \times 10 + 4 \times 10 + 10 \times 7 + 1.25 \times 10$

$\quad = 200.5kPa$

$f_s = 200.5 \times 1.10 \times \tan 23$

$\quad = 94kPa$

Gravelly clay layer (21.50-34.00)

$f_s = \alpha c_u$

$c_u = 46kPa .... \alpha = 0.93 \textit{ from Figure } 3.13.$

$f_s = 0.93 \times 46$

$\quad = 43kPa$

Gravel layer (34.00 – 40.00)

$f_s = \sigma' K \tan\delta$

$\phi = 36° .... \delta = 0.75\phi' \textit{ is selected}$

$\delta = 27°$

$K_a = 0.259; K_p = 3.852; K_o = 0.412; F_w = 1$

$K = 1.51$

$\sigma' = 3.5 \times 18 + 1.5 \times 10 + 4 \times 10 + 10 \times 7 + 2.5 \times 10 + 12.5 \times 10 + 3 \times 11$

$\quad = 370kPa$

$f_s = 370 \times 1.51 \times \tan 27$

$\quad = 285kPa$

The shaft resistance of a single pile is:

$$
\begin{aligned}
f_{s,t} &= (4\times 47 + 10\times 20 + 2.5\times 94 + 12.5\times 43 + 6\times 285) \\
&= 2870 \\
F_s &= f_{st}\times 1\times \pi \\
&= 9016 kN
\end{aligned}
$$

Pile tip stands on the gravelly clay layer and the clay content changes over the investigation site. In order to be on the safe side the pile tip resistance is not taken into account. Then the single pile bearing capacity for the safety factor is F=3:

$$
\begin{aligned}
Q &= F_s = 9016 kN \\
Q_{design} &= \frac{Q}{3} .... F = 3 \\
&= 3000 kN
\end{aligned}
$$

The weight of the structure is:

$$
\begin{aligned}
W_{structure} &= G + Q + W_{foun} \\
&= 138620 + 49490 + 53930 \\
&= 242000 kN
\end{aligned}
$$

There are 120 piles and the structural load of a single pile is:

$$
\begin{aligned}
Q_{pile} &= \frac{242000}{120} \\
&= 2016 kN
\end{aligned}
$$

$Q_{design} > Q_{pile}$ so the selected pile disposition, pile diameter and pile length satisfies the design criteria.

3.3.2.2 Piled foundation settlements

Piled foundation settlements are calculated by using the pile group assumption with the method suggested by United States Department of Transportation Federal Highway Administration [US-FHWA], (1996). Foundation settlement calculation model is given in Appendix G and analyses are given in Appendix H. The consolidation settlement, the elastic settlement and the total settlement of the piled foundation are 8.1cm, 1cm, and 9.1 cm respectively.

The consolidation settlement is calculated by using the one dimensional consolidation theory. In the real loading conditions the settlements will be lesser than the calculated value. So the three dimensional settlement is not studied here.

3.3.3Piled Raft foundation hand calculations

The piled raft foundation simple hand calculations are performed by using the method suggested by Poulos, (2000). The method has detailed in chapter two. To adopt the structural model to the hand calculation method, the filling and the basement floor were ignored. The foundation loads are then:

$$\underline{for\ short\ term\ analysis}$$

$$
\begin{aligned}
W_{structure} &\approx G+Q \\
&\approx 138620+49490 \\
&\cong 188MN
\end{aligned}
$$

$$B=27m; L=34m; A=918m^2$$

$$
\begin{aligned}
q &= \frac{W_{structure}}{A} \\
&= \frac{188000}{918} \\
q &\cong 210kPa
\end{aligned}
$$

$$
\begin{aligned}
Q_{pile} &= \frac{188}{120} \\
&= 1.56MN
\end{aligned}
$$

$$\underline{for\ long\ term\ analysis}$$

$$
\begin{aligned}
W_{structure} &\approx G+Q/2 \\
&\approx 138620+49490/2 \\
&\cong 163MN
\end{aligned}
$$

$$B=27m; L=34m; A=918m^2$$

$$
\begin{aligned}
q &= \frac{W_{structure}}{A} \\
&= \frac{163000}{918} \\
&\cong 178kPa
\end{aligned}
$$

$$
\begin{aligned}
Q_{pile} &= \frac{163}{120} \\
&= 1.36MN
\end{aligned}
$$

Unfavorable soil conditions for pilled raft foundation design observed by Poulos, (1991) were also described in section two. In the soil model there are soft compressible layers at relatively shallow depths. Although the raft foundation stands on the sand layer, just blow the sand layer a thick clay layer stands. Because of the consolidation settlement the contact pressure of the raft will decrease and piles will be loaded over the design load. To avoid from this problem piled raft foundation is designed by disconnected piles. A bedding layer will be settled on the pile heads and the raft will stand on this bedding layer. Especially for basement floor applications if horizontal and moment loads can safely carried by the raft alone, then the piles can only used as settlement reducers. Wong&Chang&Cao, (2000) also suggests this method to avoid from the high moment loads on rigid connections between piles and the raft. So piles will not carry the moment and lateral loads.

3.3.3.1 Piled Raft bearing capacity

The total bearing capacity of the raft is therefore:

$$Q_r = A \times P_{ur}$$

$$Where\ A : the\ area\ of\ the\ raft\ surface$$

$$Q_r = (27 \times 34) \times 300$$

$$= 276MN$$

The total axial capacity of piles is:

$$Q_{pc} = Q_p \times n$$

$$= 9 \times 120$$

$$= 1080MN$$

If the raft and the pile capacities are added, the total capacity of the foundation in compression is:

$$
\begin{aligned}
Q_{pr} &= Q_r + Q_{pc} \\
&= 276 + 1080 \\
&= 1356 MN
\end{aligned}
$$

The bearing capacity of the block containing the raft and the piles must now be considered. Piles are at the edges of the raft. The block capacity is:

$$
\begin{aligned}
\text{The block capacity} &= \text{shaft friction} + \text{the end axial capacity} \\
&= [2 \text{ x } (27+34) \text{ x}2870] + [48 \times (27 \times 34)] \\
&= 392 MN
\end{aligned}
$$

This is below the sum of the raft and the pile capacities, and the design value of the ultimate capacity of the foundation is 392 MN. The corresponding factor of safety is:

$$
F = \frac{392}{V} = \frac{392}{188} = 2
$$

3.3.3.2 Piled Raft settlement calculations

The most important problem of the use of the hand calculation method is hat it is not clear how to use the method in layered soil profiles. It is not practical to model the whole soil profile by one representative value.

The investigation site soil profiles consist of various soil layers. The bearing capacities were calculated for the layered soil profile and given in section 3.3.1 and 3.3.2. Although the bearing capacities of the foundation system are calculated for idealized soil profile, the raft and piles stiffness values are calculated by using the parameters of the clay layer lying 9.00 – 19.00 meter depths.

$\underline{\text{the axial stiffness of the raft for the effective (long term analysis) case } (\upsilon = 0.2)}$

$P_{av} = \dfrac{P}{\pi a^2}$.................equivalent circular raft with the same area is:

a=$\sqrt{\dfrac{17 \times 34}{\pi}} = 13.6m$

$P_{av} = \dfrac{163}{\pi \times 13.6^2} = 0.28$

$\dfrac{a}{h} = \dfrac{13.6}{35} = 0.39$........from Figure 2.10 $for\ \upsilon = 0.2$........$I_p = 1.08$

$\rho_z = \dfrac{0.28 \times 13.6}{7.22} \times 1.08$

$= 0.57m$

therefore the axial stiffness of the raft K_r is :

$K_r = \dfrac{V}{\rho_z}$

Where V : verticle load

$K_r = \dfrac{163}{0.57} = 286 MN/m$

$\underline{\text{the axial stiffness of the raft for the undrained (short term analysis) case } (\upsilon = 0.5)}$

$P_{av} = \dfrac{P}{\pi a^2}$.................equivalent circular raft with the same area is:

a=$\sqrt{\dfrac{17 \times 34}{\pi}} = 13.6m$

$P_{av} = \dfrac{188}{\pi \times 13.6^2} = 0.32$

$\dfrac{a}{h} = \dfrac{13.6}{35} = 0.39$........from Figure 2.10 $for\ \upsilon = 0.5$........$I_p = 0.72$

$\rho_z = \dfrac{0.32 \times 13.6}{5.73} \times 0.72$

$= 0.55m$

therefore the axial stiffness of the raf$t\ K_r\ is$:

$K_r = \dfrac{V}{\rho_z}$

Where V : verticle load

$K_r = \dfrac{188}{0.55} = 342 MN/m$

<u>the axial stiffness of single pile for the undrained (short term analysis) case ($\upsilon = 0.5$)</u>

$$G = \frac{7.22}{2(1+0.5)} = 2.4MPa$$

$$\rho = 1$$

$$\lambda = \frac{30250}{2.4} \text{.........................} E = 30250MPa \text{ for C25 class concrete } (Ersoy, 1985)$$

$$= 12604$$

$$r_m = 2.5 \times 35 \times (1-0.5)$$

$$= 43.75$$

$$r_0 = 0.5$$

$$\zeta = \ln(43.75/0.5)$$

$$= 4.47$$

$$\mu l = \frac{35}{0.5}\sqrt{\frac{2}{4.47 \times 12604}}$$

$$= 0.42$$

From Figure 2.11 $\frac{\tanh(\mu l)}{\mu l}$ is obtained as 0.92

$$K_p = \left(\frac{2 \times \pi}{4.47}\right) \times 35 \times 2.4 \times 1 \times 0.92$$

$$= 108MN/m$$

<u>the axial stiffness of single pile for the effective (long term analysis) case ($\upsilon = 0.2$)</u>

$$G = \frac{5.72}{2(1+0.2)} = 2.3MPa$$

$$\rho = 1$$

$$\lambda = \frac{30250}{2.3} \text{.........................} E = 30250MPa \text{ for cocrete class C25 } (Ersoy\, 1985)$$

$$= 13152$$

$$r_m = 2.5 \times 35 \times (1-0.2)$$

$$= 70$$

$$r_0 = 0.5$$

$$\zeta = \ln(70/0.5)$$

$$= 4.94$$

$$\mu l = \frac{35}{0.5}\sqrt{\frac{2}{4.94 \times 13152}}$$
$$= 0.39$$

From Figure 2.11 $\frac{\tanh(\mu l)}{\mu l}$ is obtained as 0.96

$$K_p = \left(\frac{2 \times \pi}{4.94}\right) \times 35 \times 2.3 \times 1 \times 0.96$$
$$= 98$$

Assuming that the group factor is approximated as $\sqrt{n_p}$ (where n_p is the number of piles), the following initial pile group stiffness are obtained.

- undrained case; K_{pi} =1183MN/m
- drained case ; K_{pi} =1073MN/m

For undrined case $X = \frac{1-0.6(342/1183)}{1-0.64(342/1183)} = 1.014$

K_{ue} =1.014×1183 = 1200 MN/m

For drined case $X = \frac{1-0.6(286/1073)}{1-0.64(286/1073)} = 1.012$

K_{ue} =1.012x1073 = 1085 MN/m

Proportion of load carried by the piles

$$\beta_p = 1/(1+\alpha) \tag{2.20}$$

$$\alpha = \frac{0.2}{1-0.8(K_r/K_p)}\left(\frac{K_r}{K_p}\right) \tag{2.21}$$

<u>For undrained conditions:</u>

$$\alpha = \frac{0.2}{1-0.8\left(342/1183\right)}\left(\frac{342}{1183}\right)$$
$$= 0.075$$
$$\beta_p = \frac{1}{1+0.075}$$
$$= 0.93$$

<u>For drained conditions:</u>

$$\alpha = \frac{0.2}{1-0.8\left(286/1073\right)}\left(\frac{286}{1073}\right)$$
$$= 0.068$$
$$\beta_p = \frac{1}{1+0.068}$$
$$= 0.94$$

Table 3.7 Calculation of load settlement curve for piled raft foundation in worked example. (undrained case)

Vru (MN)	276		Vpu (MN)	1080						
V	Kr (MN/m)	Kp (MN/m)	X	β_p	Vp (MN)	Vr (MN)	V_A (MN)	Kpr (MN/m)	S (mm)	V>VA
0	342.0	1183.0	1.014	0.930	0	0	1161.2	1199.8	0.0	NO
20	340.7	1172.8	1.014	0.930	18.60	1.40	1161.2	1189.5	16.8	NO
40	339.4	1162.6	1.014	0.930	37.19	2.81	1161.7	1179.2	33.9	NO
60	338.1	1152.5	1.014	0.929	55.75	4.25	1162.3	1169.0	51.3	NO
80	336.7	1142.3	1.014	0.929	74.30	5.70	1162.8	1158.8	69.0	NO
100	335.3	1132.2	1.015	0.928	92.84	7.16	1163.3	1148.6	87.1	NO
120	334.0	1122.0	1.015	0.928	111.35	8.65	1163.8	1138.4	105.4	NO
140	332.6	1111.9	1.015	0.928	129.85	10.15	1164.4	1128.2	124.1	NO
160	331.2	1101.8	1.015	0.927	148.34	11.66	1164.9	1118.1	143.1	NO
188	329.2	1087.6	1.015	0.927	174.21	13.79	1165.5	1103.8	170.3	NO
200	328.3	1081.6	1.015	0.926	185.21	14.79	1166.3	1097.8	182.2	NO
220	326.8	1071.5	1.015	0.926	203.67	16.33	1166.6	1087.6	202.3	NO

At long term design load of 188MN, the calculated immediate settlement is 170mm.

It will be assumed that the final consolidation settlement (S_{CF}) can be computed as the difference between the total final and immediate settlements from purely elastic analysis.

$$S_{cf} = V\left(\frac{1}{K'_e} - \frac{1}{K_{ue}}\right) \tag{2.25}$$
$$= 188\left(\frac{1}{1085} - \frac{1}{1200}\right)$$
$$= 16mm$$

The total final settlement is 0.170 + 0.016 = 0.186m

Although the final settlement exceeds the settlement limits, the calculations were based on the idea that the whole soil profile consists of the CH layer. The real settlement will be lesser than this.

3.3.3.3 Piled raft differential settlements

The simplified method given by Horikhoshi & Randolph (1997) was described in Section 2.23. The soil raft stiffness is:

$$K_{rs} = 5.57\frac{30000}{7.22}\frac{(1-0.2^2)}{(1-0.2^2)}\left(\frac{27}{34}\right)^{1/2}\left(\frac{2}{34}\right)^3$$
$$= 4.20$$

From the above reference, the ratio of the maximum differential settlement to the average settlement is 0.13 (Figure 2.12). Assuming that this ratio applies also to the piled raft, the maximum long term differential settlement (center to corner) is 0.13x0.186= 0.024m.

3.3.4 Finite element analysis

Plaxis 3D Foundation version 1.1 has been used for the finite element analysis. The idealized soil profile given in Figure 3.10 has been used in Plaxis. But the fill layer existing in the soil profile consists of excavation materials and the wastes of nearby constructions built in few years time. As mentioned previously in Section 3.2.5.1 it is not very well known that the clayey layers have completed their consolidation settlement under the fill load or not. So that in the finite element model the fill load around the structure has been ignored to stay at the safe side.

The finite element analyses were performed for the quarter of the foundation area because of the symmetry. No safety factors were applied in the finite element analysis. Structural loads are assumed as acting on the raft surface uniformly and the design load is:

$$h_{foun} = 2m; B = 27m; L = 34m; A = 918m^2$$
$$\begin{aligned} W_{design} &\approx G + Q + W_{foun} - W_{exca} \\ &\approx 138620 + 49490 - \left(2\times 2,4\times 918 - 5\times 18\times 918\right) \\ &\cong 149554kN \end{aligned}$$
$$q_{design} = \frac{W_{design}}{A} = \frac{149554}{918} = 163kPa$$

In the first case the raft foundation has been analyzed and the maximum settlement of the raft foundation for the design load of 163kPa is determined to be 55.8cm. This exceeds the allowable settlement value of 10cm. The plan and crossection views of vertical displacements are given in Figure 3.14 and Figure 3.15, respectively. Greater settlements generated under the centre of the structure, because the stress increment is greater in the middle of the foundation as can be seen in Figure 3.16.

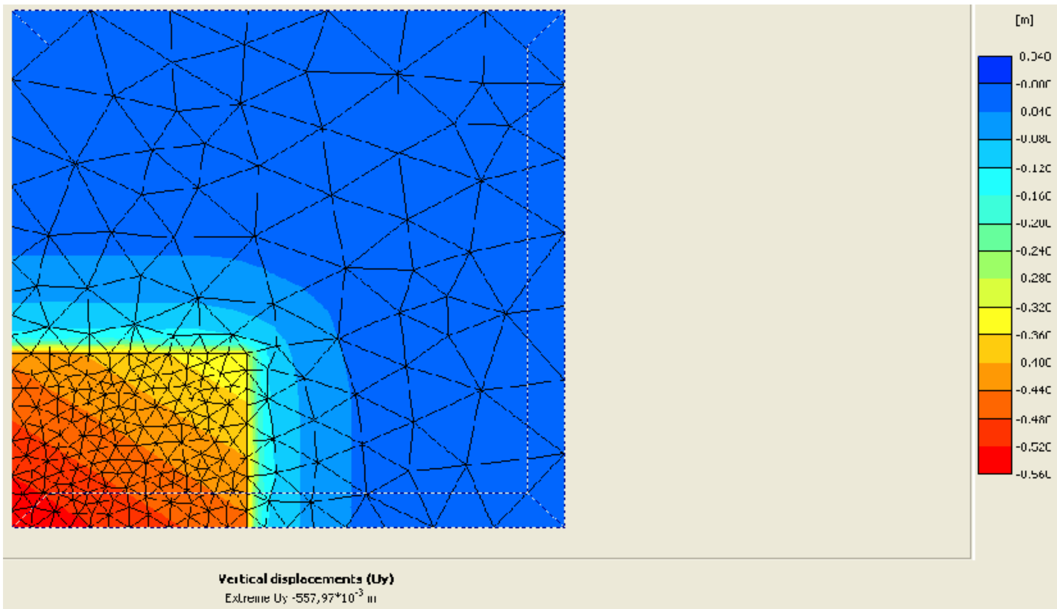

Figure 3.14 The plan view of the settlements of the raft foundation

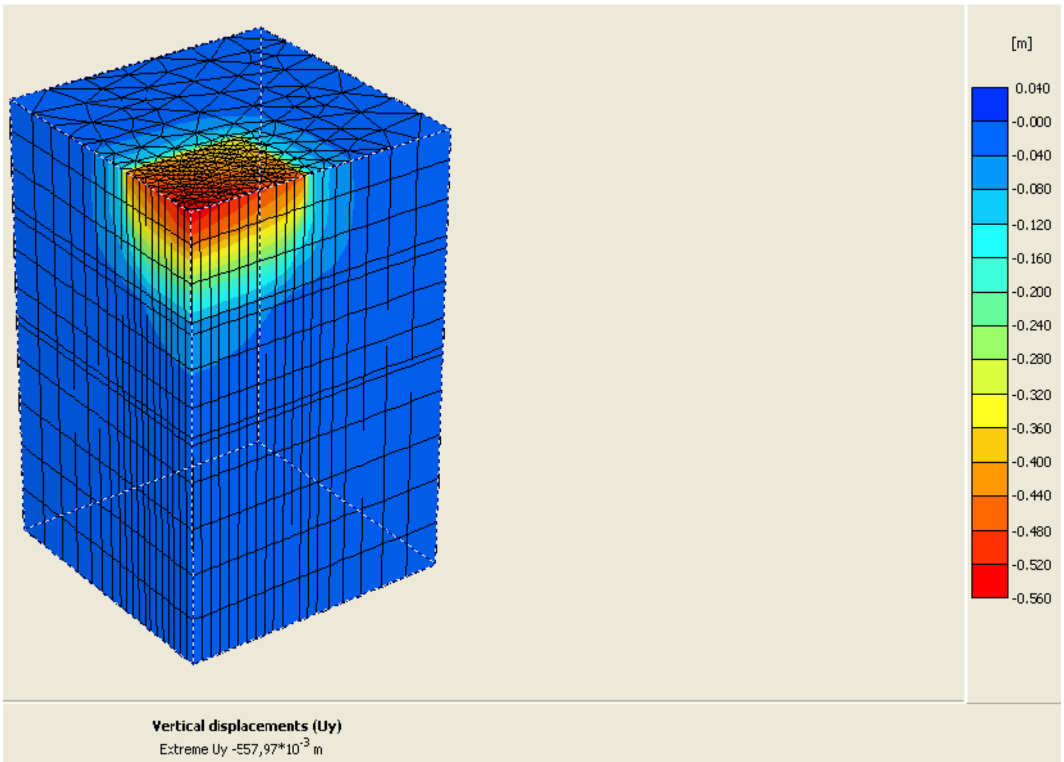

Figure 3.15 The crossectional view of the settlements of the raft foundation

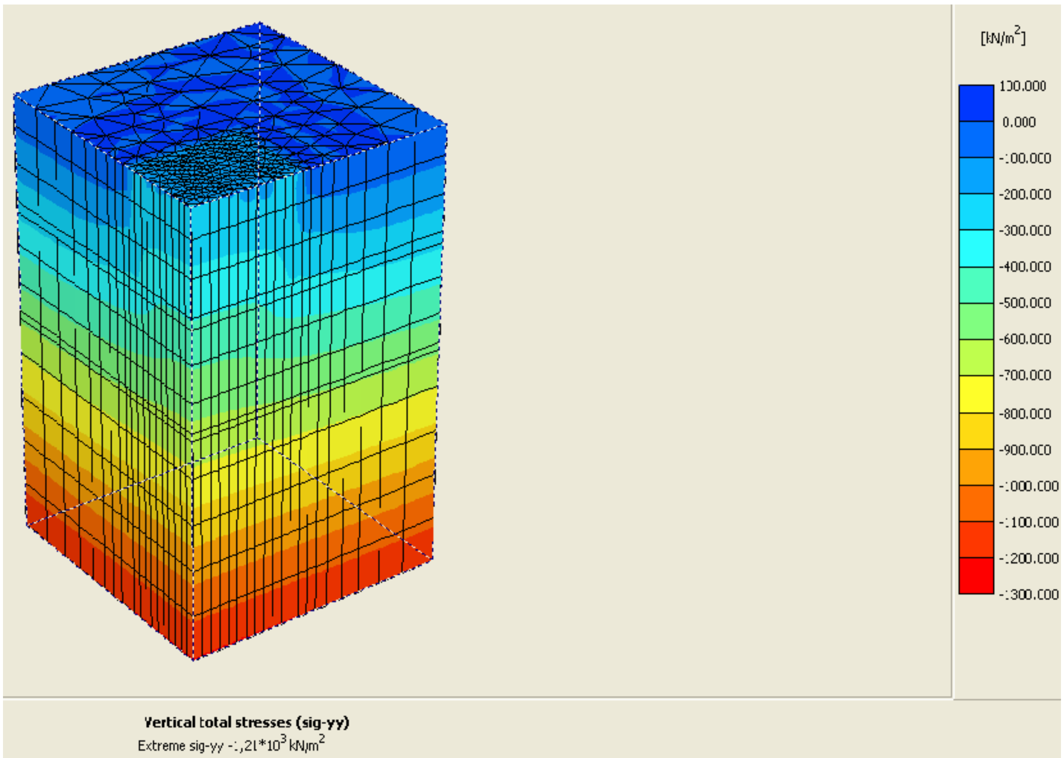

Figure 3.16 The crossectional view of the effective stresses generated under the raft foundation

The piled foundation system is used to restrict the total settlement of the foundation. Conventional calculation method resulted 30 piles with 35m length and 1m diameter with a spacing of 3m. Foundation model with 30 piles is given in Figure3.17.

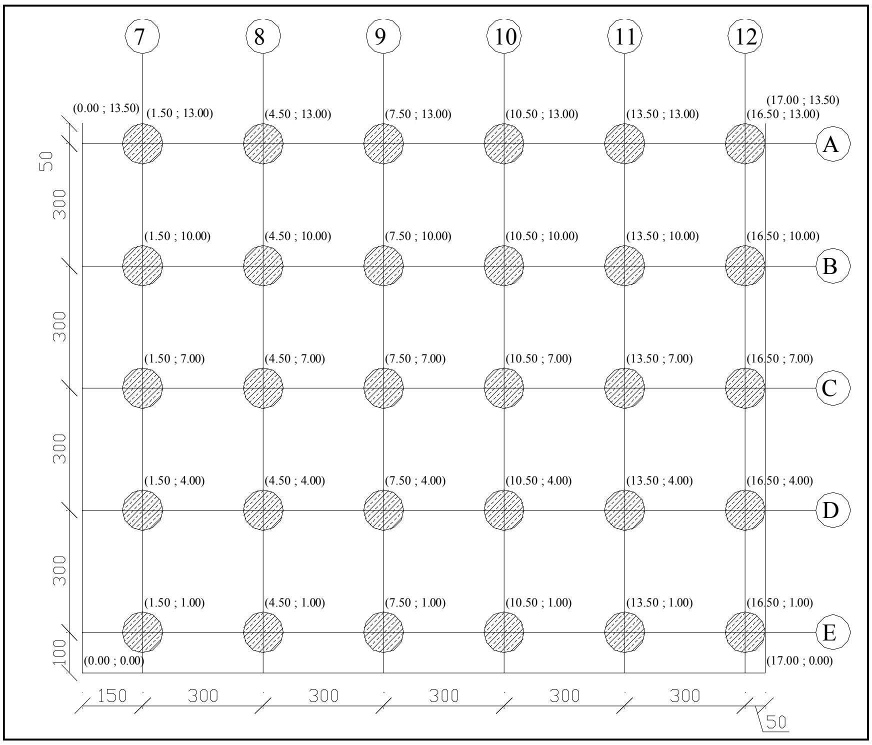

Figure 3.17 The piled foundation model for finite element analyses.

Piles are used as settlement reducers and are disconnected. The settlement criterion of raft will be provided by using disconnected piles. So that the settlement limit is assumed as 10cm. The piled foundation model has been analyzed by using the stage construction method and the maximum settlement was obtained as 4.6cm under the design load. This satisfies the settlement criteria. The maximum settlement is in the middle of the raft. The maximum differential settlement $\Delta\delta/L = 0.002119 < 1/350 = 0.002857$. The differential settlement criterion has also been satisfied. The plan and section views of vertical displacements are given in Figure 3.18 and Figure 3.19, respectively.

Both the raft foundation and piled foundation systems have been analyzed under increasing loads. The load settlement behavior of the raft foundation and the piled foundation is given in Figure 3.20. It is clearly seen that the raft foundation does not satisfy the design criterion under service loads, while the piled foundation does. But piles are over conservative under the design load.

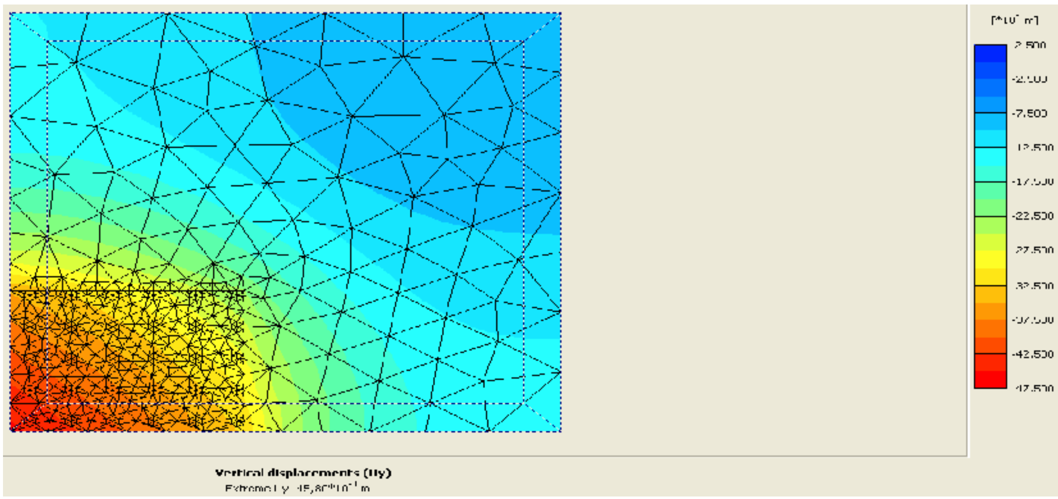

Figure 3.18 The plan view of the settlement of the piled foundation with 30 piles, 35m height

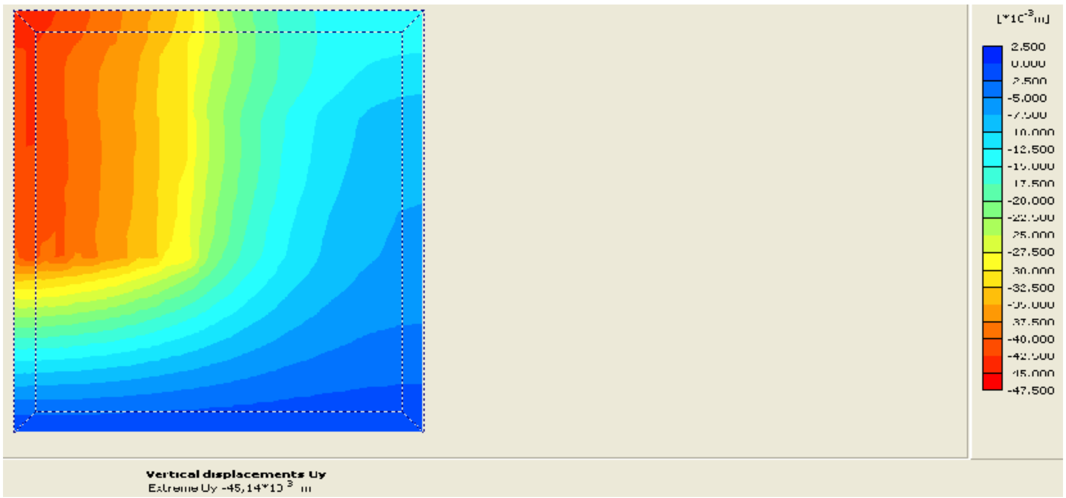

Figure 3.19 The cross section of the settlement of the piled raft foundation

Figure 3.20 Load settlement behavior of the raft and the piled foundations with 30 piles (35m)

Figure 3.21 represents the effective stresses generated under the piled foundation system. The effective stress under the raft is equal to the effective stress around the foundation cap. This means that almost all structural loads are carried by piles. Contribution of the raft to the load carrying mechanism is limited. The main reason of this is the soft layers existing on the top of the soil profile. When the raft is loaded the soft layers under the raft foundation starts to settle. They do not resist to settlement, so that piles are loaded. The major part of the structural load is carried by piles.

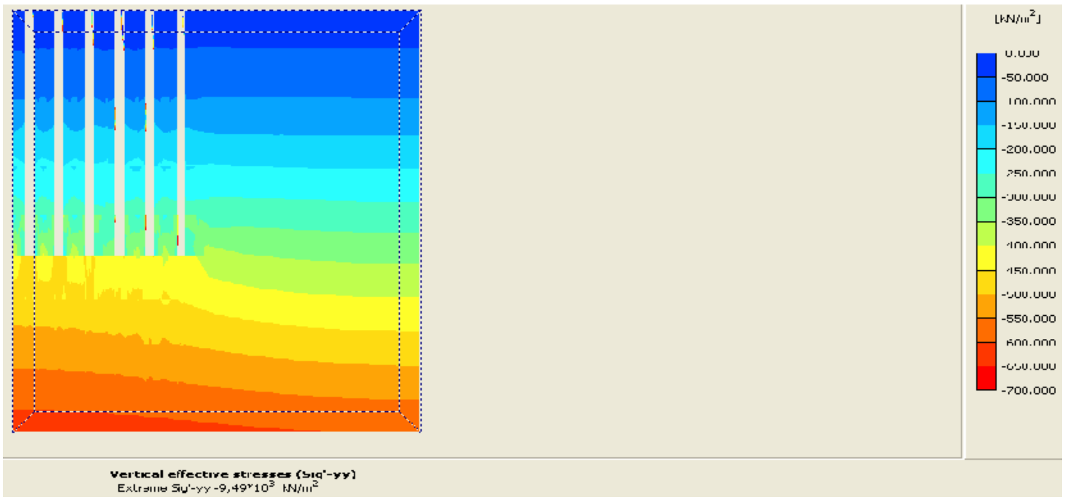

Figure 3.21 Vertical effective stresses (30 piles with 35m height)

For the same pile diameter and disposition Plaxis analyses have been performed for different lengths.. The vertical effective stress views of piles with 30m, 25m, 20m, and 14m lengths are given in Figure 3.22, Figure 3.23, Figure 3.24 and Figure 3.25, respectively. The maximum settlement for 14m length piles is found to be 16.9 cm and this exceeds the settlement limits. Although the settlement limits are over passed, the pile load carrying percentages are almost equal. The change in the pile length does not make a major effect on the load sharing behavior between the raft and piles with pile spacing of 3m.

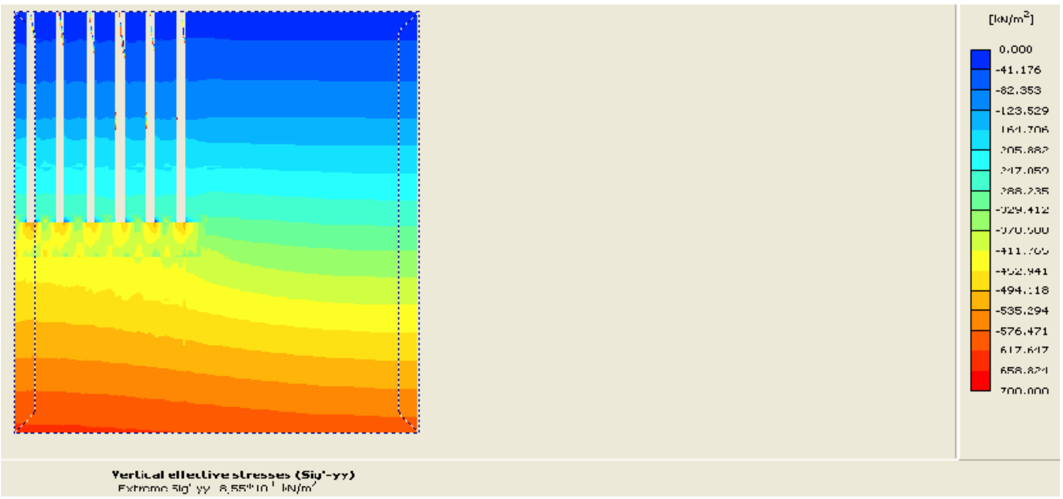

Figure 3.22 Vertical effective stresses (30 piles with 30m height)

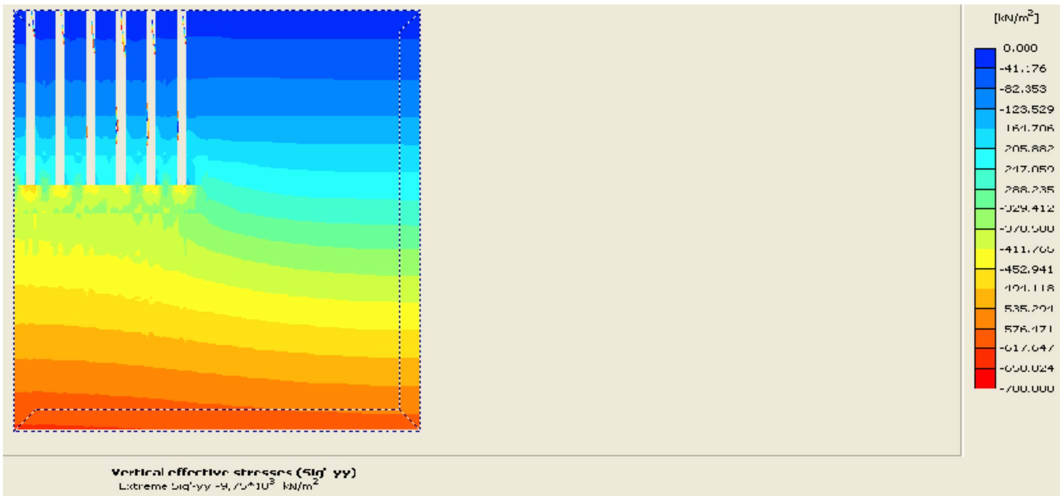

Figure 3.23 Vertical effective stresses (30 piles with 25m height)

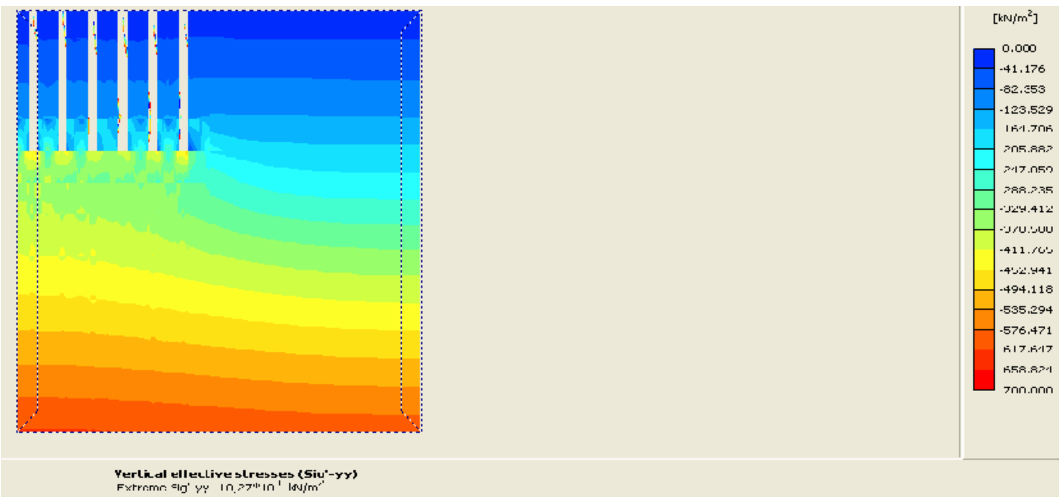

Figure 3.24 Vertical effective stresses (30 piles with 20m height)

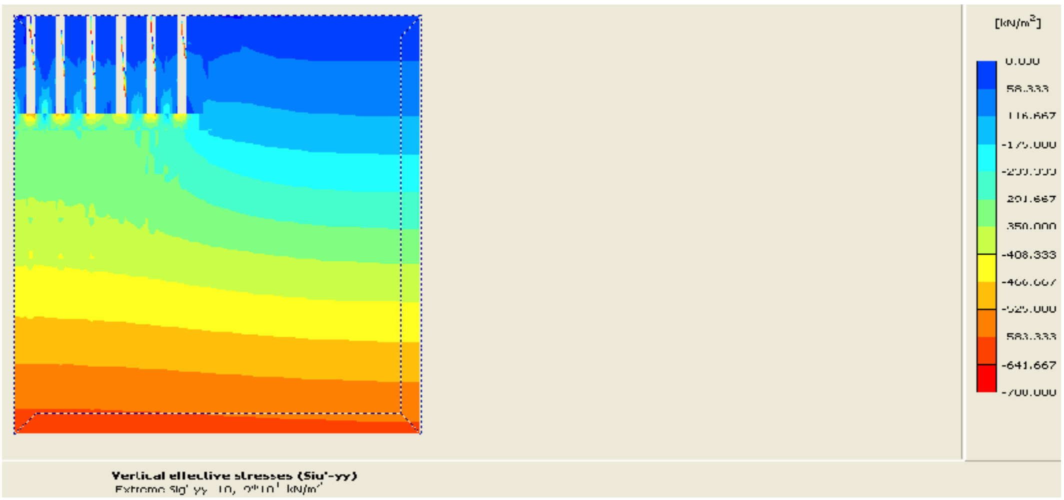

Figure 3.25 Vertical effective stresses (30 piles with 14m height)

Pile spacing is another variable in the load sharing mechanism between the raft and piles. To estimate the effect of the pile disposition and pile spacing, piled foundation has been analyzed with constant pile length of 25 meters. Also a pile loading test is modeled for the same soil model in Plaxis 3D for 25m pile length. A load of 7000kN has been applied on the pile. The load – settlement curve is given in Figure 3.31 and the shape of the curve represents the behavior of a friction pile in soft – firm clay or loose sand described by Tomlinson (1994). There are different assumptions on describing the bearing capacity of piles from pile loading test. Pile bearing capacity is assumed as 10% of pile diameter. Then the bearing capacity of pile for 1m diameter is approximately 3500 kN (Figure 3.31). Results of analyses of piled foundations with different number of piles and pile dispositions are given in Table 3.8. Maximum pile load on 30 and 20 piles are 2148 kN and 2480 kN and the load carried by 30 and 20 piled foundation are 61% and 71% of pile bearing capacity. The load on the maximum loaded pile in the foundation system with 12 piles is 3669 kN. Piles work under 100% of bearing capacity. But the load on the maximum loaded pile in 9 piled system is 4150 kN and is over the bearing capacity. Although piles work over the bearing capacity, raft only contributes to 8% of the total load. Because of the soft layers just under the foundation, the contribution of the raft to the total load is limited. The application plan and pile loads of 30 piles, 20 piles, 12 piles and 9 piles are given in Figure 3.24, Figure 3.25, Figure 3.26 and in Figure3.27, respectively.

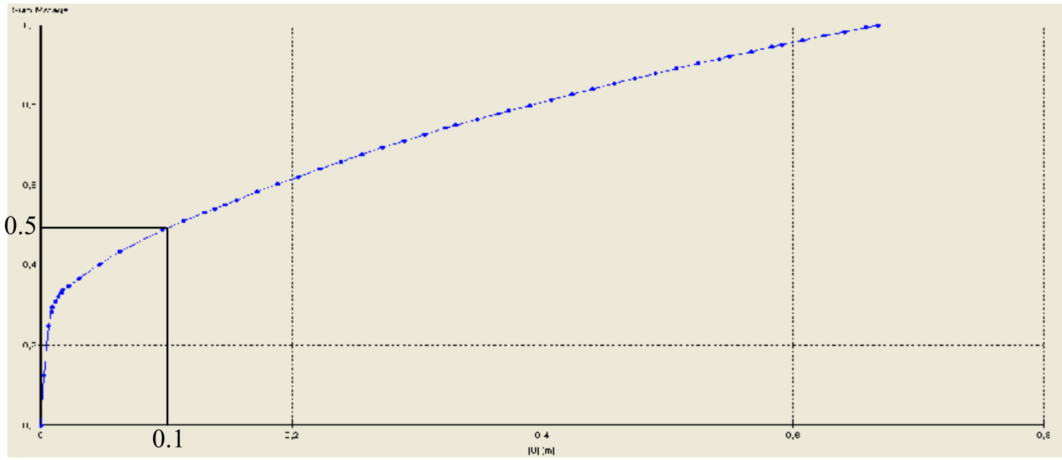

Figure 3.26 Pile loading test load settlement curve for a pile length of 25m with 1m diameter

Table 3.8 The load sharing behavior, the maximum settlement and angular distortion values of foundation system for different number of piles

Pile number	**30**	**20**	**12**	**9**
Load carried by piles β (%)	99	98	96	92
maximum settlement	6.1	5.1	6.0	6.6
angular distortion	0.00129	0.00115	0.00023	0.00051

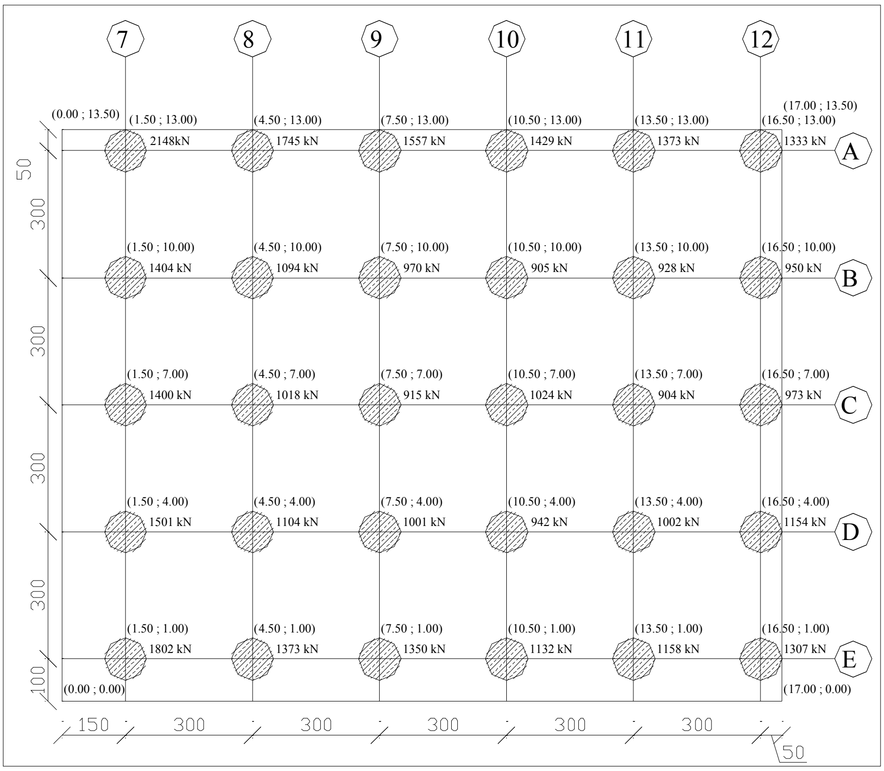

Figure 3.24 Pile disposition plan and pile loads (30 piles with 25m length)

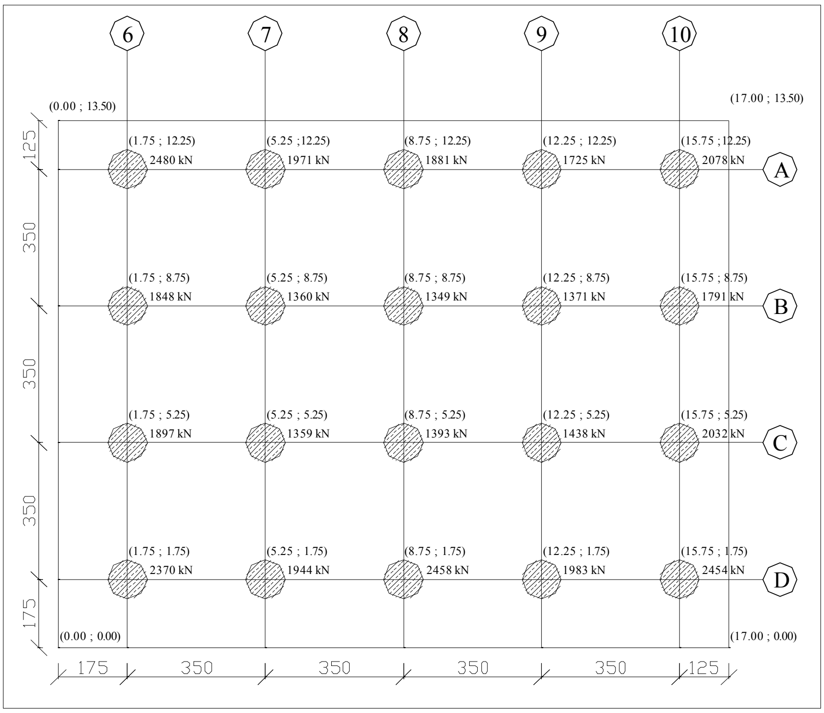

Figure 3.25 Pile disposition plan and pile loads (20 piles with 25m length)

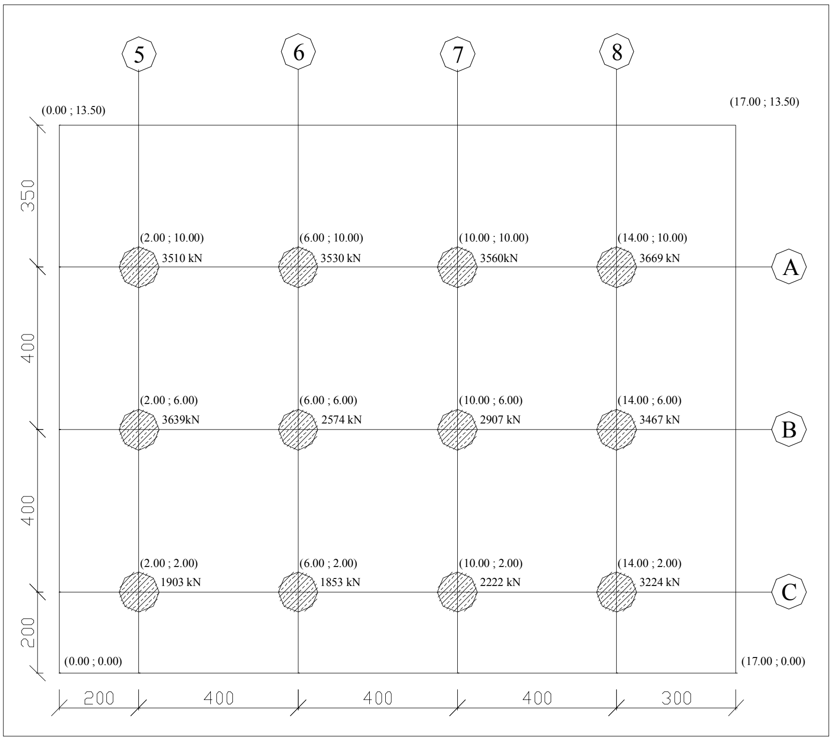

Figure 3.26 Pile disposition plan and pile loads (12 piles with 25m length)

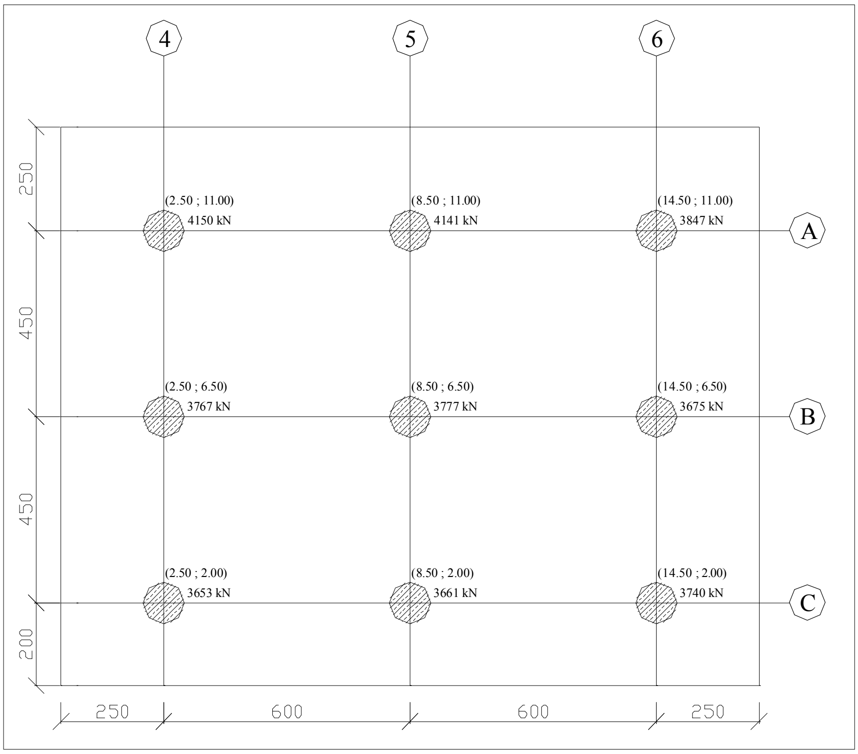

Figure 3.27 Pile disposition plan and pile loads (9 piles with 25m length)

The effective stresses distributions of piled foundation with different pile numbers are quite similar. The percentages of loads carried by piles are over 90% for different number of piles. Effective stress distribution of 30 piled foundations is given in Figure3.21. The effective stress distribution of piled foundation with 20, 12 and 9 piles are given in Figure 3.28, Figure 3.29 and Figure 3.30, respectively.

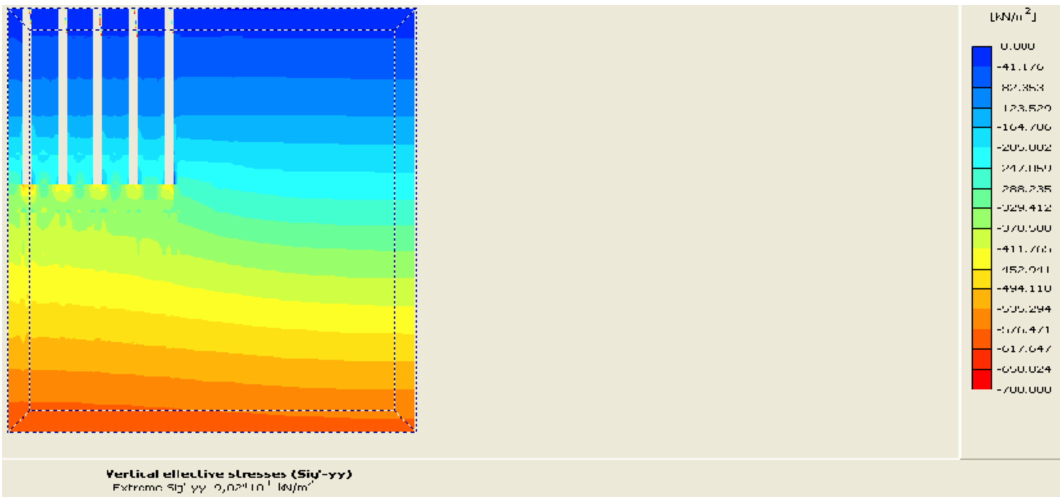

Figure 3.28 Vertical effective stresses (20 piles with 25m height)

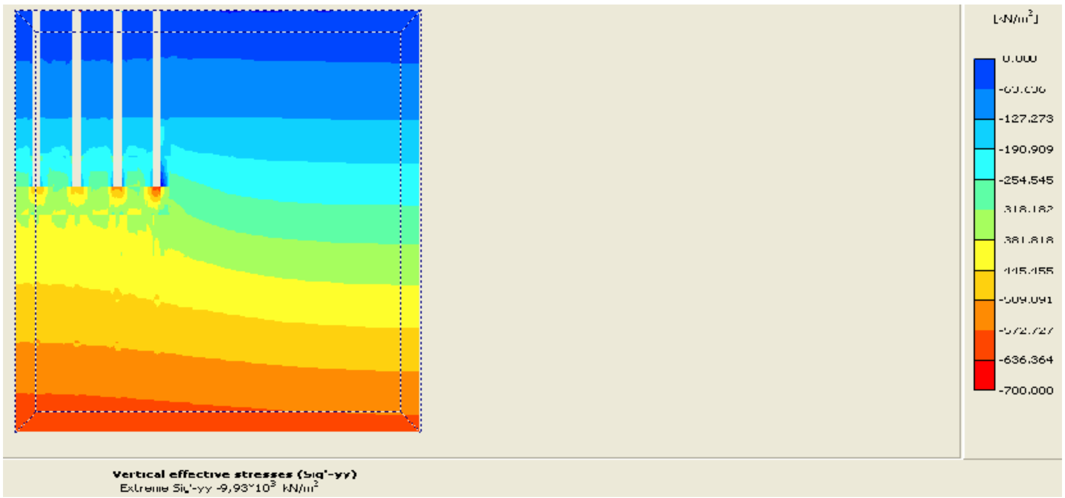

Figure 3.29 Vertical effective stresses (12 piles with 25m height)

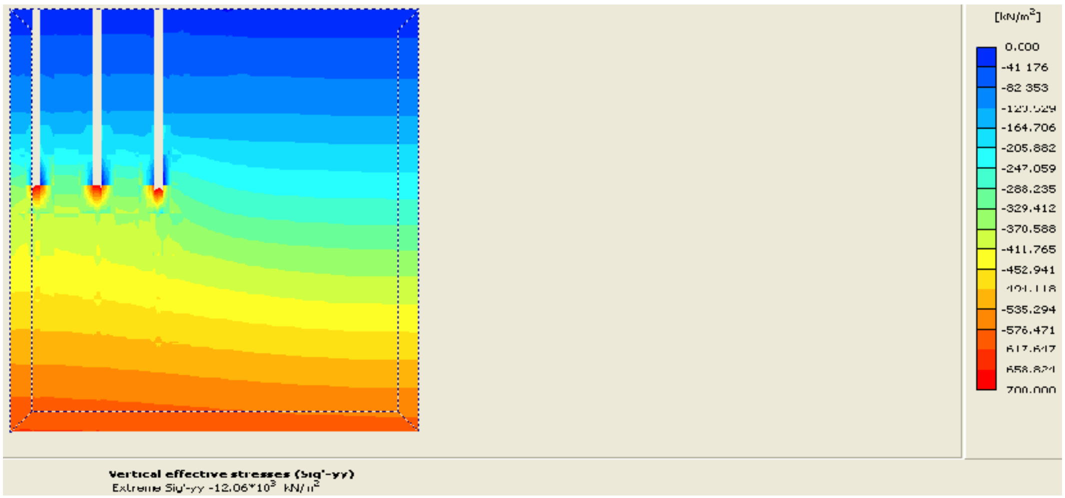

Figure 3.30 Vertical effective stresses (9 piles with 25m height)

It is seen that the investigation site soils are not readily suitable to design piled raft foundation. Soft layers existing between the top surface to approximately 14 meters depths do not provide significant resistance to structural loads. So the majority of the structural loads are carried by piles. Raft can provide a significant resistance in the case of soil profiles consisting of stiff clay and dense sand. If the investigation site soils are improved by means of one of the soil improvement methods then raft can provide enough resistance. Jet Grout is most preferable soil improvement method against the liquefaction risk and the settlement problems. Generally columns are used in a diameter of Ø60 – Ø80cm with a spacing of three times the diameter. Jet Grout columns are rigid columns in the soil profile. They result an increase of the elastic parameters of soil with the area ratio (a_r). a_r is defined as the ratio of the Jet Grout column area and the influence area of the Jet Grout column.

The elasticity modulus of soil changes according to the ratio a_r too. It is assumed that first 14 meters of the soil profile is improved by Jet Grout columns with $a_r = 0.10$. In the Plaxis model the improved soil properties were represented by means of elasticity modulus. In the calculation stages the improved soils are exchanged by the existing soft soils. The improved soil parameters are calculated as given below:

$$
\begin{aligned}
E_{JG} &= 6689000 kN/m^2 \\
a_r &= 0.10 \\
E_{sand1} &= 7150 kN/m^2 \\
E_{imp,sand1} &= 7150 \times 0.9 + 6689000 \times 0.1 \\
&= 6,7 \times 10^5 kN/m^2 \\
E_{clay} &= 5776 kN/m^2 \\
E_{imp,clay} &= 5776 \times 0.9 + 6689000 \times 0.1 \\
&= 6,7 \times 10^5 kN/m^2
\end{aligned}
$$

The foundation system with 12 and 9 piles were analyzed by improving the first 14 meters with Jet Grout columns. Pile loads are given in Figure 3.31 and Figure3.32.

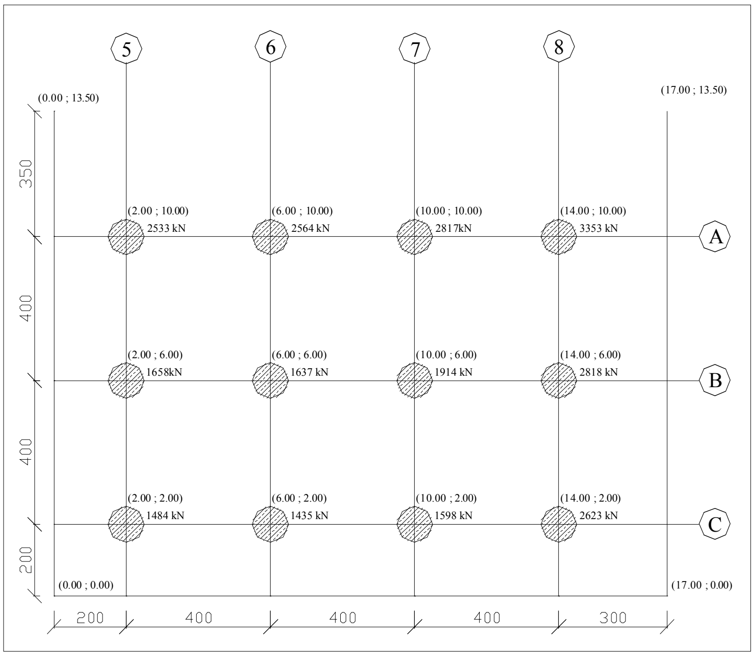

Figure 3.31 Pile disposition plan and pile loads (12 piles with 25m length) for the improved soil profile

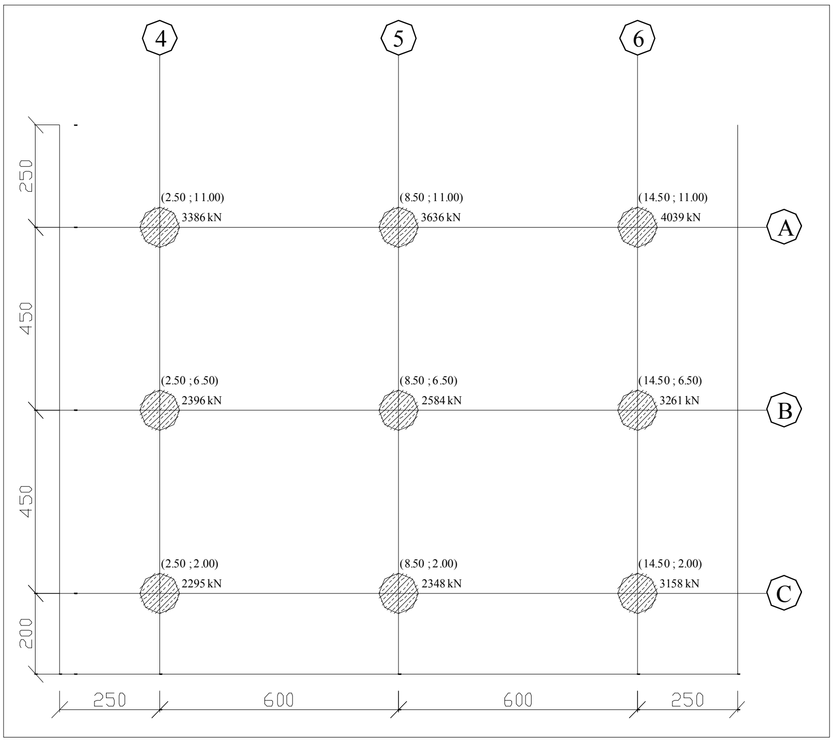

Figure 3.31 Pile disposition plan and pile loads (9 piles with 25m length) for the improved soil profile

The percentage of the load carried by piles in the foundation with 12 piles is $\beta = 0.72$ and in the foundation with 9 piles is $\beta = 0.71$. For both foundation systems the raft provided approximately 30% of the structural loads. The resistance of the raft can be seen in the shaded effective stress distribution diagram given in Figure 3.32 and in Figure 3.33. The maximum pile load on 12 piled system is 3353 kN and this load is 96% percent of the pile bearing capacity of a single pile. But in average piles are operating at 63% of the bearing capacity. Nonetheless the maximum pile load on 9 piled system is 4039 kN and this load is over the load bearing capacity. Although one of the pile is loaded over the load capacity, the piles are operating at 86% of the bearing capacity in average. The load sharing behavior can be adjusted by changing the pile disposition.

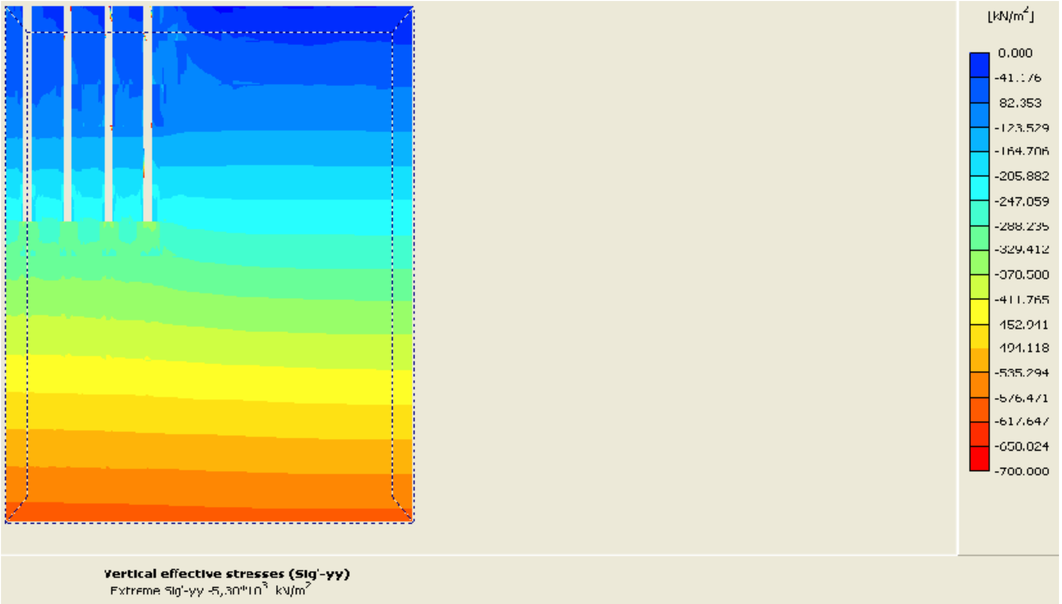

Figure 3.32 Vertical effective stresses (12 piles with 25m height) for the improved soil profile

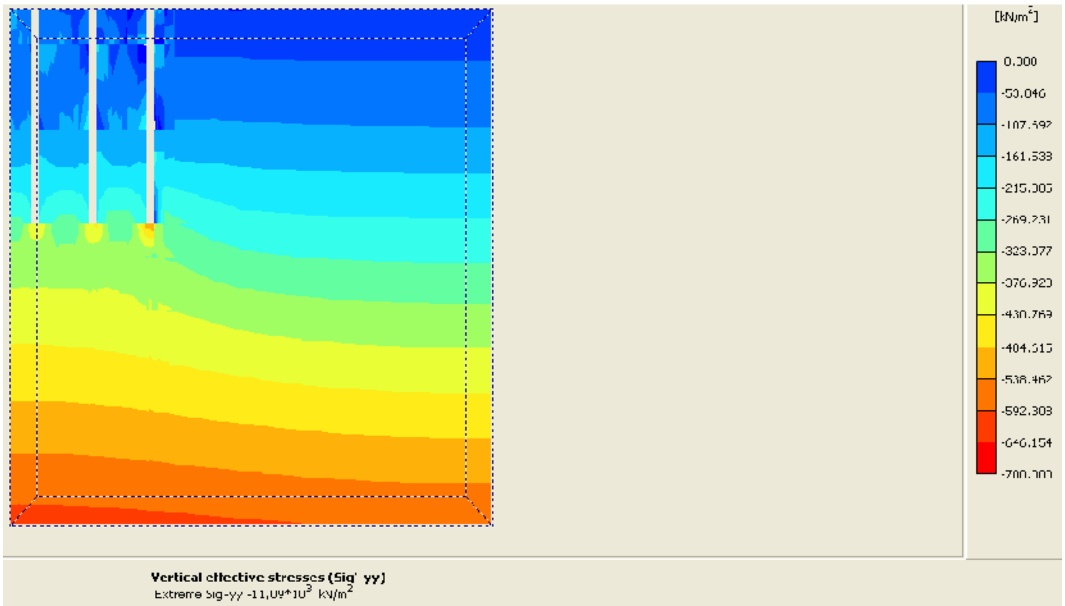

Figure 3.33 Vertical effective stresses (12 piles with 25m height) for the improved soil profile

3.4 The comparison of analysis results

The foundation design of a 16 story building in Mavişehir region of İzmir is studied in this chapter. The raft foundation, and piled foundation analyses were performed. In the first case hand calculation methods were used. Settlements were calculated by one dimensional consolidation theory and the maximum settlement of the raft is obtained as 80.8cm. Although in in-situ conditions the real settlement will be lesser than the calculated value because of three dimensional effects, it is not considered because of the great difference between the settlement and the settlement limits of rafts. The vertical settlement obtained from the finite element model is 55.8cm and this settlement value is logical according to the hand calculation method with using the three dimensional effects. The settlements of the raft and piled foundations are given in Table 3.9.

Table 3.19 The settlement of the raft, piled foundation and piled raft foundation analysis (30 piles with diameter of 1m, length of 35m with a spacing of 3m)

	raft foundation		conventional piled foundation	piled raft foundation	
	hand calculation method	finite element method		hand calculation method	finite element method
settlement (cm)	80.8	55.8	9.1	18.6	4.6

Because of settlement limits are over passed for the raft foundation, piled foundations are used for the deep foundation design. 30 piles with diameter of 1m, and length of 35m piles on 3 x 3 meter square mesh is used for design. The settlement for the conventional piled foundation is calculated as 9.1 cm by hand calculation methods. With the same pile disposition, the same diameter and the same length the piled raft foundation assumption is used by considering both the hand calculation and the finite element analysis. Because of the difficulty of calculating the stiffness values in the case of layered soil profile, only the parameters of the soft soil (CH) are considered. The settlement for the hand calculation is therefore calculated as 18.6cm. But it is known that the real settlements will be lesser then this. And at least the idealized soil model was analyzed with finite element method and the settlement is calculated as 4.6 cm for 30 piles with length of 35m, but it is seen that almost all the structural loads were carried by piles. This is because of the soft layers existing just under the building foundation.

Different variables such as pile length and pile numbers were studied to observe the load sharing behavior between the raft and piles. But the raft didn’t provide significant contribution in any of the analysis. Piled raft application in Mavişehir region of İzmir is not possible, because of the soft layers existing near the surface.

But these surfacial weak soil layers are usually improved against liquefaction risk. Also the loose sandy and silty layers existing in the investigation site has liquefaction risk. These soft layers are assumed to be improved by means of Jet Grout columns and piled foundation system is analyzed again 12 and 9 piles with the improved soil elasticity parameters. It is observed that the improved soils resisted to the settlement and carried approximately 30% of the total load. Piled raft application in this region of İzmir can only be possible with the improvement of surficial soft layers existing in the soil profile.

CHAPTER FOUR
RESULTS & CONCLUSIONS

Piled raft foundations are complicated problems and have to be designed by using appropriate computer programs. Although it is necessary to use a computer program, a simple hand calculation method is needed to check if computer solutions are logical or not. In this thesis the design philosophies of piled raft foundations are handled and a hand calculation method proposed by Poulos (2000) is studied. The worked example given in the same article is detailed. A finite element analyze program PLAXIS 3D 1.1 was used to analyze the same model used in the hand calculation example. It is seen that the results are quite similar. The idea of using the hand calculation method for preliminary design is logical. But the analyses were performed for a soil profile consists of single stiff clay layer. Also soil profiles consist of relatively stiff clays and relatively dense sand conditions were classified as favorable soil conditions by Poulos (1991). But there are some unfavorable soil conditions too, described by Poulos (1991) like soil profiles undergo consolidation settlement.

In the third chapter, the idea of using piled raft foundations in İzmir is assessed. Trough this aim a well known area in Mavişehir region is handled for a 16 stories building. To form the idealized soil profile, the entire field and laboratory test data given in the soil investigation report prepared by Ege Temel Sondajcılık with the investigation site was studied. The raft foundation analyses were performed for both hand calculation method and the finite element method. The settlements were calculated over the settlement limits. So deep foundation systems have assessed.

First the conventional piled foundation design is studied. Piles were designed with a diameter of 1m, length of 35m and with the square mesh spacing of 3m. The settlement of the foundation system was calculated as 9.1cm.

Piled raft assumption is also studied for the investigation site. The soil profile of the investigation site consists of stiff and soft layers. To avoid from the unfavorable effects of soft layers described by Poulos (1991) a disconnected pile model is used in piled raft hand calculation method. Thus the possible loses of contact pressure between the raft and soil would be prevented. Also there will be no shear and moment loads on the piles from the super structure. Piles will be operated as settlement reducers.

The piled raft hand calculation method has been performed for the idealized soil profile but it has been seen that the use of the hand calculation method was difficult in layered soil profiles because of the difficulty of defining the stiffness parameters. So the hand calculation method has been performed considering the soft clay layer only in the soil model.

Piled raft foundation analyses were also performed with finite element method. Although the settlement limits were met, almost all of the structural loads were carried by piles. By using pile length and numbers of piles as variables, different foundation models were analyzed in order to get a reasonable piled raft solution. But because of the soft layers existing just below the foundation level, the load carrying capacity provided by the raft has been limited for all runs. So that the piled raft application in Mavişehir region of İzmir has been evaluated as imposable with existing soil conditions. But it is a common practice to improve soils due to liquefaction risk by means of Jet Grout columns. Also the loose sandy and silty layers existing in the investigation site has liquefaction risk. By using Jet Grout columns with the piles, effective parameters of the soil layers are also improved. Analyses have been performed with the improved soil properties. It is observed that the raft provided approximately 30% of the total load for an area ratio of a_r= 0.10 for jet-grout columns, while piles were working at 70-90 % of the load bearing capacity.

Different foundation models were studied for a 16 stories building in Mavişehir region of İzmir. It is concluded that the piled raft approach can be achieved with the improvement of soft and loose soils near the surface.

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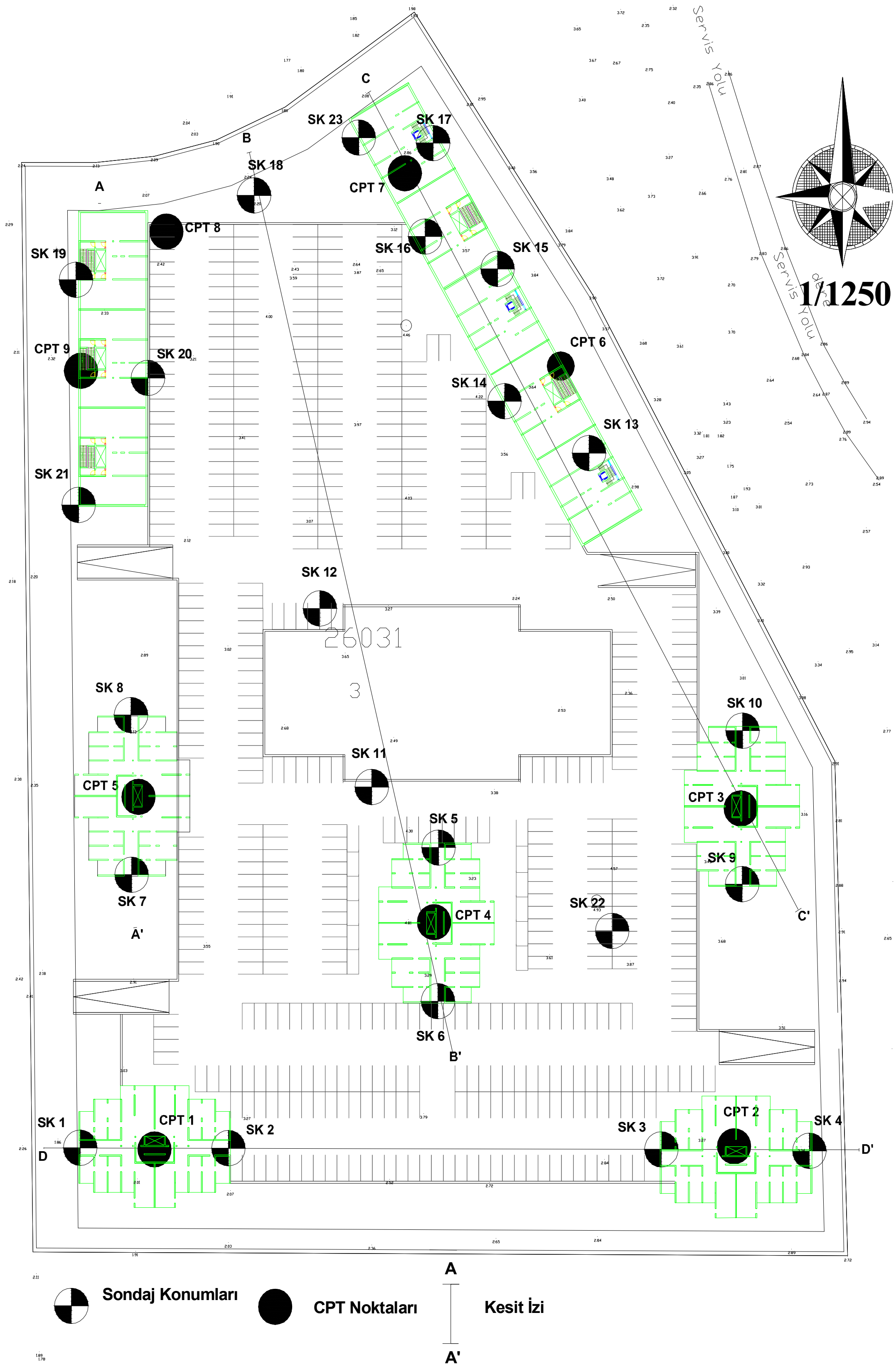
APPENDICES

APPENDICE A
THE APPLICATION PLAN OF IN-SITU TESTS AND STRUCTURES



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APPENDICE B
SPT (STANDART PENETRATION TEST) CORRECTIONS

BH-1	**gwl (m)**	3.15	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
SILTY SAND	3.30	59.7	58.2	10	1.23	0.75	1	1	1.2	11.1
SILTY CLAY	4.80	89.7	73.2	12	1.00	0.75	1	1	1.2	10.8
SM	6.60	125.7	91.2	15	1.04	0.75	1	1	1.2	14.1
SM	7.80	149.7	103.2	16	0.99	0.75	1	1	1.2	14.2
MH	9.30	179.7	118.2	6	1.00	0.75	1	1	1.2	5.4
MH	10.80	209.7	133.2	6	1.00	0.75	1	1	1.2	5.4
MH	12.80	249.7	153.2	5	1.00	0.75	1	1	1.2	4.5
MH	13.80	269.7	163.2	5	1.00	0.75	1	1	1.2	4.5
SM	15.30	299.7	178.2	8	0.74	0.75	1	1	1.2	5.3
CLAY	16.80	329.7	193.2	5	1.00	0.75	1	1	1.2	4.5
CLAY	18.80	369.7	213.2	6	1.00	0.75	1	1	1.2	5.4
CLAY	19.80	389.7	223.2	8	1.00	0.75	1	1	1.2	7.2
CLAYEY SAND	21.30	419.7	238.2	11	0.61	0.75	1	1	1.2	6.1
GRAVELLY CLAY	22.80	449.7	253.2	20	1.00	0.75	1	1	1.2	18.0
GRAVELLY CLAY	24.30	479.7	268.2	16	1.00	0.75	1	1	1.2	14.4
GRAVELLY CLAY	25.80	509.7	283.2	21	1.00	0.75	1	1	1.2	18.9
GRAVELLY CLAY	27.30	539.7	298.2	26	1.00	0.75	1	1	1.2	23.4
GRAVELLY CLAY	28.80	569.7	313.2	31	1.00	0.75	1	1	1.2	27.9
SC	30.30	599.7	328.2	34	0.49	0.75	1	1	1.2	15.0
SC	31.80	629.7	343.2	32	0.47	0.75	1	1	1.2	13.7
CLAYEY GRAVEL	33.30	659.7	358.2	50	0.46	0.75	1	1	1.2	20.7
CLAYEY GRAVEL	34.80	689.7	373.2	33	0.45	0.75	1	1	1.2	13.2
CLAYEY GRAVEL	36.30	719.7	388.2	38	0.43	0.75	1	1	1.2	14.8
CLAYEY GRAVEL	37.54	744.5	400.6	R	0.42	0.75	1	1	1.2	R
CLAYEY GRAVEL	39.05	774.7	415.7	R	0.41	0.75	1	1	1.2	R

BH-2	**gwl (m)**	3.15	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.05	10.0	54.9	R	1.26	0.75	1	1	1.2	R
ML	4.80	89.7	73.2	3	1.14	0.75	1	1	1.2	3.1
SM	6.30	119.7	88.2	12	1.06	0.75	1	1	1.2	11.4
SM	7.80	149.7	103.2	13	0.99	0.75	1	1	1.2	11.5
SM	9.30	179.7	118.2	14	0.92	0.75	1	1	1.2	11.6
CLAY	10.80	209.7	133.2	3	1.00	0.75	1	1	1.2	2.7
CLAY	12.30	239.7	148.2	3	1.00	0.75	1	1	1.2	2.7
CLAY	13.80	269.7	163.2	4	1.00	0.75	1	1	1.2	3.6
SILTY CLAY	15.30	299.7	178.2	7	1.00	0.75	1	1	1.2	6.3
SILTY CLAY	16.80	329.7	193.2	10	1.00	0.75	1	1	1.2	9.0
SILTY CLAY	18.30	359.7	208.2	8	1.00	0.75	1	1	1.2	7.2
SC	19.54	384.5	220.6	R	0.65	0.75	1	1	1.2	R
SC	21.30	419.7	238.2	30	0.61	0.75	1	1	1.2	16.6
GRAVELLY CLAY	22.80	449.7	253.2	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	24.30	479.7	268.2	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	25.80	509.7	283.2	32	1.00	0.75	1	1	1.2	28.8
GRAVELLY CLAY	27.30	539.7	298.2	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	28.80	569.7	313.2	28	1.00	0.75	1	1	1.2	25.2
SC	30.30	599.7	328.2	28	0.49	0.75	1	1	1.2	12.4
SC	31.80	629.7	343.2	31	0.47	0.75	1	1	1.2	13.3
SC	33.30	659.7	358.2	31	0.46	0.75	1	1	1.2	12.8
GC	34.52	684.1	370.4	R	0.45	0.75	1	1	1.2	R
GC	36.00	713.7	385.2	R	0.44	0.75	1	1	1.2	R
GC	37.52	744.1	400.4	R	0.42	0.75	1	1	1.2	R
GC	40.03	794.3	425.5	R	0.40	0.75	1	1	1.2	R

BH-3	**gwl (m)**	3.3	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
SILTY CLAY	4.80	89.4	74.4	3	1.00	0.75	1	1	1.2	2.7
SILTY CLAY	6.30	119.4	89.4	3	1.00	0.75	1	1	1.2	2.7
SM	7.80	149.4	104.4	14	0.98	0.75	1	1	1.2	12.4
SM	9.30	179.4	119.4	15	0.92	0.75	1	1	1.2	12.4
ML	10.80	209.4	134.4	12	0.86	0.75	1	1	1.2	9.3
CH	12.80	249.4	154.4	17	1.00	0.75	1	1	1.2	15.3
CH	13.80	269.4	164.4	19	1.00	0.75	1	1	1.2	17.1
CH	15.30	299.4	179.4	21	1.00	0.75	1	1	1.2	18.9
CH	16.80	329.4	194.4	9	1.00	0.75	1	1	1.2	8.1
CH	18.80	369.4	214.4	6	1.00	0.75	1	1	1.2	5.4
CH	19.80	389.4	224.4	7	1.00	0.75	1	1	1.2	6.3
CH	21.30	419.4	239.4	4	1.00	0.75	1	1	1.2	3.6
GRAVELLY CLAY	22.80	449.4	254.4	22	1.00	0.75	1	1	1.2	19.8
SANDY GRAVEL	24.30	479.4	269.4	22	0.56	0.75	1	1	1.2	11.2
SANDY GRAVEL	25.80	509.4	284.4	30	0.54	0.75	1	1	1.2	14.7
GM	27.30	539.4	299.4	58	0.52	0.75	1	1	1.2	27.4
GM	28.80	569.4	314.4	R	0.51	0.75	1	1	1.2	R
GM	30.30	599.4	329.4	R	0.49	0.75	1	1	1.2	R
GM	31.80	629.4	344.4	R	0.47	0.75	1	1	1.2	R
CLAYEY GRAVEL	33.30	659.4	359.4	R	0.46	0.75	1	1	1.2	R
CLAYEY GRAVEL	34.80	689.4	374.4	R	0.44	0.75	1	1	1.2	R
CLAYEY GRAVEL	36.30	719.4	389.4	R	0.43	0.75	1	1	1.2	R
CLAYEY GRAVEL	37.54	744.2	401.8	R	0.42	0.75	1	1	1.2	R
CLAYEY GRAVEL	39.05	774.4	416.9	R	0.41	0.75	1	1	1.2	R

BH-4	gwl (m)	3.4	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.2	72.2	R	1.14	0.75	1	1	1.2	R
SILT	6.30	119.2	90.2	3	1.05	0.75	1	1	1.2	2.8
SILT	7.80	149.2	105.2	4	0.98	0.75	1	1	1.2	3.5
SILT	9.30	179.2	120.2	7	0.92	0.75	1	1	1.2	5.8
SILT	10.80	209.2	135.2	10	0.86	0.75	1	1	1.2	7.8
SILT	12.30	239.2	150.2	9	0.81	0.75	1	1	1.2	6.6
SILT	13.80	269.2	165.2	12	0.77	0.75	1	1	1.2	8.3
CLAY	15.80	309.2	185.2	12	1.00	0.75	1	1	1.2	10.8
CLAY	16.80	329.2	195.2	13	1.00	0.75	1	1	1.2	11.7
GRAVELLY CLAY	18.30	359.2	210.2	14	1.00	0.75	1	1	1.2	12.6
GRAVELLY CLAY	19.80	389.2	225.2	19	1.00	0.75	1	1	1.2	17.1
GRAVELLY CLAY	21.30	419.2	240.2	25	1.00	0.75	1	1	1.2	22.5
GRAVELLY SAND	22.80	449.2	255.2	25	0.59	0.75	1	1	1.2	13.2
GRAVELLY CLAY	24.30	479.2	270.2	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	25.80	509.2	285.2	34	1.00	0.75	1	1	1.2	30.6
GRAVELLY CLAY	27.30	539.2	300.2	41	1.00	0.75	1	1	1.2	36.9
GRAVELLY CLAY	28.80	569.2	315.2	35	1.00	0.75	1	1	1.2	31.5
GRAVELLY CLAY	30.30	599.2	330.2	37	1.00	0.75	1	1	1.2	33.3
GRAVELLY CLAY	31.80	629.2	345.2	36	1.00	0.75	1	1	1.2	32.4
GRAVELLY CLAY	33.00	653.2	357.2	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	34.80	689.2	375.2	41	1.00	0.75	1	1	1.2	36.9
SM	36.00	713.2	387.2	R	0.43	0.75	1	1	1.2	R
SM	37.50	743.2	402.2	R	0.42	0.75	1	1	1.2	R
SANDY GRAVEL	39.00	773.2	417.2	R	0.41	0.75	1	1	1.2	R

BH-5	gwl (m)	3.4	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.2	72.2	R	1.14	0.75	1	1	1.2	R
SM	6.30	119.2	90.2	15	1.05	0.75	1	1	1.2	14.1
SM	7.80	149.2	105.2	14	0.98	0.75	1	1	1.2	12.3
SM	9.30	179.2	120.2	20	0.92	0.75	1	1	1.2	16.5
SANDY SILT	10.80	209.2	135.2	12	0.86	0.75	1	1	1.2	9.3
SANDY SILT	12.80	249.2	155.2	14	0.80	0.75	1	1	1.2	10.1
SILTY CLAY	13.80	269.2	165.2	11	1.00	0.75	1	1	1.2	9.9
CH	15.80	309.2	185.2	10	1.00	0.75	1	1	1.2	9.0
CH	16.80	329.2	195.2	9	1.00	0.75	1	1	1.2	8.1
CH	18.30	359.2	210.2	12	1.00	0.75	1	1	1.2	10.8
CH	19.80	389.2	225.2	14	1.00	0.75	1	1	1.2	12.6
SILTY CLAY	21.30	419.2	240.2	18	1.00	0.75	1	1	1.2	16.2
GRAVELLY CLAY	22.80	449.2	255.2	22	1.00	0.75	1	1	1.2	19.8
GRAVELLY CLAY	24.30	479.2	270.2	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	25.80	509.2	285.2	24	1.00	0.75	1	1	1.2	21.6
GRAVELLY CLAY	27.30	539.2	300.2	28	1.00	0.75	1	1	1.2	25.2
CL	28.80	569.2	315.2	30	1.00	0.75	1	1	1.2	27.0
CL	30.30	599.2	330.2	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	31.80	629.2	345.2	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	33.30	659.2	360.2	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	34.80	689.2	375.2	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	36.30	719.2	390.2	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	37.80	749.2	405.2	34	1.00	0.75	1	1	1.2	30.6
SC	39.30	779.2	420.2	36	0.41	0.75	1	1	1.2	13.2

BH-6	gwl (m)	3.5	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.0	73.0	R	1.14	0.75	1	1	1.2	R
SILT	6.30	119.0	91.0	2	1.04	0.75	1	1	1.2	1.9
SILT	7.90	151.0	107.0	11	0.97	0.75	1	1	1.2	9.6
SILT	9.30	179.0	121.0	15	0.91	0.75	1	1	1.2	12.3
SILT	10.80	209.0	136.0	20	0.86	0.75	1	1	1.2	15.5
SILT	12.30	239.0	151.0	20	0.81	0.75	1	1	1.2	14.6
SILTY CLAY	13.80	269.0	166.0	10	1.00	0.75	1	1	1.2	9.0
SILTY CLAY	15.30	299.0	181.0	8	1.00	0.75	1	1	1.2	7.2
SILTY CLAY	16.80	329.0	196.0	10	1.00	0.75	1	1	1.2	9.0
CLAYEY SILT	18.30	359.0	211.0	11	1.00	0.75	1	1	1.2	9.9
SILTY CLAY	19.80	389.0	226.0	12	1.00	0.75	1	1	1.2	10.8
SILTY CLAY	21.30	419.0	241.0	17	1.00	0.75	1	1	1.2	15.3
GRAVELLY CLAY	22.80	449.0	256.0	23	1.00	0.75	1	1	1.2	20.7
SM	24.30	479.0	271.0	24	0.56	0.75	1	1	1.2	12.2
SM	25.80	509.0	286.0	23	0.54	0.75	1	1	1.2	11.2
SM	27.30	539.0	301.0	18	0.52	0.75	1	1	1.2	8.5
SM	28.80	569.0	316.0	22	0.50	0.75	1	1	1.2	10.0
SM	30.30	599.0	331.0	24	0.49	0.75	1	1	1.2	10.5
SM	31.80	629.0	346.0	31	0.47	0.75	1	1	1.2	13.2
SM	33.30	659.0	361.0	37	0.46	0.75	1	1	1.2	15.2
SANDY GRAVEL	34.80	689.0	376.0	44	0.44	0.75	1	1	1.2	17.6
SANDY GRAVEL	36.00	713.0	388.0	R	0.43	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	743.0	403.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	773.0	418.0	R	1.00	0.75	1	1	1.2	R

BH-7	**gwl (m)**	4.3	**γ (kN/m^3)**	18	**$γ_{sat}$ (kN/m^3)**	20	**$γ_w$ (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**$σ_{vo}$ (kPa)**	**$σ_{vo}$' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
SM	4.80	87.4	82.4	12	1.09	0.75	1	1	1.2	11.7
SM	6.80	127.4	102.4	16	0.99	0.75	1	1	1.2	14.2
CH	7.80	147.4	112.4	9	1.00	0.75	1	1	1.2	8.1
CH	9.30	177.4	127.4	6	1.00	0.75	1	1	1.2	5.4
CH	10.80	207.4	142.4	4	1.00	0.75	1	1	1.2	3.6
CH	12.80	247.4	162.4	4	1.00	0.75	1	1	1.2	3.6
CH	13.80	267.4	172.4	5	1.00	0.75	1	1	1.2	4.5
CH	15.30	297.4	187.4	6	1.00	0.75	1	1	1.2	5.4
CH	16.80	327.4	202.4	6	1.00	0.75	1	1	1.2	5.4
CH	18.80	367.4	222.4	7	1.00	0.75	1	1	1.2	6.3
CH	19.80	387.4	232.4	10	1.00	0.75	1	1	1.2	9.0
CH	21.30	417.4	247.4	11	1.00	0.75	1	1	1.2	9.9
CH	22.80	447.4	262.4	12	1.00	0.75	1	1	1.2	10.8
GRAVELLY CLAY	24.30	477.4	277.4	17	1.00	0.75	1	1	1.2	15.3
GRAVELLY CLAY	25.80	507.4	292.4	21	1.00	0.75	1	1	1.2	18.9
GRAVELLY CLAY	27.30	537.4	307.4	25	1.00	0.75	1	1	1.2	22.5
GRAVELLY CLAY	28.80	567.4	322.4	28	1.00	0.75	1	1	1.2	25.2
GRAVELLY CLAY	30.30	597.4	337.4	34	1.00	0.75	1	1	1.2	30.6
GRAVELLY CLAY	31.80	627.4	352.4	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	33.30	657.4	367.4	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	34.80	687.4	382.4	33	1.00	0.75	1	1	1.2	29.7
GM	36.30	717.4	397.4	40	0.43	0.75	1	1	1.2	15.3
GM	37.54	742.2	409.8	R	0.42	0.75	1	1	1.2	R
GM	39.00	771.4	424.4	R	0.40	0.75	1	1	1.2	R

BH-8	gwl (m)	3.50	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.0	73.0	R	1.14	0.75	1	1	1.2	R
SP-SM	6.30	119.0	91.0	13	1.04	0.75	1	1	1.2	12.2
SP-SM	7.80	149.0	106.0	14	0.97	0.75	1	1	1.2	12.3
CLAY	9.30	179.0	121.0	4	1.00	0.75	1	1	1.2	3.6
CLAY	10.80	209.0	136.0	4	1.00	0.75	1	1	1.2	3.6
CLAY	12.30	239.0	151.0	5	1.00	0.75	1	1	1.2	4.5
SM	13.80	269.0	166.0	12	0.77	0.75	1	1	1.2	8.3
CLAYEY SILT	15.30	299.0	181.0	6	1.00	0.75	1	1	1.2	5.4
CLAY	16.80	329.0	196.0	5	1.00	0.75	1	1	1.2	4.5
CLAY	18.30	359.0	211.0	2	1.00	0.75	1	1	1.2	1.8
CLAYEY GRAVEL	19.50	383.0	223.0	R	0.64	0.75	1	1	1.2	R
GRAVELLY CLAY	21.30	419.0	241.0	12	1.00	0.75	1	1	1.2	10.8
GRAVELLY CLAY	22.80	449.0	256.0	28	1.00	0.75	1	1	1.2	25.2
CLAY	24.30	479.0	271.0	23	1.00	0.75	1	1	1.2	20.7
CLAY	25.80	509.0	286.0	26	1.00	0.75	1	1	1.2	23.4
CL	27.30	539.0	301.0	27	1.00	0.75	1	1	1.2	24.3
CL	28.80	569.0	316.0	26	1.00	0.75	1	1	1.2	23.4
CL	30.30	599.0	331.0	29	1.00	0.75	1	1	1.2	26.1
CL	31.80	629.0	346.0	32	1.00	0.75	1	1	1.2	28.8
CL	33.30	659.0	361.0	32	1.00	0.75	1	1	1.2	28.8
GRAVELLY CLAY	34.50	683.0	373.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	36.00	713.0	388.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	743.0	403.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	773.0	418.0	R	1.00	0.75	1	1	1.2	R

BH-9	**gwl (m)**	3.3	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.4	71.4	R	1.15	0.75	1	1	1.2	R
ML	6.30	119.4	89.4	5	1.05	0.75	1	1	1.2	4.7
ML	7.80	149.4	104.4	7	0.98	0.75	1	1	1.2	6.2
SILTY SAND	9.30	179.4	119.4	11	0.92	0.75	1	1	1.2	9.1
CH	11.30	219.4	139.4	12	1.00	0.75	1	1	1.2	10.8
CH	12.30	239.4	149.4	11	1.00	0.75	1	1	1.2	9.9
CH	13.80	269.4	164.4	9	1.00	0.75	1	1	1.2	8.1
CH	15.30	299.4	179.4	12	1.00	0.75	1	1	1.2	10.8
CH	16.80	329.4	194.4	6	1.00	0.75	1	1	1.2	5.4
CH	18.30	359.4	209.4	7	1.00	0.75	1	1	1.2	6.3
SM	19.80	389.4	224.4	17	0.64	0.75	1	1	1.2	9.8
GRAVELLY CLAY	21.30	419.4	239.4	23	1.00	0.75	1	1	1.2	20.7
GRAVELLY CLAY	22.80	449.4	254.4	22	1.00	0.75	1	1	1.2	19.8
GRAVELLY CLAY	24.30	479.4	269.4	24	1.00	0.75	1	1	1.2	21.6
GRAVELLY CLAY	25.80	509.4	284.4	24	1.00	0.75	1	1	1.2	21.6
GRAVELLY CLAY	27.30	539.4	299.4	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	28.80	569.4	314.4	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	30.30	599.4	329.4	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	31.80	629.4	344.4	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	33.30	659.4	359.4	40	1.00	0.75	1	1	1.2	36.0
GRAVELLY CLAY	34.80	689.4	374.4	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	36.30	719.4	389.4	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.54	744.2	401.8	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.05	774.4	416.9	R	1.00	0.75	1	1	1.2	R

BH-10	gwl (m)	3.2	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.6	70.6	R	1.15	0.75	1	1	1.2	R
CLAYEY SILT	6.30	119.6	88.6	14	1.00	0.75	1	1	1.2	12.6
CLAYEY SILT	7.80	149.6	103.6	18	1.00	0.75	1	1	1.2	16.2
ML	9.30	179.6	118.6	22	0.92	0.75	1	1	1.2	18.3
SILTY CLAY	10.80	209.6	133.6	5	1.00	0.75	1	1	1.2	4.5
SILTY CLAY	12.30	239.6	148.6	5	1.00	0.75	1	1	1.2	4.5
SILTY CLAY	13.80	269.6	163.6	7	1.00	0.75	1	1	1.2	6.3
SILTY CLAY	15.30	299.6	178.6	10	1.00	0.75	1	1	1.2	9.0
SILTY CLAY	16.80	329.6	193.6	10	1.00	0.75	1	1	1.2	9.0
SILTY CLAY	18.30	359.6	208.6	13	1.00	0.75	1	1	1.2	11.7
SANDY GRAVEL	19.80	389.6	223.6	30	0.64	0.75	1	1	1.2	17.3
SANDY GRAVEL	21.30	419.6	238.6	32	0.61	0.75	1	1	1.2	17.7
CLAYEY GRAVEL	22.80	449.6	253.6	24	0.59	0.75	1	1	1.2	12.7
CL	24.30	479.6	268.6	31	1.00	0.75	1	1	1.2	27.9
CL	25.80	509.6	283.6	26	1.00	0.75	1	1	1.2	23.4
CL	27.30	539.6	298.6	29	1.00	0.75	1	1	1.2	26.1
CL	28.80	569.6	313.6	31	1.00	0.75	1	1	1.2	27.9
CL	30.30	599.6	328.6	35	1.00	0.75	1	1	1.2	31.5
GRAVELLY CLAY	31.80	629.6	343.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	33.30	659.6	358.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	34.80	689.6	373.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	36.30	719.6	388.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.54	744.4	401.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.05	774.6	416.1	R	1.00	0.75	1	1	1.2	R

BH-11	**gwl (m)**	3.2	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.6	70.6	R	1.15	0.75	1	1	1.2	R
SM	6.30	119.6	88.6	17	1.05	0.75	1	1	1.2	16.1
SM	7.80	149.6	103.6	19	0.98	0.75	1	1	1.2	16.8
SILTY CLAY	9.30	179.6	118.6	6	1.00	0.75	1	1	1.2	5.4
SILTY CLAY	10.80	209.6	133.6	8	1.00	0.75	1	1	1.2	7.2
SILTY CLAY	12.30	239.6	148.6	4	1.00	0.75	1	1	1.2	3.6
SILTY CLAY	13.80	269.6	163.6	9	1.00	0.75	1	1	1.2	8.1
SILTY CLAY	15.30	299.6	178.6	6	1.00	0.75	1	1	1.2	5.4
SILTY CLAY	16.80	329.6	193.6	7	1.00	0.75	1	1	1.2	6.3
SILTY CLAY	18.30	359.6	208.6	3	1.00	0.75	1	1	1.2	2.7
SM	19.80	389.6	223.6	25	0.64	0.75	1	1	1.2	14.4
CLAYEY GRAVEL	21.00	413.6	235.6	R	0.62	0.75	1	1	1.2	R
GRAVELLY CLAY	22.80	449.6	253.6	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	24.30	479.6	268.6	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	25.80	509.6	283.6	37	1.00	0.75	1	1	1.2	33.3
GRAVELLY CLAY	27.30	539.6	298.6	46	1.00	0.75	1	1	1.2	41.4
GC	28.50	563.6	310.6	R	0.51	0.75	1	1	1.2	R
GC	30.00	593.6	325.6	R	0.49	0.75	1	1	1.2	R
GC	31.50	623.6	340.6	R	0.48	0.75	1	1	1.2	R
CLAYEY GRAVEL	33.00	653.6	355.6	R	0.46	0.75	1	1	1.2	R
CLAYEY GRAVEL	34.50	683.6	370.6	R	0.45	0.75	1	1	1.2	R
CLAYEY GRAVEL	36.00	713.6	385.6	R	0.44	0.75	1	1	1.2	R
CLAYEY GRAVEL	37.50	743.6	400.6	R	0.42	0.75	1	1	1.2	R
CLAYEY GRAVEL	39.00	773.6	415.6	R	0.41	0.75	1	1	1.2	R

BH-12	**gwl (m)**	3.15	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.30	59.7	58.2	R	1.23	0.75	1	1	1.2	R
FILL	4.80	89.7	73.2	R	1.14	0.75	1	1	1.2	R
SILTY CLAY	6.60	125.7	91.2	16	1.00	0.75	1	1	1.2	14.4
SM	7.80	149.7	103.2	17	0.99	0.75	1	1	1.2	15.1
CLAY	9.30	179.7	118.2	14	1.00	0.75	1	1	1.2	12.6
CLAY	10.80	209.7	133.2	9	1.00	0.75	1	1	1.2	8.1
CLAY	12.80	249.7	153.2	6	1.00	0.75	1	1	1.2	5.4
CLAY	13.80	269.7	163.2	4	1.00	0.75	1	1	1.2	3.6
CLAY	15.30	299.7	178.2	4	1.00	0.75	1	1	1.2	3.6
SILTY CLAY	16.80	329.7	193.2	9	1.00	0.75	1	1	1.2	8.1
SILTY CLAY	18.80	369.7	213.2	9	1.00	0.75	1	1	1.2	8.1
SANDY CLAY	19.80	389.7	223.2	18	1.00	0.75	1	1	1.2	16.2
SM	21.30	419.7	238.2	20	0.61	0.75	1	1	1.2	11.1
SM	22.80	449.7	253.2	22	0.59	0.75	1	1	1.2	11.7
CL	24.30	479.7	268.2	22	1.00	0.75	1	1	1.2	19.8
CL	25.80	509.7	283.2	25	1.00	0.75	1	1	1.2	22.5
CL	27.30	539.7	298.2	24	1.00	0.75	1	1	1.2	21.6
CL	28.80	569.7	313.2	27	1.00	0.75	1	1	1.2	24.3
SANDY GRAVEL	30.30	599.7	328.2	R	0.49	0.75	1	1	1.2	R
SANDY GRAVEL	31.80	629.7	343.2	R	0.47	0.75	1	1	1.2	R
SANDY GRAVEL	33.30	659.7	358.2	R	0.46	0.75	1	1	1.2	R
SANDY GRAVEL	34.80	689.7	373.2	R	0.45	0.75	1	1	1.2	R
CLAYEY GRAVEL	36.30	719.7	388.2	R	0.43	0.75	1	1	1.2	R
CLAYEY GRAVEL	37.54	744.5	400.6	R	0.42	0.75	1	1	1.2	R
CLAYEY GRAVEL	39.05	774.7	415.7	R	0.41	0.75	1	1	1.2	R

BH-13	**gwl (m)**	3.4	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
MH	4.80	89.2	75.2	15	1.00	0.75	1	1	1.2	13.5
MH	6.30	119.2	90.2	12	1.00	0.75	1	1	1.2	10.8
SILTY CLAY	7.80	149.2	105.2	6	1.00	0.75	1	1	1.2	5.4
SILTY CLAY	9.30	179.2	120.2	7	1.00	0.75	1	1	1.2	6.3
SILTY CLAY	10.80	209.2	135.2	4	1.00	0.75	1	1	1.2	3.6
SILTY CLAY	12.30	239.2	150.2	3	1.00	0.75	1	1	1.2	2.7
SILTY CLAY	13.80	269.2	165.2	5	1.00	0.75	1	1	1.2	4.5
SILTY CLAY	15.30	299.2	180.2	4	1.00	0.75	1	1	1.2	3.6
SILTY CLAY	16.80	329.2	195.2	5	1.00	0.75	1	1	1.2	4.5
SILTY CLAY	18.30	359.2	210.2	6	1.00	0.75	1	1	1.2	5.4
SM	19.80	389.2	225.2	17	0.64	0.75	1	1	1.2	9.8
GRAVELLY CLAY	21.30	419.2	240.2	20	1.00	0.75	1	1	1.2	18.0
GRAVELLY CLAY	22.80	449.2	255.2	26	1.00	0.75	1	1	1.2	23.4
GRAVELLY CLAY	24.30	479.2	270.2	28	1.00	0.75	1	1	1.2	25.2
GRAVELLY CLAY	25.80	509.2	285.2	28	1.00	0.75	1	1	1.2	25.2
CLAYEY GRAVEL	27.30	539.2	300.2	31	0.52	0.75	1	1	1.2	14.6
CLAYEY GRAVEL	28.80	569.2	315.2	35	0.51	0.75	1	1	1.2	15.9
CLAYEY GRAVEL	30.30	599.2	330.2	38	0.49	0.75	1	1	1.2	16.7
GC	31.50	623.2	342.2	R	0.48	0.75	1	1	1.2	R
GC	33.00	653.2	357.2	R	0.46	0.75	1	1	1.2	R
GC	34.50	683.2	372.2	R	0.45	0.75	1	1	1.2	R
GC	36.00	713.2	387.2	R	0.43	0.75	1	1	1.2	R
GC	37.50	743.2	402.2	R	0.42	0.75	1	1	1.2	R
GC	39.00	773.2	417.2	R	0.41	0.75	1	1	1.2	R

BH-14	**gwl (m)**	3.3	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
CLAYEY SILT	4.80	89.4	74.4	10	1.00	0.75	1	1	1.2	9.0
SM	6.30	119.4	89.4	29	1.05	0.75	1	1	1.2	27.4
SM	7.80	149.4	104.4	11	0.98	0.75	1	1	1.2	9.7
CLAY	9.30	179.4	119.4	7	1.00	0.75	1	1	1.2	6.3
CLAY	10.80	209.4	134.4	8	1.00	0.75	1	1	1.2	7.2
CLAY	12.30	239.4	149.4	5	1.00	0.75	1	1	1.2	4.5
CLAY	13.80	269.4	164.4	3	1.00	0.75	1	1	1.2	2.7
CLAY	15.30	299.4	179.4	6	1.00	0.75	1	1	1.2	5.4
CLAY	16.80	329.4	194.4	4	1.00	0.75	1	1	1.2	3.6
CLAY	18.30	359.4	209.4	5	1.00	0.75	1	1	1.2	4.5
CLAYEY SAND	19.50	383.4	221.4	R	0.64	0.75	1	1	1.2	R
CH	21.30	419.4	239.4	20	1.00	0.75	1	1	1.2	18.0
CH	22.80	449.4	254.4	24	1.00	0.75	1	1	1.2	21.6
CH	24.30	479.4	269.4	26	1.00	0.75	1	1	1.2	23.4
GRAVELLY CLAY	25.80	509.4	284.4	21	1.00	0.75	1	1	1.2	18.9
GRAVELLY CLAY	27.30	539.4	299.4	23	1.00	0.75	1	1	1.2	20.7
GRAVELLY CLAY	28.80	569.4	314.4	24	1.00	0.75	1	1	1.2	21.6
GRAVELLY CLAY	30.30	599.4	329.4	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	31.80	629.4	344.4	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	33.30	659.4	359.4	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	34.80	689.4	374.4	33	1.00	0.75	1	1	1.2	29.7
GRAVELLY CLAY	36.00	713.4	386.4	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	743.4	401.4	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	773.4	416.4	R	1.00	0.75	1	1	1.2	R

BH-15	**gwl (m)**	3	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
SILTY CLAY	3.00	10.0	54.0	4	1.00	0.75	1	1	1.2	3.6
ML	4.80	90.0	72.0	7	1.15	0.75	1	1	1.2	7.2
ML	6.30	120.0	87.0	12	1.06	0.75	1	1	1.2	11.5
ML	7.80	150.0	102.0	20	0.99	0.75	1	1	1.2	17.8
CLAY	9.30	180.0	117.0	5	1.00	0.75	1	1	1.2	4.5
CLAY	10.80	210.0	132.0	6	1.00	0.75	1	1	1.2	5.4
CLAY	12.30	240.0	147.0	5	1.00	0.75	1	1	1.2	4.5
CLAY	13.80	270.0	162.0	7	1.00	0.75	1	1	1.2	6.3
CLAY	15.30	300.0	177.0	6	1.00	0.75	1	1	1.2	5.4
CLAY	16.80	330.0	192.0	4	1.00	0.75	1	1	1.2	3.6
CLAY	18.30	360.0	207.0	6	1.00	0.75	1	1	1.2	5.4
MH	20.30	400.0	227.0	8	1.00	0.75	1	1	1.2	7.2
SM	21.30	420.0	237.0	14	0.62	0.75	1	1	1.2	7.8
GRAVELLY CLAY	22.80	450.0	252.0	19	1.00	0.75	1	1	1.2	17.1
GRAVELLY CLAY	24.30	480.0	267.0	21	1.00	0.75	1	1	1.2	18.9
CL	25.80	510.0	282.0	19	1.00	0.75	1	1	1.2	17.1
CL	27.30	540.0	297.0	24	1.00	0.75	1	1	1.2	21.6
CL	28.80	570.0	312.0	27	1.00	0.75	1	1	1.2	24.3
CL	30.30	600.0	327.0	28	1.00	0.75	1	1	1.2	25.2
GRAVELLY CLAY	31.80	630.0	342.0	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	33.30	660.0	357.0	32	1.00	0.75	1	1	1.2	28.8
GRAVELLY CLAY	34.80	690.0	372.0	33	1.00	0.75	1	1	1.2	29.7
GRAVELLY CLAY	36.00	714.0	384.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	744.0	399.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	774.0	414.0	R	1.00	0.75	1	1	1.2	R

BH-16	**gwl (m)**	3.2	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.6	70.6	R	1.15	0.75	1	1	1.2	R
SILTY SAND	6.30	119.6	88.6	10	1.05	0.75	1	1	1.2	9.5
SILTY SAND	7.80	149.6	103.6	12	0.98	0.75	1	1	1.2	10.6
CH	9.30	179.6	118.6	4	1.00	0.75	1	1	1.2	3.6
CH	10.80	209.6	133.6	6	1.00	0.75	1	1	1.2	5.4
CH	12.30	239.6	148.6	7	1.00	0.75	1	1	1.2	6.3
CH	13.80	269.6	163.6	6	1.00	0.75	1	1	1.2	5.4
CH	15.30	299.6	178.6	9	1.00	0.75	1	1	1.2	8.1
CH	16.80	329.6	193.6	8	1.00	0.75	1	1	1.2	7.2
CH	18.30	359.6	208.6	7	1.00	0.75	1	1	1.2	6.3
GRAVELLY SAND	19.50	383.6	220.6	R	0.65	0.75	1	1	1.2	R
CLAY	21.30	419.6	238.6	20	1.00	0.75	1	1	1.2	18.0
GRAVELLY CLAY	22.80	449.6	253.6	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	24.30	479.6	268.6	22	1.00	0.75	1	1	1.2	19.8
GRAVELLY CLAY	25.80	509.6	283.6	28	1.00	0.75	1	1	1.2	25.2
GRAVELLY CLAY	27.30	539.6	298.6	29	1.00	0.75	1	1	1.2	26.1
CL	28.80	569.6	313.6	31	1.00	0.75	1	1	1.2	27.9
CL	30.30	599.6	328.6	32	1.00	0.75	1	1	1.2	28.8
GC	31.50	623.6	340.6	R	0.48	0.75	1	1	1.2	R
GC	33.00	653.6	355.6	R	0.46	0.75	1	1	1.2	R
GRAVELLY CLAY	34.50	683.6	370.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	36.00	713.6	385.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	743.6	400.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	773.6	415.6	R	1.00	0.75	1	1	1.2	R

BH-17	**gwl (m)**	2.2	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
MI	1.80	10.0	32.4	3	1.44	0.75	1	0.75	1.2	2.9
MI	3.30	61.6	50.6	2	1.29	0.75	1	1	1.2	2.3
MI	4.80	91.6	65.6	5	1.19	0.75	1	1	1.2	5.3
SILTY SAND	6.30	121.6	80.6	10	1.10	0.75	1	1	1.2	9.9
SILTY SAND	7.80	151.6	95.6	12	1.02	0.75	1	1	1.2	11.0
CH	9.30	181.6	110.6	3	1.00	0.75	1	1	1.2	2.7
CH	10.80	211.6	125.6	4	1.00	0.75	1	1	1.2	3.6
CH	12.30	241.6	140.6	4	1.00	0.75	1	1	1.2	3.6
CH	13.80	271.6	155.6	7	1.00	0.75	1	1	1.2	6.3
CH	15.30	301.6	170.6	6	1.00	0.75	1	1	1.2	5.4
CH	16.80	331.6	185.6	5	1.00	0.75	1	1	1.2	4.5
CH	18.80	371.6	205.6	4	1.00	0.75	1	1	1.2	3.6
SAND	19.80	391.6	215.6	16	0.66	0.75	1	1	1.2	9.4
CH	21.30	421.6	230.6	23	1.00	0.75	1	1	1.2	20.7
GRAVELLY CLAY	22.80	451.6	245.6	25	1.00	0.75	1	1	1.2	22.5
GRAVELLY CLAY	24.30	481.6	260.6	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	25.80	511.6	275.6	27	1.00	0.75	1	1	1.2	24.3
GRAVELLY CLAY	27.30	541.6	290.6	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	28.80	571.6	305.6	18	1.00	0.75	1	1	1.2	16.2
GRAVELLY CLAY	30.30	601.6	320.6	19	1.00	0.75	1	1	1.2	17.1
SANDY CLAY	31.80	631.6	335.6	22	1.00	0.75	1	1	1.2	19.8
SANDY CLAY	33.30	661.6	350.6	25	1.00	0.75	1	1	1.2	22.5
SANDY CLAY	34.80	691.6	365.6	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	36.00	715.6	377.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	745.6	392.6	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	775.6	407.6	R	1.00	0.75	1	1	1.2	R

BH-18	**gwl (m)**	3.5	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.50	10.0	27.0	R	1.50	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.0	73.0	R	1.14	0.75	1	1	1.2	R
SILTY SAND	6.30	119.0	91.0	22	1.04	0.75	1	1	1.2	20.6
SANDY SILT	7.80	149.0	106.0	24	0.97	0.75	1	1	1.2	21.0
SANDY CLAY	9.30	179.0	121.0	20	1.00	0.75	1	1	1.2	18.0
SANDY CLAY	10.80	209.0	136.0	23	1.00	0.75	1	1	1.2	20.7
SANDY CLAY	12.30	239.0	151.0	6	1.00	0.75	1	1	1.2	5.4
SANDY CLAY	13.80	269.0	166.0	5	1.00	0.75	1	1	1.2	4.5
CL	15.30	299.0	181.0	4	1.00	0.75	1	1	1.2	3.6
CL	16.80	329.0	196.0	7	1.00	0.75	1	1	1.2	6.3
CL	18.30	359.0	211.0	13	1.00	0.75	1	1	1.2	11.7
CLAYEY SAND	19.80	389.0	226.0	22	0.64	0.75	1	1	1.2	12.6
CLAYEY SAND	21.30	419.0	241.0	21	0.61	0.75	1	1	1.2	11.5
GRAVELLY CLAY	22.80	449.0	256.0	22	1.00	0.75	1	1	1.2	19.8
GRAVELLY CLAY	24.30	479.0	271.0	25	1.00	0.75	1	1	1.2	22.5
GRAVELLY CLAY	25.80	509.0	286.0	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	27.30	539.0	301.0	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	28.80	569.0	316.0	30	1.00	0.75	1	1	1.2	27.0
GC	30.30	599.0	331.0	32	0.49	0.75	1	1	1.2	14.0
GC	31.50	623.0	343.0	R	0.48	0.75	1	1	1.2	R
GC	33.00	653.0	358.0	R	0.46	0.75	1	1	1.2	R
GRAVELLY SAND	34.50	683.0	373.0	R	0.45	0.75	1	1	1.2	R
SC	36.00	713.0	388.0	R	0.43	0.75	1	1	1.2	R
SC	37.50	743.0	403.0	R	0.42	0.75	1	1	1.2	R
GC	39.00	773.0	418.0	R	0.41	0.75	1	1	1.2	R

BH-19	gwl (m)	3.3	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.4	71.4	R	1.15	0.75	1	1	1.2	R
SILTY SAND	6.30	119.4	89.4	17	1.05	0.75	1	1	1.2	16.1
CH-MH	7.80	149.4	104.4	20	0.98	0.75	1	1	1.2	17.6
CH-MH	9.30	179.4	119.4	24	0.92	0.75	1	1	1.2	19.8
CH-MH	10.80	209.4	134.4	11	0.86	0.75	1	1	1.2	8.6
CLAY	12.30	239.4	149.4	7	1.00	0.75	1	1	1.2	6.3
CLAY	13.80	269.4	164.4	3	1.00	0.75	1	1	1.2	2.7
CLAY	15.40	301.4	180.4	4	1.00	0.75	1	1	1.2	3.6
CLAY	16.80	329.4	194.4	6	1.00	0.75	1	1	1.2	5.4
CLAY	18.30	359.4	209.4	4	1.00	0.75	1	1	1.2	3.6
SM	19.80	389.4	224.4	18	0.64	0.75	1	1	1.2	10.3
SM	21.30	419.4	239.4	19	0.61	0.75	1	1	1.2	10.5
GRAVELLY CLAY	22.80	449.4	254.4	16	1.00	0.75	1	1	1.2	14.4
SM	24.30	479.4	269.4	20	0.56	0.75	1	1	1.2	10.2
GRAVELLY CLAY	25.80	509.4	284.4	25	1.00	0.75	1	1	1.2	22.5
GRAVELLY CLAY	27.30	539.4	299.4	26	1.00	0.75	1	1	1.2	23.4
GRAVELLY CLAY	28.80	569.4	314.4	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	30.30	599.4	329.4	33	1.00	0.75	1	1	1.2	29.7
GRAVELLY CLAY	31.80	629.4	344.4	40	1.00	0.75	1	1	1.2	36.0
GRAVELLY CLAY	33.30	659.4	359.4	41	1.00	0.75	1	1	1.2	36.9
GRAVELLY CLAY	34.80	689.4	374.4	40	1.00	0.75	1	1	1.2	36.0
GRAVELLY CLAY	36.30	719.4	389.4	39	1.00	0.75	1	1	1.2	35.1
GRAVELLY CLAY	37.80	749.4	404.4	42	1.00	0.75	1	1	1.2	37.8
GRAVELLY CLAY	39.00	773.4	416.4	R	1.00	0.75	1	1	1.2	R

BH-20	**gwl (m)**	3.3	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.4	71.4	R	1.15	0.75	1	1	1.2	R
SP-SM	6.30	119.4	89.4	22	1.05	0.75	1	1	1.2	20.8
SP-SM	7.80	149.4	104.4	18	0.98	0.75	1	1	1.2	15.9
SP-SM	9.30	179.4	119.4	21	0.92	0.75	1	1	1.2	17.4
SP-SM	10.80	209.4	134.4	21	0.86	0.75	1	1	1.2	16.3
CH	12.30	239.4	149.4	14	1.00	0.75	1	1	1.2	12.6
CH	13.80	269.4	164.4	5	1.00	0.75	1	1	1.2	4.5
CH	15.30	299.4	179.4	7	1.00	0.75	1	1	1.2	6.3
CH	16.80	329.4	194.4	5	1.00	0.75	1	1	1.2	4.5
CH	18.30	359.4	209.4	3	1.00	0.75	1	1	1.2	2.7
CLAYEY SAND	19.80	389.4	224.4	17	0.64	0.75	1	1	1.2	9.8
CLAYEY SAND	21.30	419.4	239.4	26	0.61	0.75	1	1	1.2	14.3
GRAVELLY CLAY	22.80	449.4	254.4	26	1.00	0.75	1	1	1.2	23.4
GRAVELLY CLAY	24.30	479.4	269.4	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	25.80	509.4	284.4	29	1.00	0.75	1	1	1.2	26.1
GRAVELLY CLAY	27.30	539.4	299.4	32	1.00	0.75	1	1	1.2	28.8
GRAVELLY CLAY	28.80	569.4	314.4	34	1.00	0.75	1	1	1.2	30.6
GRAVELLY CLAY	30.30	599.4	329.4	37	1.00	0.75	1	1	1.2	33.3
GRAVELLY SAND	31.50	623.4	341.4	R	0.48	0.75	1	1	1.2	R
GRAVELLY SAND	33.00	653.4	356.4	R	0.46	0.75	1	1	1.2	R
GRAVELLY SAND	34.50	683.4	371.4	R	0.45	0.75	1	1	1.2	R
CL	36.00	713.4	386.4	R	1.00	0.75	1	1	1.2	R
CL	37.50	743.4	401.4	R	1.00	0.75	1	1	1.2	R
CL	39.00	773.4	416.4	R	1.00	0.75	1	1	1.2	R

BH-21	**gwl (m)**	3.4	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	83.2	72.2	R	1.14	0.75	1	1	1.2	R
SM	6.30	119.2	90.2	10	1.05	0.75	1	1	1.2	9.4
SM	7.80	149.2	105.2	14	0.98	0.75	1	1	1.2	12.3
SM	9.30	179.2	120.2	15	0.92	0.75	1	1	1.2	12.4
SILTY CLAY	10.80	209.2	135.2	9	1.00	0.75	1	1	1.2	8.1
SILTY CLAY	12.30	239.2	150.2	5	1.00	0.75	1	1	1.2	4.5
SILTY CLAY	13.80	269.2	165.2	3	1.00	0.75	1	1	1.2	2.7
SILTY CLAY	15.30	299.2	180.2	9	1.00	0.75	1	1	1.2	8.1
SILTY CLAY	16.80	329.2	195.2	9	1.00	0.75	1	1	1.2	8.1
SILTY CLAY	18.40	361.2	211.2	10	1.00	0.75	1	1	1.2	9.0
SM	19.80	389.2	225.2	18	0.64	0.75	1	1	1.2	10.3
SILTY SAND	21.30	419.2	240.2	23	0.61	0.75	1	1	1.2	12.6
SM	22.80	449.2	255.2	22	0.59	0.75	1	1	1.2	11.6
SM	24.30	479.2	270.2	25	0.56	0.75	1	1	1.2	12.7
SM	25.80	509.2	285.2	24	0.54	0.75	1	1	1.2	11.7
SM	27.30	539.2	300.2	29	0.52	0.75	1	1	1.2	13.7
SM	28.80	569.2	315.2	31	0.51	0.75	1	1	1.2	14.1
GRAVELLY SAND	30.30	599.2	330.2	38	0.49	0.75	1	1	1.2	16.7
GRAVELLY SAND	31.80	629.2	345.2	39	0.47	0.75	1	1	1.2	16.6
GRAVELLY CLAY	33.00	653.2	357.2	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	34.50	683.2	372.2	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	36.00	713.2	387.2	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	743.2	402.2	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	773.2	417.2	R	1.00	0.75	1	1	1.2	R

BH-22	**gwl (m)**	3.6	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
FILL	4.50	82.8	73.8	R	1.14	0.75	1	1	1.2	R
SILTY CLAY	6.30	118.8	91.8	11	1.00	0.75	1	1	1.2	9.9
SILTY CLAY	7.80	148.8	106.8	13	1.00	0.75	1	1	1.2	11.7
SILTY SAND	9.30	178.8	121.8	16	0.91	0.75	1	1	1.2	13.1
SILTY SAND	10.80	208.8	136.8	21	0.86	0.75	1	1	1.2	16.2
ML	12.30	238.8	151.8	3	0.81	0.75	1	1	1.2	2.2
ML	13.80	268.8	166.8	4	0.77	0.75	1	1	1.2	2.8
ML	15.30	298.8	181.8	4	0.73	0.75	1	1	1.2	2.6
ML	16.80	328.8	196.8	5	0.69	0.75	1	1	1.2	3.1
ML	18.30	358.8	211.8	7	0.66	0.75	1	1	1.2	4.2
SM	19.80	388.8	226.8	17	0.63	0.75	1	1	1.2	9.7
SANDY GRAVEL	21.00	412.8	238.8	R	0.61	0.75	1	1	1.2	R
CL	22.80	448.8	256.8	17	1.00	0.75	1	1	1.2	15.3
CL	24.30	478.8	271.8	17	1.00	0.75	1	1	1.2	15.3
GRAVELLY CLAY	25.80	508.8	286.8	21	1.00	0.75	1	1	1.2	18.9
GRAVELLY CLAY	27.30	538.8	301.8	23	1.00	0.75	1	1	1.2	20.7
GRAVELLY CLAY	28.80	568.8	316.8	24	1.00	0.75	1	1	1.2	21.6
GRAVELLY CLAY	30.30	598.8	331.8	30	1.00	0.75	1	1	1.2	27.0
GRAVELLY CLAY	31.80	628.8	346.8	31	1.00	0.75	1	1	1.2	27.9
GRAVELLY CLAY	33.30	658.8	361.8	36	1.00	0.75	1	1	1.2	32.4
GRAVELLY CLAY	34.80	688.8	376.8	41	1.00	0.75	1	1	1.2	36.9
GRAVELLY CLAY	36.00	712.8	388.8	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	37.50	742.8	403.8	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	39.00	772.8	418.8	R	1.00	0.75	1	1	1.2	R

BH-23	**gwl (m)**	3.2	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1.2	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1.2	R
SILTY SAND	4.80	89.6	73.6	18	1.14	0.75	1	1	1.2	18.4
SILTY SAND	6.30	119.6	88.6	19	1.05	0.75	1	1	1.2	18.0
SILTY SAND	7.80	149.6	103.6	22	0.98	0.75	1	1	1.2	19.5
SILTY SAND	9.30	179.6	118.6	10	0.92	0.75	1	1	1.2	8.3
CLAY	10.80	209.6	133.6	6	1.00	0.75	1	1	1.2	5.4
CLAY	12.30	239.6	148.6	5	1.00	0.75	1	1	1.2	4.5
CLAY	13.80	269.6	163.6	6	1.00	0.75	1	1	1.2	5.4
CLAY	15.30	299.6	178.6	4	1.00	0.75	1	1	1.2	3.6
CLAY	16.80	329.6	193.6	5	1.00	0.75	1	1	1.2	4.5
CLAYEY SILT	18.40	361.6	209.6	6	1.00	0.75	1	1	1.2	5.4
CLAYEY SILT	19.80	389.6	223.6	16	1.00	0.75	1	1	1.2	14.4
CLAYEY SAND	21.30	419.6	238.6	18	0.61	0.75	1	1	1.2	9.9
CLAYEY SAND	22.80	449.6	253.6	25	0.59	0.75	1	1	1.2	13.2
CLAYEY SAND	24.30	479.6	268.6	27	0.57	0.75	1	1	1.2	13.8
CLAYEY GRAVEL	25.80	509.6	283.6	28	0.55	0.75	1	1	1.2	13.7
CLAYEY GRAVEL	27.30	539.6	298.6	30	0.53	0.75	1	1	1.2	14.2
CLAYEY GRAVEL	28.50	563.6	310.6	R	0.51	0.75	1	1	1.2	R
CLAYEY GRAVEL	30.00	593.6	325.6	R	0.49	0.75	1	1	1.2	R
CLAYEY GRAVEL	31.50	623.6	340.6	R	0.48	0.75	1	1	1.2	R
CLAYEY GRAVEL	33.00	653.6	355.6	R	0.46	0.75	1	1	1.2	R
CLAYEY GRAVEL	34.50	683.6	370.6	R	0.45	0.75	1	1	1.2	R
CLAYEY GRAVEL	36.00	713.6	385.6	R	0.44	0.75	1	1	1.2	R
CLAYEY GRAVEL	37.50	743.6	400.6	R	0.42	0.75	1	1	1.2	R
CLAYEY GRAVEL	39.00	773.6	415.6	R	0.41	0.75	1	1	1.2	R

BH-Add1	**gwl (m)**	5.5	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1	R
FILL	4.50	10.0	81.0	R	1.09	0.75	1	1	1	R
GRAVELLY SILT	6.80	125.0	112.0	11	0.95	0.75	1	1	1	7.8
GRAVELLY SILT	7.80	145.0	122.0	16	0.91	0.75	1	1	1	10.9
GRAVELLY SILT	9.30	175.0	137.0	17	0.86	0.75	1	1	1	10.9
CLAY	10.80	205.0	152.0	14	1.00	0.75	1	1	1	10.5
CLAY	12.30	235.0	167.0	4	1.00	0.75	1	1	1	3.0
CLAY	13.80	265.0	182.0	2	1.00	0.75	1	1	1	1.5
CLAY	15.30	295.0	197.0	4	1.00	0.75	1	1	1	3.0
CLAY	16.80	325.0	212.0	8	1.00	0.75	1	1	1	6.0
CLAY	18.30	355.0	227.0	9	1.00	0.75	1	1	1	6.8
CLAY	19.80	385.0	242.0	10	1.00	0.75	1	1	1	7.5
SILTY CLAY	21.30	415.0	257.0	10	1.00	0.75	1	1	1	7.5
GRAVELLY CLAY	22.80	445.0	272.0	18	1.00	0.75	1	1	1	13.5
GRAVELLY CLAY	24.30	475.0	287.0	20	1.00	0.75	1	1	1	15.0
GRAVELLY CLAY	25.80	505.0	302.0	24	1.00	0.75	1	1	1	18.0
GRAVELLY CLAY	27.30	535.0	317.0	27	1.00	0.75	1	1	1	20.3
GRAVELLY CLAY	28.80	565.0	332.0	29	1.00	0.75	1	1	1	21.8
GRAVEL	30.30	595.0	347.0	32	0.47	0.75	1	1	1	11.3
CLAY	31.80	625.0	362.0	32	1.00	0.75	1	1	1	24.0
CLAY	33.30	655.0	377.0	40	1.00	0.75	1	1	1	30.0
CLAY	34.80	685.0	392.0	42	1.00	0.75	1	1	1	31.5
GRAVEL	36.00	709.0	404.0	R	0.42	0.75	1	1	1	R
GRAVEL	37.50	739.0	419.0	R	0.41	0.75	1	1	1	R
GRAVEL	39.00	769.0	434.0	R	0.40	0.75	1	1	1	R

continue

USCS	Z (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	N_{30}	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$
SANDY CLAY	40.80	805.0	452.0	46	1.00	0.75	1	1	1	34.5
SANDY CLAY	42.00	829.0	464.0	R	1.00	0.75	1	1	1	R
GRAVELLY SILT	43.80	865.0	482.0	44	0.37	0.75	1	1	1	12.1
SANDY GRAVEL	45.30	895.0	497.0	47	0.36	0.75	1	1	1.2	15.1
SANDY SILT	46.80	925.0	512.0	45	0.35	0.75	1	1	1.2	14.1
SANDY SILT	48.30	955.0	527.0	45	0.34	0.75	1	1	1.2	13.8
SANDY CLAY	49.80	985.0	542.0	43	1.00	0.75	1	1	1.2	38.7
SANDY CLAY	51.30	1015.0	557.0	48	1.00	0.75	1	1	1.2	43.2
GRAVELLY CLAY	52.50	1039.0	569.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	54.00	1069.0	584.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	55.50	1099.0	599.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	57.00	1129.0	614.0	R	1.00	0.75	1	1	1.2	R
GRAVELLY CLAY	58.50	1159.0	629.0	R	1.00	0.75	1	1	1.2	R

BH-Add2	gwl (m)	7.2	γ (kN/m^3)	18	γ_{sat} (kN/m^3)	20	γ_w (kN/m^3)	10	γ' (kN/m^3)	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1	R
FILL	4.50	10.0	81.0	R	1.09	0.75	1	1	1	R
CLAYEY SILT	6.30	10.0	113.4	9	1.00	0.75	1	1	1	6.8
SILT	7.80	141.6	135.6	13	0.86	0.75	1	1	1	8.4
CLAY	9.30	171.6	150.6	12	1.00	0.75	1	1	1	9.0
SILTY CLAY	10.80	201.6	165.6	15	1.00	0.75	1	1	1	11.3
SILTY CLAY	12.30	231.6	180.6	13	1.00	0.75	1	1	1	9.8
SILTY SAND	13.80	261.6	195.6	18	0.70	0.75	1	1	1	9.4
SILTY SAND	15.30	291.6	210.6	17	0.67	0.75	1	1	1	8.5
CLAY	16.80	321.6	225.6	7	1.00	0.75	1	1	1	5.3
CLAY	18.30	351.6	240.6	6	1.00	0.75	1	1	1	4.5
CLAY	19.80	381.6	255.6	7	1.00	0.75	1	1	1	5.3
SANDY CLAY	21.30	411.6	270.6	18	1.00	0.75	1	1	1	13.5
SANDY CLAY	22.80	441.6	285.6	28	1.00	0.75	1	1	1	21.0
SANDY CLAY	24.30	471.6	300.6	30	1.00	0.75	1	1	1	22.5
GRAVELLY CLAY	25.80	501.6	315.6	30	1.00	0.75	1	1	1	22.5
GRAVELLY CLAY	27.30	531.6	330.6	35	1.00	0.75	1	1	1	26.3
GRAVELLY CLAY	28.80	561.6	345.6	35	1.00	0.75	1	1	1	26.3
GRAVELLY CLAY	30.30	591.6	360.6	43	1.00	0.75	1	1	1	32.3
GRAVELLY CLAY	31.80	621.6	375.6	42	1.00	0.75	1	1	1	31.5
GRAVELLY CLAY	33.30	651.6	390.6	R	1.00	0.75	1	1	1	R
GRAVELLY CLAY	34.50	675.6	402.6	R	1.00	0.75	1	1	1	R
GRAVEL	36.00	705.6	417.6	R	0.41	0.75	1	1	1	R
GRAVEL	37.50	735.6	432.6	R	0.40	0.75	1	1	1	R
GRAVEL	39.00	765.6	447.6	R	0.39	0.75	1	1	1	R

continue

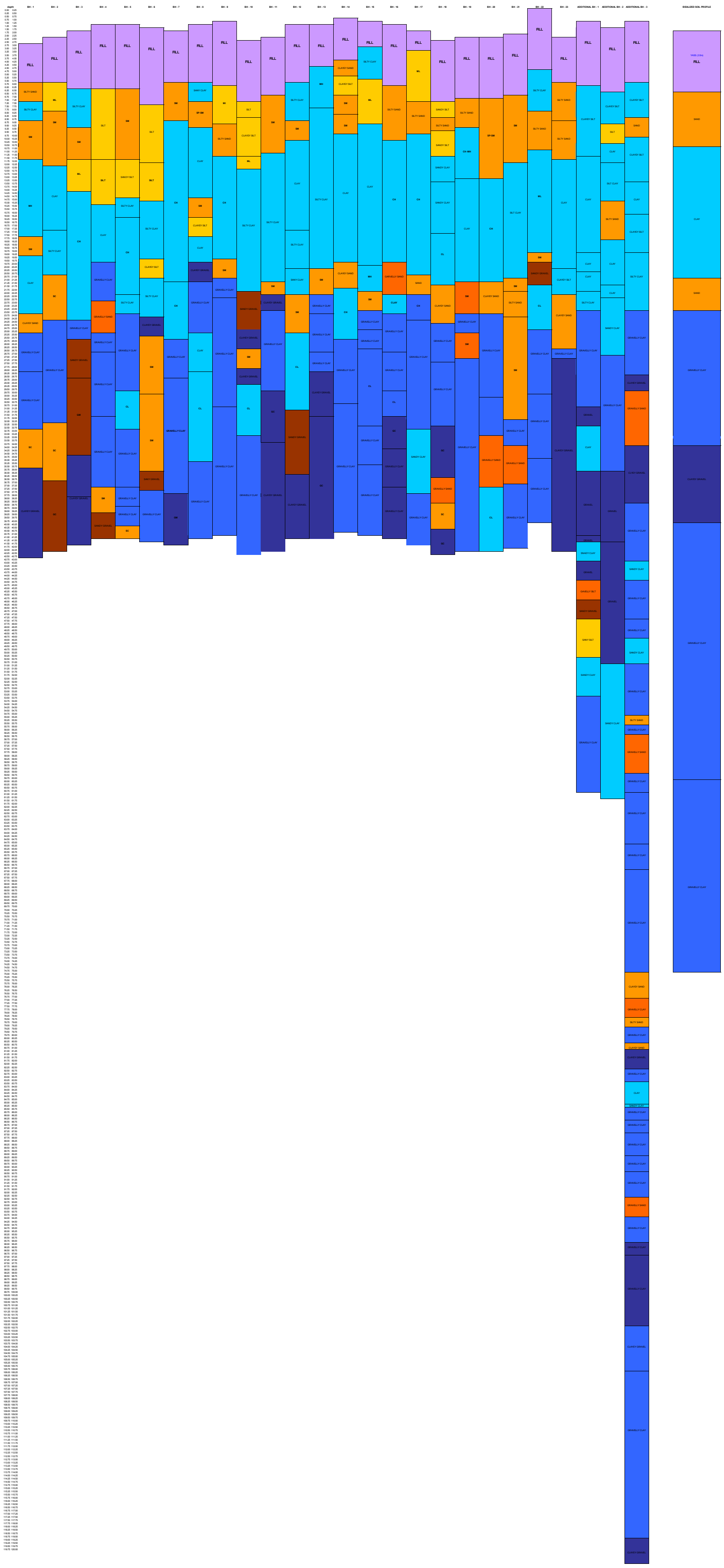
USCS	Z (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	N_{30}	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$
GRAVEL	40.50	795.6	462.6	R	0.38	0.75	1	1	1.2	R
GRAVEL	42.00	825.6	477.6	R	0.37	0.75	1	1	1.2	R
GRAVEL	43.50	855.6	492.6	R	0.36	0.75	1	1	1.2	R
GRAVEL	45.00	885.6	507.6	R	0.35	0.75	1	1	1.2	R
GRAVEL	46.50	915.6	522.6	R	0.34	0.75	1	1	1.2	R
GRAVEL	48.00	945.6	537.6	R	0.33	0.75	1	1	1.2	R
SANDY CLAY	49.80	981.6	555.6	32	1.00	0.75	1	1	1.2	28.8
SANDY CLAY	51.30	1011.6	570.6	35	1.00	0.75	1	1	1.2	31.5
SANDY CLAY	52.80	1041.6	585.6	36	1.00	0.75	1	1	1.2	32.4
SANDY CLAY	54.30	1071.6	600.6	40	1.00	0.75	1	1	1.2	36.0
SANDY CLAY	55.80	1101.6	615.6	32	1.00	0.75	1	1	1.2	28.8
SANDY CLAY	57.30	1131.6	630.6	38	1.00	0.75	1	1	1.2	34.2
SANDY CLAY	58.80	1161.6	645.6	41	1.00	0.75	1	1	1.2	36.9

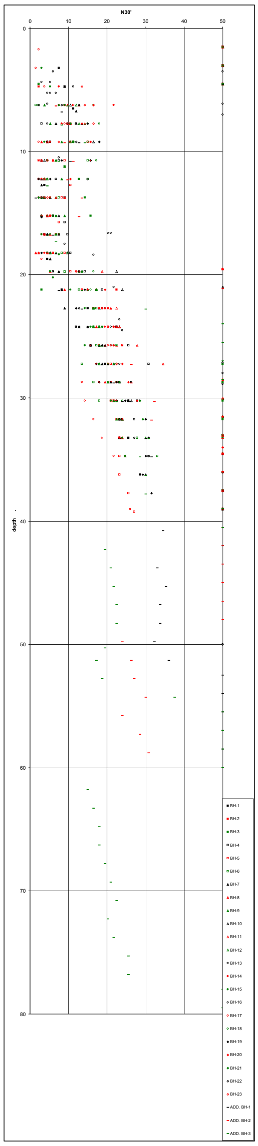
BH-Add3	**gwl (m)**	3.1	**γ (kN/m^3)**	18	**γ_{sat} (kN/m^3)**	20	**γ_w (kN/m^3)**	10	**γ' (kN/m^3)**	10
USCS	**Z (m)**	**σ_{vo} (kPa)**	**σ_{vo}' (kPa)**	**N_{30}**	**C_N**	**C_E**	**C_B**	**C_R**	**C_S**	**$(N_1)_{60}$**
FILL	1.52	10.0	27.4	R	1.49	0.75	1	0.75	1	R
FILL	3.00	10.0	54.0	R	1.26	0.75	1	1	1	R
FILL	4.50	83.8	69.8	R	1.16	0.75	1	1	1	R
CLAYEY SILT	6.30	119.8	87.8	13	1.00	0.75	1	1	1	9.8
SAND	7.80	149.8	102.8	19	0.99	0.75	1	1	1	14.1
CLAYEY SILT	9.30	179.8	117.8	19	1.00	0.75	1	1	1	14.3
CLAYEY SILT	10.80	209.8	132.8	10	1.00	0.75	1	1	1	7.5
CLAY	12.80	249.8	152.8	6	1.00	0.75	1	1	1	4.5
CLAY	13.80	269.8	162.8	5	1.00	0.75	1	1	1	3.8
CLAYEY SILT	15.30	299.8	177.8	9	1.00	0.75	1	1	1	6.8
CLAYEY SILT	17.30	339.8	197.8	9	1.00	0.75	1	1	1	6.8
SILTY CLAY	18.30	359.8	207.8	7	1.00	0.75	1	1	1	5.3
SILTY CLAY	19.80	389.8	222.8	12	1.00	0.75	1	1	1	9.0
SILTY CLAY	21.30	419.8	237.8	18	1.00	0.75	1	1	1	13.5
GRAVELLY CLAY	22.80	449.8	252.8	40	1.00	0.75	1	1	1	30.0
GRAVELLY CLAY	24.00	473.8	264.8	R	1.00	0.75	1	1	1	R
GRAVELLY CLAY	25.50	503.8	279.8	R	1.00	0.75	1	1	1	R
GRAVELLY CLAY	27.00	533.8	294.8	R	1.00	0.75	1	1	1	R
CLAYEY GRAVEL	28.50	563.8	309.8	R	0.51	0.75	1	1	1	R
GRAVELLY SAND	30.30	599.8	327.8	30	0.49	0.75	1	1	1	11.1
GRAVELLY SAND	31.80	629.8	342.8	32	0.48	0.75	1	1	1	11.4
CLAYEY GRAVEL	33.30	659.8	357.8	41	0.46	0.75	1	1	1	14.2
CLAYEY GRAVEL	34.80	689.8	372.8	38	0.45	0.75	1	1	1	12.7
CLAYEY GRAVEL	36.30	719.8	387.8	40	0.43	0.75	1	1	1	13.0
GRAVELLY CLAY	37.80	749.8	402.8	40	1.00	0.75	1	1	1	30.0
GRAVELLY CLAY	39.00	773.8	414.8	R	1.00	0.75	1	1	1	R

continue

USCS	Z (m)	σ_{vo} (kPa)	σ_{vo}' (kPa)	N_{30}	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$
GRAVELLY CLAY	40.50	803.8	429.8	R	1.00	0.75	1	1	1	R
SANDY CLAY	42.30	839.8	447.8	26	1.00	0.75	1	1	1	19.5
GRAVELLY CLAY	43.80	869.8	462.8	28	1.00	0.75	1	1	1	21.0
GRAVELLY CLAY	45.30	899.8	477.8	29	1.00	0.75	1	1	1	21.8
GRAVELLY CLAY	46.80	929.8	492.8	30	1.00	0.75	1	1	1	22.5
SANDY CLAY	48.30	959.8	507.8	30	1.00	0.75	1	1	1	22.5
GRAVELLY CLAY	50.30	999.8	527.8	26	1.00	0.75	1	1	1	19.5
GRAVELLY CLAY	51.30	1019.8	537.8	23	1.00	0.75	1	1	1	17.3
GRAVELLY CLAY	52.80	1049.8	552.8	25	1.00	0.75	1	1	1	18.8
SILTY SAND	54.30	1079.8	567.8	50	0.32	0.75	1	1	1	12.0
GRAVELLY SAND	55.50	1103.8	579.8	R	0.31	0.75	1	1	1	R
GRAVELLY SAND	57.00	1133.8	594.8	R	0.31	0.75	1	1	1	R
GRAVELLY SAND	58.50	1163.8	609.8	R	0.30	0.75	1	1	1	R
GRAVELLY CLAY	60.00	1193.8	624.8	R	1.00	0.75	1	1	1	R
GRAVELLY CLAY	61.80	1229.8	642.8	20	1.00	0.75	1	1	1	15.0
GRAVELLY CLAY	63.30	1259.8	657.8	22	1.00	0.75	1	1	1	16.5
GRAVELLY CLAY	64.80	1289.8	672.8	24	1.00	0.75	1	1	1	18.0
GRAVELLY CLAY	66.30	1319.8	687.8	24	1.00	0.75	1	1	1	18.0
GRAVELLY CLAY	67.80	1349.8	702.8	26	1.00	0.75	1	1	1	19.5
GRAVELLY CLAY	69.30	1379.8	717.8	28	1.00	0.75	1	1	1	21.0
GRAVELLY CLAY	70.80	1409.8	732.8	30	1.00	0.75	1	1	1	22.5
GRAVELLY CLAY	72.30	1439.8	747.8	27	1.00	0.75	1	1	1	20.3
GRAVELLY CLAY	73.80	1469.8	762.8	29	1.00	0.75	1	1	1	21.8
CLAYEY SAND	75.30	1499.8	777.8	34	0.25	0.75	1	1	1	6.2
GRAVELLY CLAY	76.80	1529.8	792.8	34	1.00	0.75	1	1	1	25.5
SILTY SAND	78.00	1553.8	804.8	R	0.24	0.75	1	1	1	R
GRAVELLY CLAY	79.50	1583.8	819.8	R	1.00	0.75	1	1	1	R

APPENDICE C
CROSS – SECTIONS OF BOREHOLES AND THE IDEALISED SOIL PROFILE





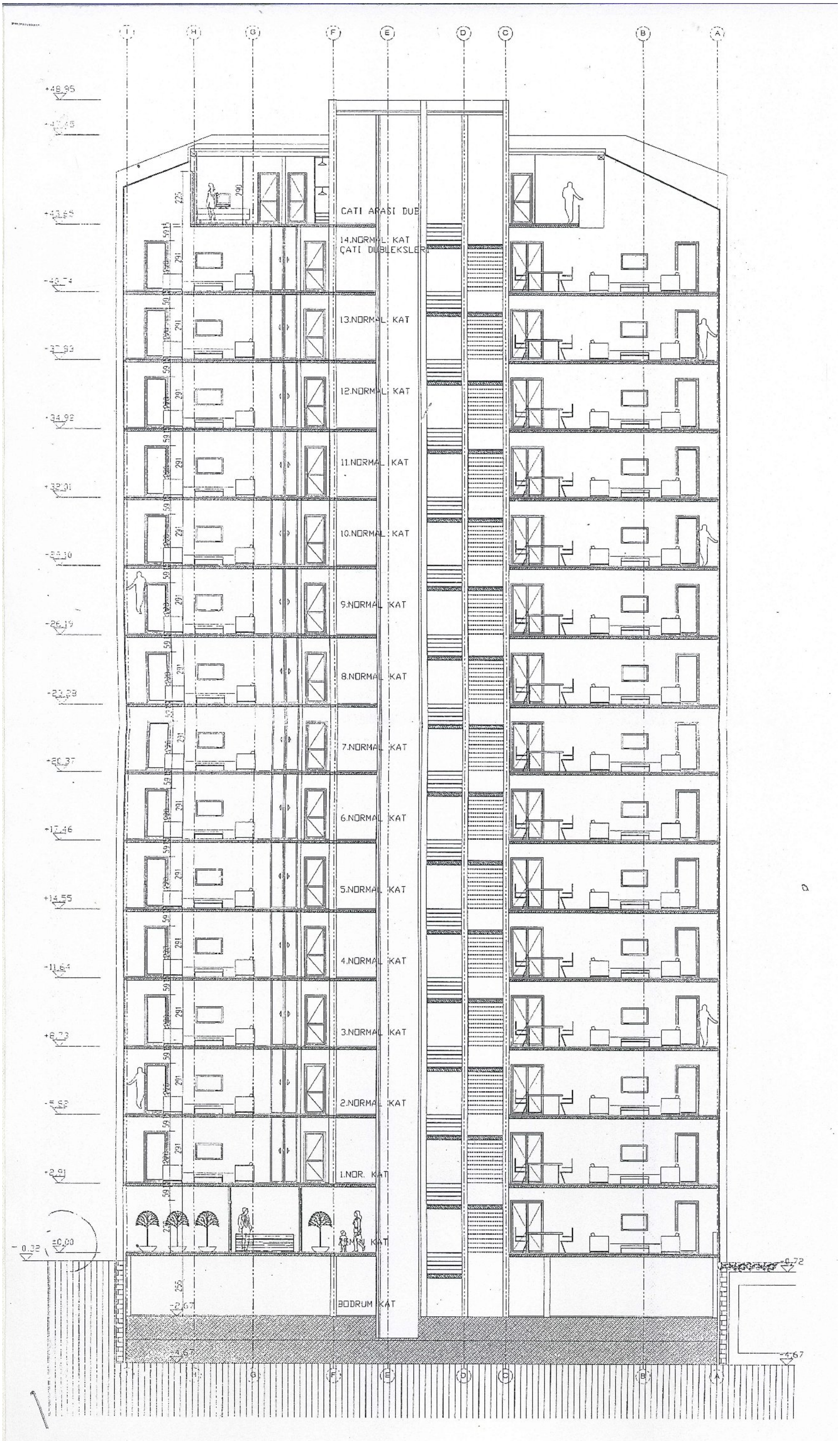
N30'
0
10
20
30
40
50
depth
60
70
80
BH-1
BH-2
BH-3
BH-4
BH-5
BH-6
BH-7
BH-8
BH-9
BH-10
BH-11
BH-12
BH-13
BH-14
BH-15
BH-16
BH-17
BH-18
BH-19
BH-20
BH-21
BH-22
BH-23
ADD. BH-1
ADD. BH-2
ADD. BH-3

APPENDICE D
FVST (FIELD VANE SHEAR TEST) CALCULATIONS

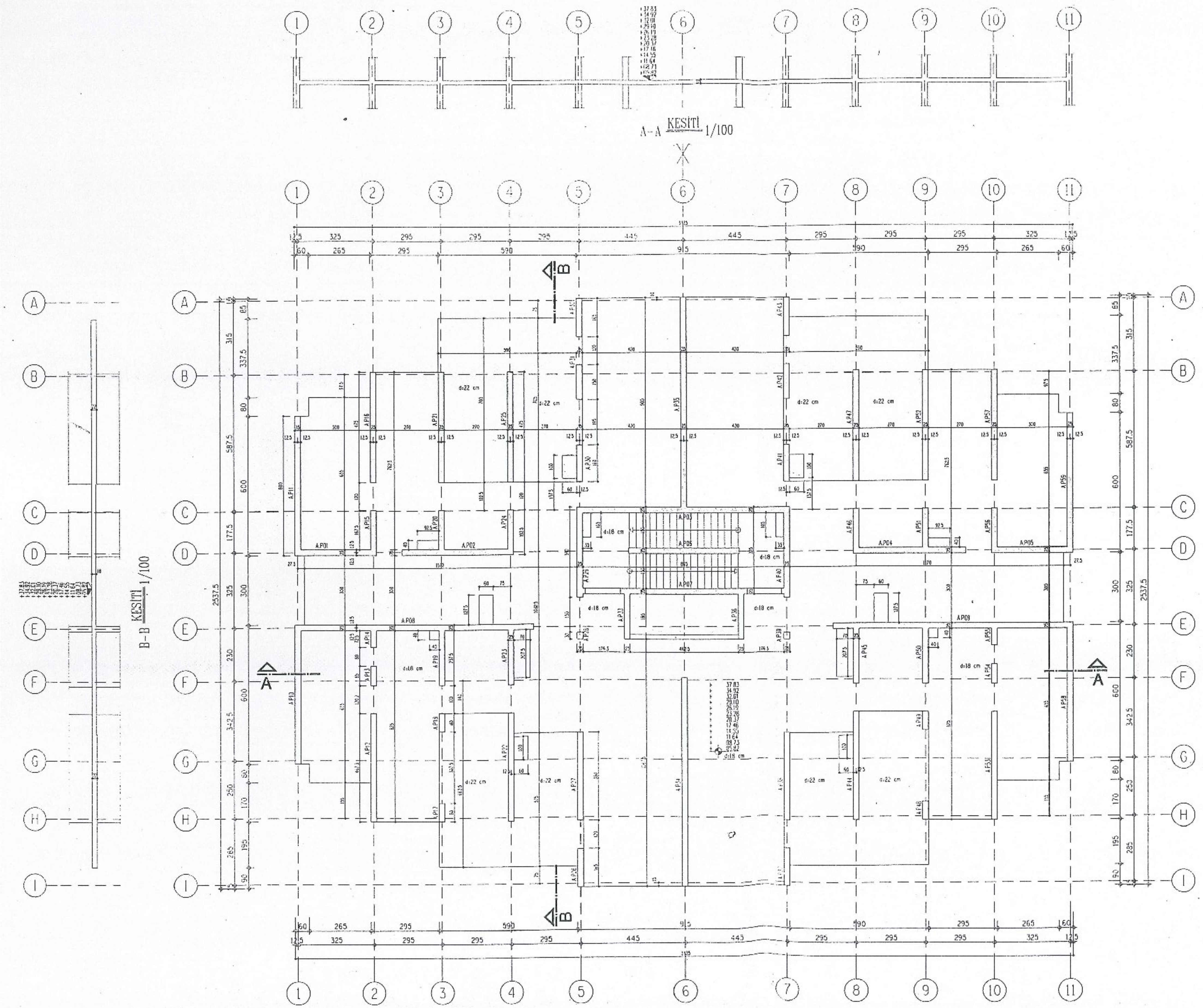
d (m)	0,065
d_1 (m)	0.0127

bore hole number	**depth (m)**	**USCS**	**T (Nm)**	**$s_{u,v}$ (kPa)**	**I_p**	**λ**	**$s_{u,design}$ (kPa)**
BH5	13.00 - 13.10	silty clay	45	36,95	36	0,85	31
BH6	14.00 - 14.10	silty clay	25	20,53	35	0,85	17
BH6	17.00 - 17.10	silty clay	30	24,63	35	0,85	21
BH9	13.00 - 13.10	CH	30	24,63	32	0,87	21
BH9	14.00 - 13.10	CH	35	28,74	32	0,87	25
BH9	17.00 - 13.10	CH	35	28,74	32	0,87	25
BH10	11.00 - 11.10	silty clay	30	24,63	35	0,85	21
BH10	13.00 - 13.10	silty clay	30	24,63	35	0,85	21
BH10	14.00 - 14.10	silty clay	35	28,74	35	0,85	24
BH10	17.00 - 17.10	silty clay	35	28,74	35	0,85	24
BH15	14.00 - 14.10	clay	30	24,63	35	0,85	21
BH16	16.00 - 16.10	silty clay	35	28,74	31	0,87	25
BH18	14.00 - 14.10	CL	20	16,42	37	0,84	14
BH18	16.00 - 16.10	CL	30	24,63	37	0,84	21
BH18	17.00 - 17.10	CL	35	28,74	37	0,84	24
BH19	15.00 - 15.10	clay	30	24,63	35	0,85	21
BH19	17.00 - 17.10	clay	35	28,74	35	0,85	24
BH20	13.00 - 13.10	CH	45	36,95	42	0,83	31
BH20	16.00 - 16.10	CH	30	24,63	42	0,83	20
BH21	17.00 - 17.10	CH	35	28,74	35	0,85	24
BH21	18.00 - 18.10	CH	30	24,63	35	0,85	21
BH23	14.00 - 14.10	clay	35	28,74	38	0,84	24
BH23	16.00 - 16.10	clay	30	24,63	38	0,84	21
BH23	18.00 - 18.10	clayey silt	30	24,63	22	0,96	24
						Average	**23**

APPENDICE E
THE PLAN AND CROSSECTION OF THE STRUCTURE

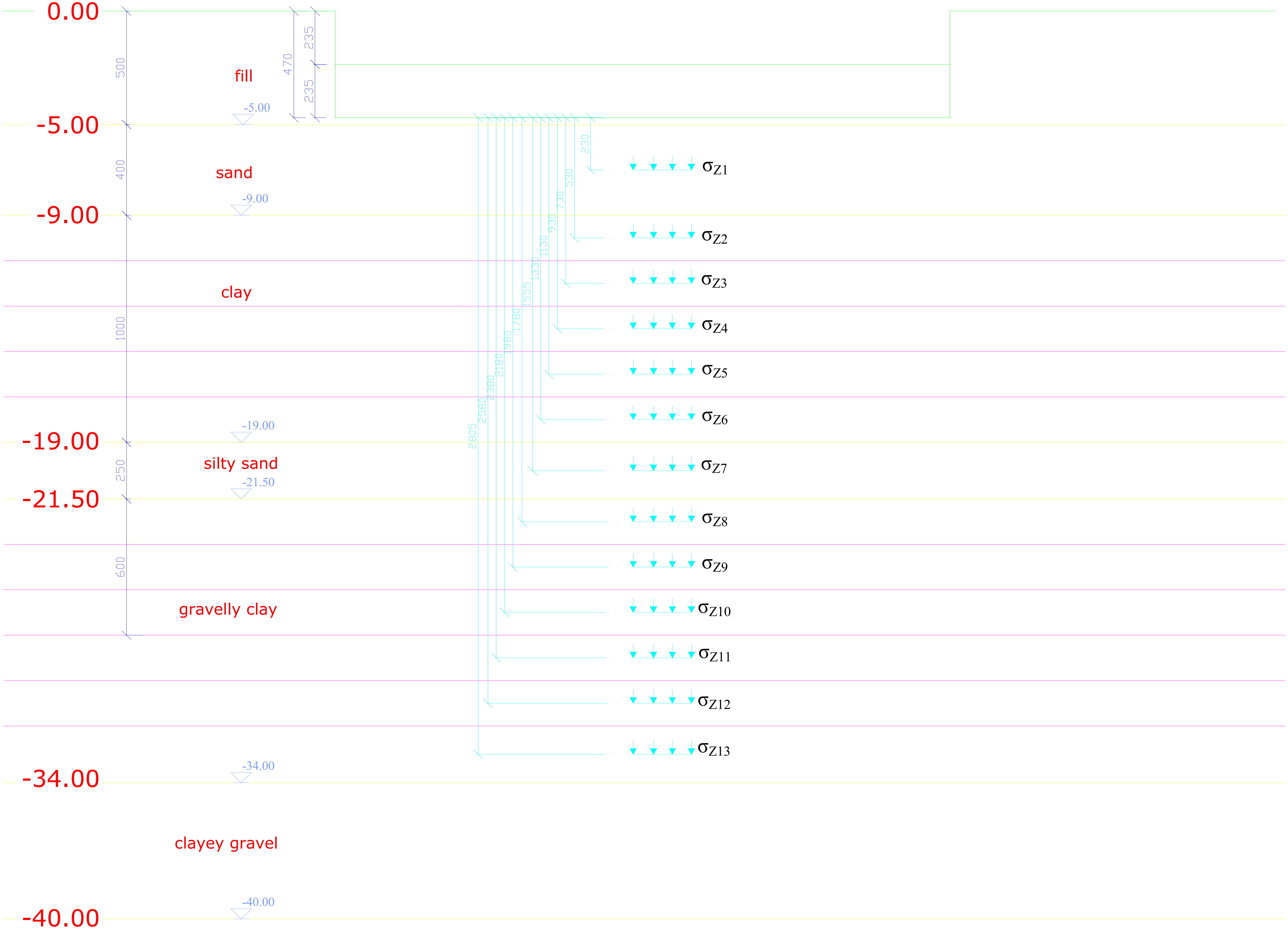


ÇATI ARASI DUB
14.NORMAL KAT ÇATI DUBLEKSLERİ
13.NORMAL KAT
12.NORMAL KAT
11.NORMAL KAT
10.NORMAL KAT
9.NORMAL KAT
8.NORMAL KAT
7.NORMAL KAT
6.NORMAL KAT
5.NORMAL KAT
4.NORMAL KAT
3.NORMAL KAT
2.NORMAL KAT
1.NOR. KAT
BODRUM KAT
+48.95
+43.65
+40.74
+37.83
+34.92
+32.01
+26.19
+23.28
+20.37
+17.46
+14.55
+11.64
+8.73
+5.82
+2.91
±0.00
-0.32
-0.72
-2.67
-4.67



NORMAL KAT PLANI

APPENDICE F
RAFT FOUNDATION CALCULATIONS

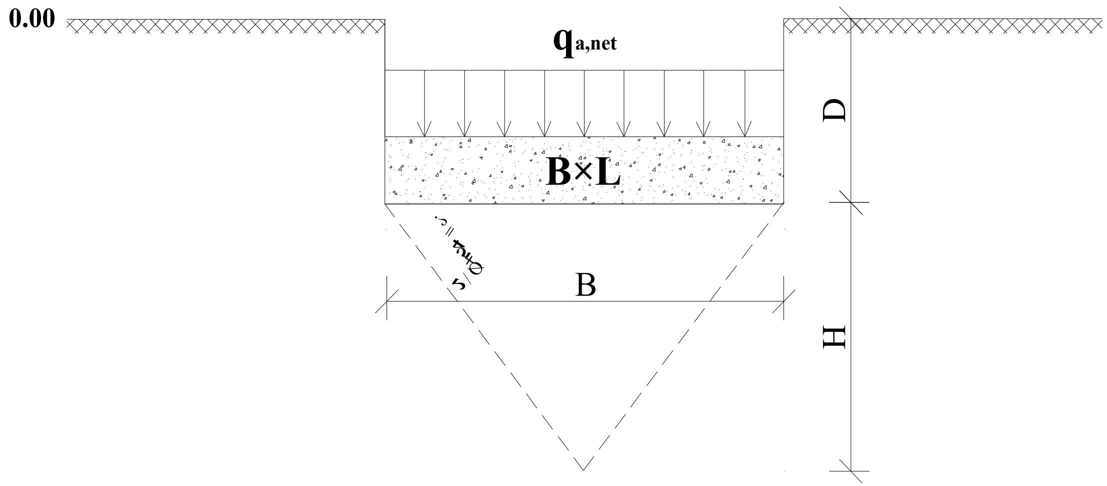


0.00
fill
-5.00
sand
-9.00
clay
-19.00
silty sand
-21.50
gravelly clay
-34.00
clayey gravel
-40.00
500
400
1000
250
600
470
235
235
σ_{Z1}
σ_{Z2}
σ_{Z3}
σ_{Z4}
σ_{Z5}
σ_{Z6}
σ_{Z7}
σ_{Z8}
σ_{Z9}
σ_{Z10}
σ_{Z11}
σ_{Z12}
σ_{Z13}
230
530
730
930
1130
1330
1555
1780
1980
2180
2380
2580
2805

TheBearing Capacity of the Raft Foundation

CLAY

c	**Cohession (t/m^2)**	1,9
Φ	**Internal friction angle (°)**	1
D	**Depth of the foundation (m)**	5
B	**Width of the foundaation (m)**	31
L	**Length of the foundation (m)**	38
d_w	**gwl (m)**	3,5
γ_1	**saturated density (t/m^3)**	2
r_γ	**The correction factor (B≥2m)**	0,70



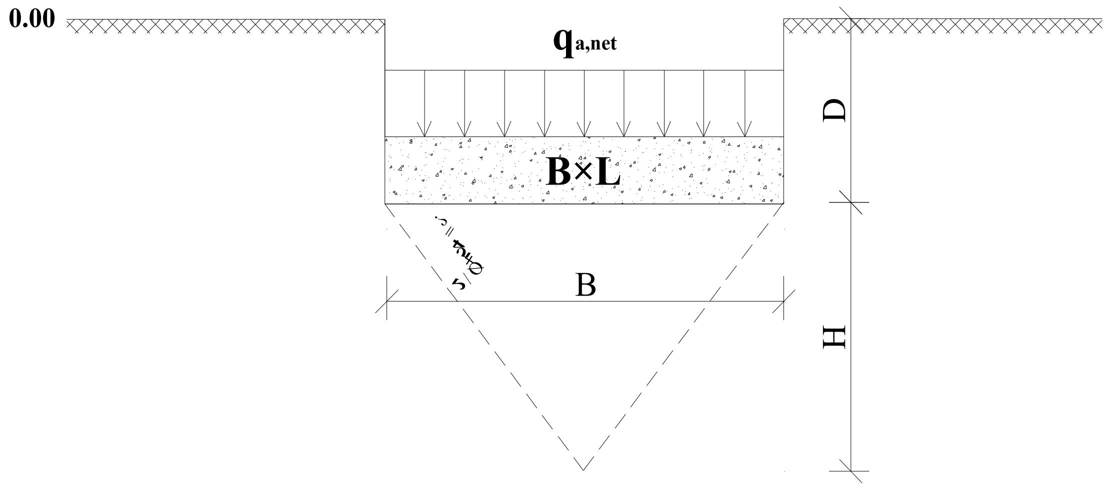
$$H = 0,5B\tan\left(45 + \frac{\phi}{2}\right)$$

H= 15,77 m

	bearing capacity factors			shape fators			depth factors					
	N_c	N_γ	N_q	s_c	s_q	s_γ	d_c	d_q	d_γ	K_p	F	q_a (t/m^2)
Meyerhof	5,379	0,002	1,094	1,17	1,00	1,00	1,03	1,00	1,00	1,036	3	**7,77**
Hansen	5,379	0,002	1,094	1,17	1,01	1,00	1,06	1,01	1,00	1,036	3	**7,96**

SAND

c	Cohession (t/m^2)	1
Φ	Internal friction angle (°)	32
D	Depth of the foundation (m)	5
B	Width of the foundaation (m)	27
L	Length of the foundation (m)	34
d_w	gwl (m)	3,5
γ_1	saturated density (t/m^3)	2
r_γ	The correction factor (B≥2m)	0,72



$$H = 0,5B\tan\left(45+\frac{\phi}{2}\right)$$

H= 24,35 m

	bearing capacity factors			shape fators			depth factors					
	N_c	N_γ	N_q	s_c	s_q	s_γ	d_c	d_q	d_γ	K_p	F	q_a (t/m^2)
Meyerhof	35,490	22,022	23,177	1,52	1,26	1,26	1,07	1,20	1,20	3,255	3	**242,46**
Hansen	35,490	20,786	23,177	1,52	1,50	1,00	1,07	1,05	1,00	3,255	3	**207,88**

The Consolidation Settlement of The Raft Foundation

q_d (kPa)	B (m)	L (m)	Z (m)	I_r	Δσ (kPa)	σ' (kPa)	H (cm)	C_c	e_o	**ΔH_{k1} (cm)**
143,0	27	34								
CLAY (9.00-19.00)			5,30	0,243	138,82	125,00	200	0,50	1,27	**14,3**
			7,30	0,233	133,41	139,00	200	0,50	1,27	**12,9**
			9,30	0,220	126,03	153,00	200	0,50	1,27	**11,5**
			11,30	0,205	117,34	167,00	200	0,50	1,27	**10,2**
			13,30	0,189	108,05	181,00	200	0,50	1,27	**9,0**
GRAVELLY CLAY (21.5-34.00)			17,80	0,153	87,60	226,00	200	0,19	0,47	**3,7**
			19,80	0,139	79,40	246,00	200	0,19	0,47	**3,1**
			21,80	0,126	71,90	286,00	200	0,19	0,47	**2,5**
			23,80	0,114	65,14	306,00	200	0,19	0,47	**2,2**
			25,80	0,103	59,08	326,00	200	0,19	0,47	**1,9**
			28,05	0,093	53,05	348,50	250	0,19	0,47	**2,0**
						consolidation settlement				73,2

$$I_r = \frac{1}{2\pi}\left[\frac{LBz\left(L^2+B^2+2z^2\right)}{\left(L^2+z^2\right)\left(B^2+z^2\right)\sqrt{L^2+B^2+z^2}} + \tan^{-1}\underbrace{\frac{LB}{z\sqrt{L^2+B^2+z^2}}}_{radyan}\right]$$

$$\Delta\sigma_z = qI_r$$

$$Se_0 = wG_s \text{.........} S = 1$$

$$e_0(\%) = wG_s$$

$$e_0(\%) = 17.75 \times 2.67$$

$$e_0(\%) = 47$$

$$\Delta H = \frac{C_c H}{1+e_o}\log\frac{\sigma'_v + \Delta\sigma}{\sigma'_v}$$

CH (21.50 - 34.00)

C_c= 0.046+0.0104Ip ....Nakes et al. (1988)
C_c=0.18

C_c=0.00234$w_L G_s$ ……...........................Nagaraj and Srinivasa Murthy (1985, 1986)
C_c=0.20

C_c=0.009(w_L-10)………....................Terzaghi and Peck (1967)
C_c=0.20

Cc=0.01w_N
Cc=18
$C_{c\ secilen}$=0.19

The Elastic Settlement of The Raft Foundation

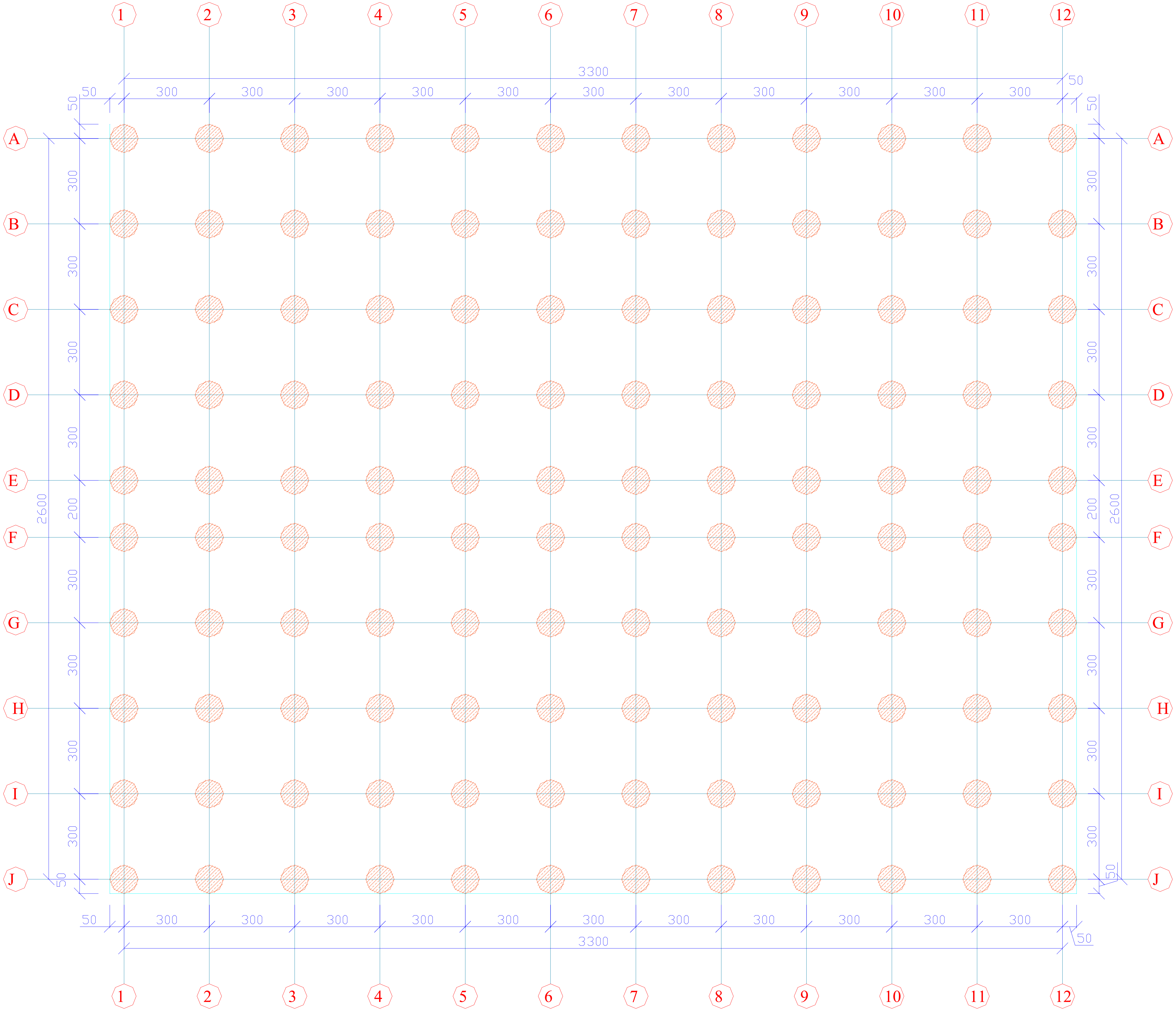
SAND ; $5^{,00}$ - $9^{,00}$													
q_d (kPa)	H (m)	yass (m)	L (m)	B (m)	M	N	I1	I2	Is	If	E (kPa)	μ	ΔH_{e1} (cm)
170,0	4,00	3,1	34	27	1,26	0,59	0,061	0,084	0,109	0,670	7150	0,3	4,3
									Elastic settlement (cm)				4,3

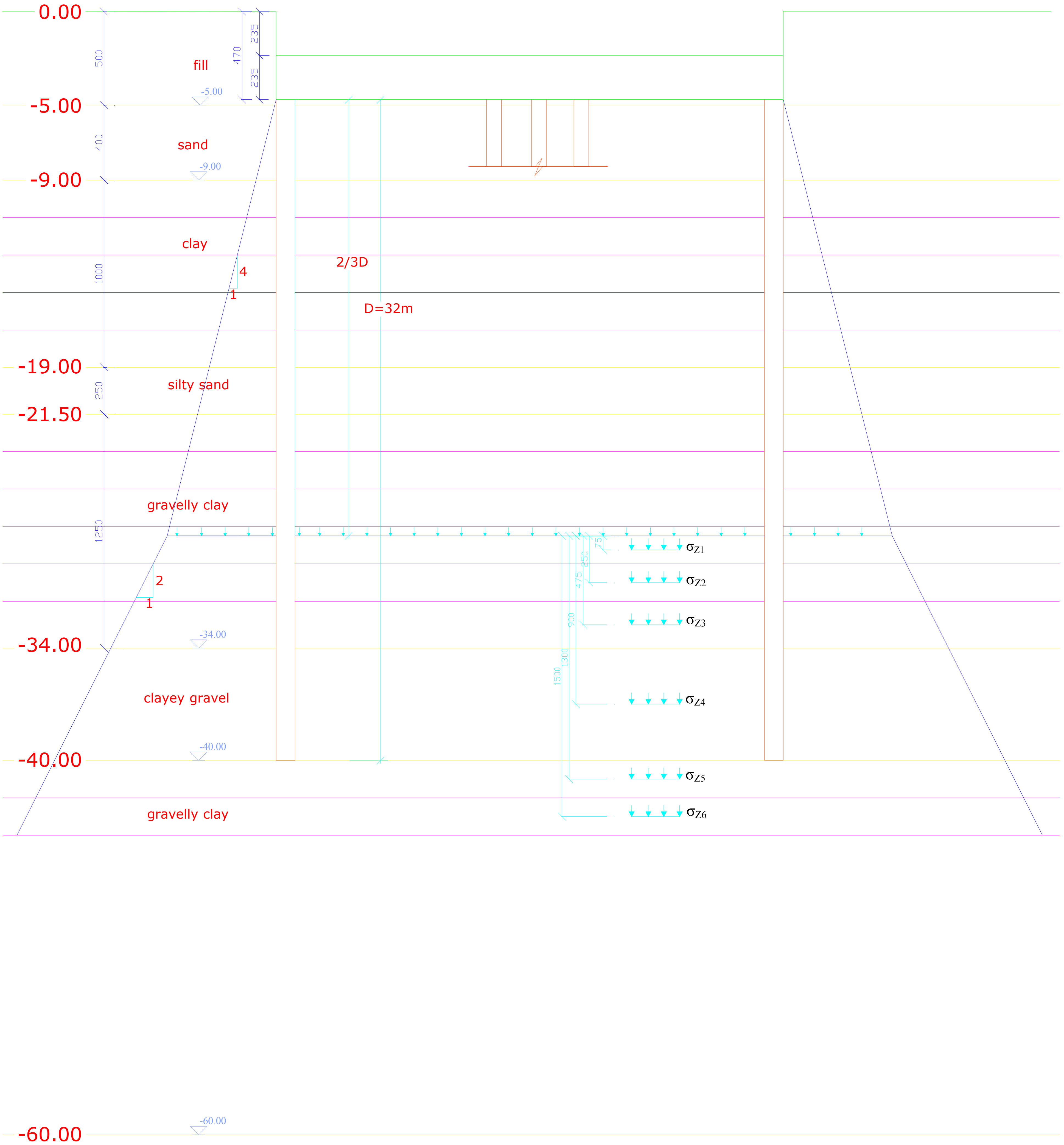
SILTY SAND; $19^{,00}$ - $21^{,50}$													
q_d (kPa)	H (m)	yass (m)	L (m)	B (m)	M	N	I1	I2	Is	If	E (kPa)	μ	ΔH_{e2} (cm)
170,0	14,00	3,1	34	27	1,259	2,07	0,300	0,075	0,343	0,670	5730	0,3	16,7
	16,50				1,259	2,44	0,338	0,068	0,406	0,670			19,8
									Elastic settlement (cm)				3,1

GRAVEL; $34^{,00}$ - $40^{,00}$													
q_d (kPa)	H (m)	yass (m)	L (m)	B (m)	M	N	I1	I2	Is	If	E (kPa)	μ	ΔH_{e2} (cm)
170,0	30,00	3,1	34	27	1,259	4,44	0,454	0,042	0,478	0,670	50000	0,3	2,7
	36,00				1,259	5,33	0,481	0,036	0,517	0,670			2,9
									Elastic settlement (cm)				0,2

$$\Delta H = q_a . B' . \frac{1-\mu^2}{E_s} . 4 . I_s . I_f \quad (Settlement\ in\ the\ middle\ of\ a\ rectangular\ uniform\ loaded\ area)$$

APPENDICE G
PILED FOUNDATION APPLICATION PLAN AND CROSSECTION


1
2
3
4
5
6
7
8
9
10
11
12
3300
50
300
A
B
C
D
E
F
G
H
I
J
200
2600


0.00
-5.00
-9.00
-19.00
-21.50
-34.00
-40.00
-60.00
fill
sand
clay
silty sand
gravelly clay
clayey gravel
gravelly clay
2/3D
D=32m
4
1
2
1
500
400
1000
250
1250
470
235
235
-5.00
-9.00
-34.00
-40.00
-60.00
σ_{Z1}
σ_{Z2}
σ_{Z3}
σ_{Z4}
σ_{Z5}
σ_{Z6}

σ_{Z1}

APPENDICE H
PILED FOUNDATION SETTLEMENT ANALYSES

CONSOLIDATION SETTLEMENT

q_d (t/m^2)	B (m)	L (m)
7,5	38,6	45,6

	Z (m)	$\Delta\sigma$ (t/m^2)	σ' (t/m^2)	H (cm)	C_c	e_o	ΔH_{k1} (cm)
$\Delta\sigma_1$	0,75	7,24	28,50	150	0,19	0,47	**1,9**
$\Delta\sigma_2$	2,50	6,68	30,30	200	0,19	0,47	**2,2**
$\Delta\sigma_3$	4,75	6,05	32,50	200	0,19	0,47	**1,9**
$\Delta\sigma_5$	13,00	4,37	42,20	200	0,19	0,47	**1,1**
$\Delta\sigma_6$	15,00	4,06	44,60	200	0,19	0,47	**1,0**
					TOPLAM		**8,1**

$$Se_0 = wG_s ....S = 1$$

$$e_0(\%) = wG_s$$

$$e_0(\%) = 17.75 \times 2.67$$

$$e_0(\%) = 47$$

$$\Delta H = \frac{C_c H}{1+e_o}\log\frac{\sigma'_v + \Delta\sigma}{\sigma'_v}$$

CH (2.00 - 5.00)

C_c= 0.046+0.0104Ip ....Nakes et al. (1988)
C_c=0.18

C_c=0.00234$w_L G_s$ ……...........................Nagaraj and Srinivasa Murthy (1985, 1986)
C_c=0.20

C_c=0.009(w_L-10)………......................Terzaghi and Peck (1967)
C_c=0.20

Cc=0.01w_N
Cc=18
$C_{c,seçilen}$=0.19

ELASTIC SETTLEMENT

$\Delta\sigma_4$

q_d (t/m^2)	B (m)	L (m)	Z (m)	$\Delta\sigma$ (t/m^2)	σ' (t/m^2)	H (cm)	C'	e_o	ΔH_{k1} (cm)
7,5	38,6	45,6	2,40	6,71	37,40	600	43,00	0,47	**1,00**

$$\Delta H = H\left[\frac{1}{C'}\log\left(\frac{\sigma' + \Delta\sigma}{\sigma'}\right)\right]$$