A COMPARISON OF TWO – DIMENSIONAL AND THREE – DIMENSIONAL FINITE ELEMENT ANALYSIS FOR SETTLEMENT BEHAVIOR OF PILED RAFT FOUNDATIONS

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A COMPARISON OF TWO – DIMENSIONAL AND THREE – DIMENSIONAL FINITE ELEMENT ANALYSIS FOR SETTLEMENT BEHAVIOR OF PILED RAFT FOUNDATIONS

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ABSTRACT

A COMPARISON OF
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In this study, the settlement behavior of the piled raft foundations resting on overconsolidated clays under uniform loading, is investigated for different pile configurations and load levels. A total of 100 plane – strain and three – dimensional finite element analyses are carried out and the results of these analyses are compared both with each other and with the results presented by Reul & Randolph (2004). The material parameters used in the analysis are selected mainly referring to the previous studies cited above on the same subject and slight modifications are made for convenience in the analysis. The analysis method and the applied pile configurations and load levels are directly taken from the reference study, excluding the soil model employed. A drained Mohr – Coulomb failure criterion is employed in the analysis of this study in modeling the soil instead of an elastoplastic model which was used in the analysis of the reference study. The results are evaluated for the average and differential settlements of the foundations and it is seen that; although the average and differential settlements calculated in this study are not always very close to the values calculated in the reference study, the calculated settlement reduction factors due to piles (especially for the average settlements) compared well with the findings of the reference study for all pile configurations and load levels considered. Based on this, a new approach is suggested to estimate the average settlements of the piled raft foundations. Moreover, correction factors are recommended in order to estimate the average settlements of the piled rafts by directly using the programs employed throughout the thesis.

Keywords: Piled raft foundations, settlement, finite element analysis, Plaxis, Abaqus

ÖZ

KAZIKLI RADYE TEMELLERİN OTURMA DAVRANIŞLARININ 2 BOYUTLU ve 3 BOYUTLU SONLU ELEMANLAR ANALİZLERİ İLE KARŞILAŞTIRILMASI

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Bu çalışmada, aşırı konsolide killer üzerinde inşa edilmiş kazıklı radye temellerin düzgün yayılı yük altında ki oturma davranışları, farklı kazık yerleşim planları ve yük seviyeleri için incelenmiştir. 2 boyutlu ve 3 boyutlu analizlerden oluşmak üzere, toplamda 100 adet sonlu elemanlar analizi yapılmış ve sonuçları hem birbirleriyle hem de Reul ve Randolph tarafından 2004 yılında yapılmış bir çalışmanın sonuçları ile karşılaştırılmıştır. Analizlerde kullanılan malzeme parametreleri esas olarak, aynı konu üzerinde daha önce yapılmış çalışmalardan alınmış ve üzerinde analiz tipine uygunluk için gereken değişiklikler yapılmıştır. Kullanılan analiz metodu (zemin malzeme modeli hariç), kazık yerleşim planları ve yük seviyeleri bu tezin karşılaştırmalı analizlerine konu olan daha önce ki bir çalışmadan aynen alınmıştır. Zemini oluşturan kil tabakaların davranışlarının modellenmesinde referans çalışmada kullanılan elastoplastik yenilme kriteri yerine, drenajlı Mohr – Coulomb yenilme kriteri kullanılmıştır. Sonuçlar; temellerin ortalama ve farklı oturmaları için incelenmiş ve her ne kadar bulunan sonuçlar referans çalışmada ki sonuçlarla çokta uyumlu olmasa da, bu çalışmada hesaplanan ve kazıksız radye temellere kazık eklenmesi ile oturmalarda meydana gelen (özellikle ortalama oturmalarda) oransal azalmanın, bu çalışmada gözönünde bulundurulan tüm kazık yerleşimleri ve yük seviyeleri için, referans çalışmada verilen değerlere çok yakın olduğu görülmüştür. Bu noktadan hareketle, kazıklı radye temellerin ortalama oturmalarının hesaplanmasında kullanılabilecek yeni bir yaklaşım geliştirilmiş ve tez kapsamında sunulmuştur. Ayrıca, kullanılan programlar ile kazıklı radye temellerin oturmalarının tahmin edilebilmesi için düzeltme katsayıları yine tez kapsamında önerilmiştir.

Anahtar Kelimeler: Kazıklı radye temeller, oturma, sonlu elemanlar analizi, Plaxis, Abaqus

To My Family…

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CHAPTER 1

INTRODUCTION

The foundations enhanced with deep foundation elements (namely with "piles") have been extensively used in recent decades either to provide enough bearing capacity or to avoid excessive average or differential settlements or tilting of the structure. The traditional piled foundation design is based on to make the pile group carry all the structural load, that is imposed by the overlying structure. However, a raft foundation enhanced with piles is a geotechnical composite structure which consists of three main elements: piles, raft and subsoil; (excluding offshore structures) and neglecting the contribution of the raft to the bearing behavior of the foundation leads to highly overconservative solutions especially for the foundations constructed on stiff soils.

" Piled Rafts " differs from the traditional piled foundations at this point, since the load sharing mechanism between the piles and the raft is taken into account and the piles are used up to a load level that can be of the same order of magnitude as the bearing capacity of a comparable single pile or even greater. Therefore, employing a piled raft foundation allows reduction of average and differential settlements, in a very economic way compared to the traditional piled foundations.

In this study, the average and differential settlement behavior of the piled rafts for different pile and load configurations is investigated both in two and three dimensional models via a well – known finite element software “PLAXIS” . The investigated pile and load configurations are taken from a past research of Reul & Randolph (2004) in which the finite element code “ABAQUS” was employed for the analysis and the obtained results were very close to the situation in reality since the models used were calibrated according to the past measurements. The results of this study are compared with the results obtained from the study of Reul & Randolph, and a conclusion is made on the usability of the “PLAXIS” finite element software for the design of piled raft foundations. Also, some recommendations for constructing the computer models for the analysis of piled rafts are developed and discussed throughout the thesis.

After a brief introduction in Chapter 1, a detailed literature review which includes nearly all aspects of the “piled raft” issue as well as the codes developed for the solution of the problem is provided in Chapter 2. The materials and the method used in the finite element analyses of the thesis is described in Chapter 3 together with the summary of the reference studies on which this thesis is based. The details of the model and the mesh used in the finite element analyses are given within Chapter 4 which also includes the results of the performed analyses together with the recommended method of the thesis. Finally, Chapter 5 includes the summary and the conclusions.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Although the piled rafts are investigated in detail by many researchers especially in the recent decades, a well – established design procedure is still not available. However, an enormous progress is achieved in understanding the behavior of piled rafts and some researchers developed some design recommendations based on their approaches to the problem. Also, there are some local codes which provides a basic guideline for the design of piled rafts. In this chapter, after a short introduction of the “piled raft concept”, the widely accepted main approaches and local codes for the design of piled rafts will be discussed.

2.2 Piled Rafts

2.2.1 The “Piled Raft “ Term

Piled rafts are defined as geotechnical composite structures combined of three elements : piles, raft and subsoil ; which takes into account the contribution of the raft to the overall bearing behavior; by Reul & Randolph (2003, 2004). Katzenbach et al. (2001, 2004, 2005) named this system as “Combined Pile Raft Foundation” (CPRF). There are some other researchers calling this system as “Pile – Raft Foundations” or “Piled Raft Foundations “ . The term of “Piled Raft” will be used to address the described system throughout the thesis.

2.2.2 The Piled Raft Concept

Piled raft foundations are especially favorable for the constructions founded on stiff soils like dense sands or overconsolidated clays (where the contact between the raft and the subsoil is unlikely to be lost), where adequate bearing capacity with an allowable safety margin is satisfied by the raft alone but the piles are introduced to act as settlement reducers either to reduce the average or differential settlements. However, there are some recent researches which enable the use of piled rafts on soft soils.

The key problem that arises in the design of piled rafts is to determine the load proportion between the piles and the raft, in other words the amount of load carried either by the piles or raft. Correspondingly, all the recommended methods first aim to determine the load proportion of the system by comparing the stiffnesses of the pile group and raft, before going further into the design.

2.2.3 Basic Definitions

- Average Settlement (s_{avg}) : Average settlement is defined as the average of the settlements at the center and corner of the piled raft.
- Differential Settlement (Δs) : Differential settlement is defined as the difference between the settlements of the center and corner of the piled raft.
- The Piled Raft Coefficient (α_{pr}) : The piled raft coefficient is defined as the ratio of the load carried by the piles (ΣP_{pile}) to the total load carried by the whole system(P_{tot}).

$$\alpha_{pr} = \frac{\sum P_{pile}}{\sum P_{tot}} \tag{2.1}$$

A piled raft coefficient of unity indicates a free-standing pile group, whereas a piled raft coefficient of zero describes an unpiled raft. The piled rafts fall in the range of $0 < \alpha_{pr} < 1$, which indicates the contribution of the raft to the system.

Below in the Fig 2.1 a typical relationship between the settlement and the piled raft coefficient can be seen.

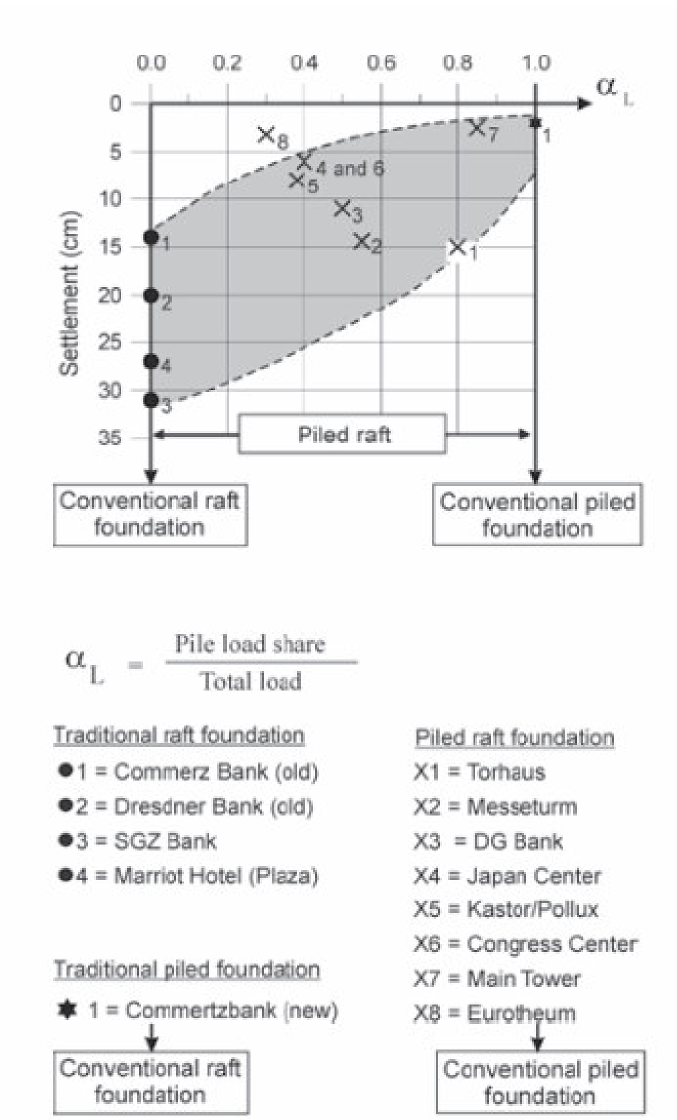


Fig 2.1 Settlement Behaviour of High – Rise Buildings in Frankfurt, Germany

(El – Mosallamy, 2008)

There are some other definitions related to the methodologies of the researchers and these will be described later while describing the related researcher's approaches to the discussed issue.

2.3 Main Approaches

Since, the behavior of the piled rafts is of big concern in recent decades through the researchers, there are various approaches developed by different researchers, each of which looks from a different aspect to the same problem. However, there are some approaches which are one step forward beyond others and they generally belong to the founders of the issue. In this part, the three main approaches which are widely accepted through the world will be introduced.

2.3.1 Poulos Approach

In the design procedure developed by Poulos, at first the favorable and unfavorable conditions for the application of a piled raft system are described. Then the analysis techniques are classified according to their capabilities, theoretical backgrounds, etc… Then finally, a 3 staged design process is recommended which involves both geotechnical and structural computations.

2.3.1.1 Favorable and Unfavorable Circumstances for Piled Rafts (Poulos, 2001)

The most effective application of piled rafts occurs when the raft can provide adequate load capacity, but the settlement and/or differential settlements of the raft alone exceed the allowable values. Poulos (1991) has examined a number of idealized soil profiles, and has found that the following situations may be favorable:

a) soil profiles consisting of relatively stiff clays
b) soil profiles consisting of relatively dense sands

In both cases , the raft alone can provide all or at least much of the required load capacity and the piles are added to boost the performance of the foundation.

In contrast, there are some soil profiles which are classified as unfavorable by Poulos :

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

The adequate bearing capacity may not be provided in the first two cases and the settlement at the compressible layers may weaken the contact between the raft and subsoil for the third case. In the fourth case, the contact between the raft and the subsoil may be lost. At the final case, due to swelling, additional tensile forces may be induced in the piles which results in a decrease in the load capacity of the piles.

2.3.1.2 Methods of Analysis of Piled Rafts (Poulos et al., 1997)

Methods of analysis for piled rafts are categorized into three main groups. These are :

a) Simplified Analysis Methods
b) Approximate Computer Methods
c) More Rigorous Computer Methods

a) Simplified Analysis Methods :

There are two main simplified analysis methods which enable to evaluate the performance of the piled rafts. These are the equivalent raft and equivalent pier methods. These methods are extensively used also in the analysis of traditional piled foundations and many contributions to these methods by different researchers are available in the literature and some will be included here.

i. Equivalent Raft Method :

Traditionally, the settlement of a pile group is estimated by locating an imaginary "equivalent raft" at two-thirds of the way down the part of the piles which penetrate the main founding stratum, or at the level of the pile bases for end bearing piles. (See Fig 2.2)

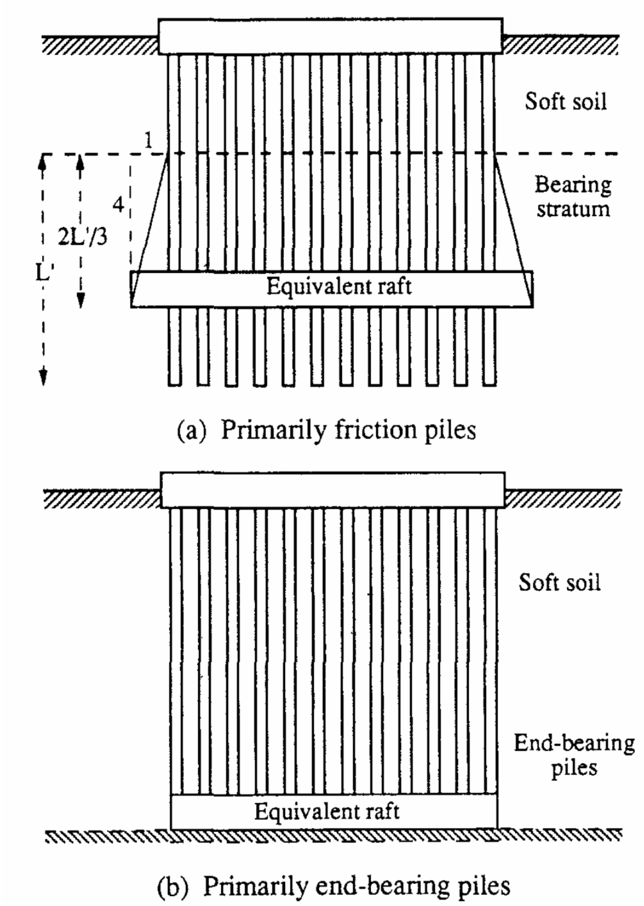


Fig 2.2 Equivalent Raft Approach for Pile Groups
(Randolph, 1994)

The average settlement at ground level (w_{avg}) is calculated as the total of the settlement of the raft (w_{raft}) and the elastic compression of the piles above the level of the equivalent raft (Δw), treated as free-standing columns.

$$w_{avg} = w_{raft} + \Delta w \tag{2.2}$$

As seen in Fig 2.2, generally a load spread of 1:4 is assumed to evaluate the size of the equivalent raft and the settlement of the raft (w_{raft}) is calculated as:

$$w_{raft} = F_D * q * \sum_{i=1}^{n} \left(\frac{I\varepsilon}{E_s} \right)_i * h_i \tag{2.3}$$

where :

F_D : embedment correction factor for raft settlement from Fox (1948)

q : applied pressure

$I\varepsilon$: influence factor for calculation of vertical strain

E_s : Young's Modulus of soil

h_i : thickness of the i^{th} soil layer

i : number of the soil layer

The main advantage of the equivalent raft method is that, one can take into account the variations in soil stiffnesses below the level of the raft. This is especially an important advantage when there exists a softer soil layer at a level below the base of the piles.

ii. Equivalent Pier Method :

An alternative simplified analysis method is the equivalent pier method which considers the region of soil in which the piles are embedded as an equivalent continuum, effectively replacing the pile group by an equivalent pier (Poulos & Davis, 1980). (See Fig 2.3)

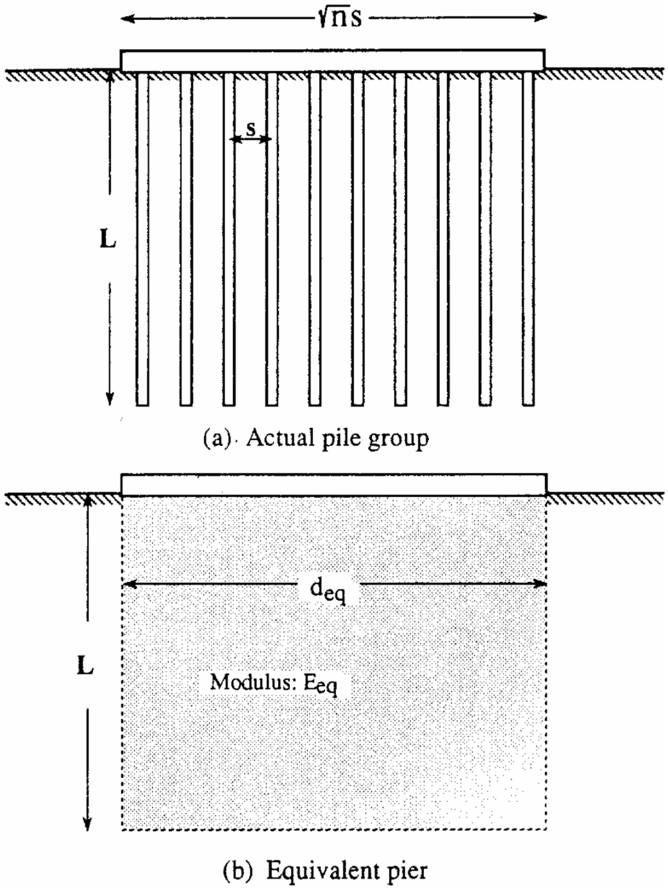


Fig 2.3 Replacement of Pile Group , or Piled Raft, by Equivalent Pier
(Randolph, 1994)

For a pile group with an area of A_g, the equivalent diameter of the pier is defined as:

$$d_{eq} = \sqrt{\frac{4 * Ag}{\pi}} = 1.13\sqrt{Ag} \quad (2.4)$$

and the equivalent Young's modulus of the pier is defined as :

$$E_{eq} = E_s + (E_p - E_s) * \left(\frac{Ap}{Ag}\right) \quad (2.5)$$

where ;

E_p : Young's Modulus of the Piles

E_s : average Young's Modulus of the soil penetrated by the piles

A_p = total cross-sectional area of the piles in the group

Through the investigations, it was observed that the circumstances at which either the equivalent raft or equivalent pier methods shows a better performance depends mainly on the ratio of *l/s*. As a result, Clancy & Randolph (1993) has developed an appropriate parameter "R" , to categorize the pile groups :

$$R = \sqrt{\frac{n * s}{l}} \qquad (2.6)$$

where :

n : number of piles

s : spacing between piles

l : embedded length of the pile

For pile groups with an R larger than 4 equivalent raft method was evaluated as suitable whereas equivalent pier method was observed to be more suitable for the cases with an R less than 2.

Both the equivalent raft and the equivalent pier methods have nothing to do with the differential settlement but they may be used just to estimate the average settlements of a piled raft. However, there are some relationships developed by Randolph & Clancy (1993) at least to give a sense about the order of magnitude of the differential settlements:

$$\frac{\Delta w}{w_{avg}} \approx f * \frac{R}{4} \qquad \text{for R} \leq 4$$

$$\frac{\Delta w}{w_{avg}} \approx f \qquad \text{for R} > 4 \tag{2.7}$$

Where f = 0.3 for center to mid-side and f = 0.5 for center to corner.

b) Approximate Computer Methods :

There are two approximate computer methods available in the literature, recommended by Poulos for the solution of foundation systems. These are:

i. Strip on Springs Method (Poulos, 1991)
ii. Plate on Springs Method (Poulos, 1994)

i. Strip on Springs Method :

In the "Strip on Springs Method", the raft is represented by a strip and the raft – soil and pile – soil contacts are represented by springs. All the interactions that occur between pile – pile, pile – raft and raft – raft are taken into account.

ii. Plate on Springs Method :

In this method, the raft is modeled as a plate with elastic behavior and the contact areas with soil both beneath the raft and piles are represented by springs. Also, the soil is modeled as an elastic continuum.

c) More Rigorous Computer Methods :

Following the great advances in the computer technologies, performing more detailed and professional analysis have become possible. As a result, more complex methods, which take into account the effects that were disregarded by the aforementioned methods, are developed and these methods are becoming better day – by – day with the contributions of various researchers. These methods are as follows:

i. Boundary Element Method
ii. Finite Element Method
iii. Combination of Boundary Element and Finite Element Methods

i. Boundary Element Method :

In this method, an appropriate Green's function is adopted to the system, to relate the average displacement of each element to the traction on each element. Moreover, the interfaces between the soil, raft and pile are fully taken into account.

Butterfield and Benarjee (1971), was first to study the piled rafts with the boundary element method, analyzing a pile group with a rigid cap. Then a more detailed analysis was performed by Kuwabara (1989) employing an elastic boundary element analysis and modeling both free- standing pile groups and piled rafts. Some more contributions to the method was made by several researchers and they are available in the literature.

ii. Finite Element Method :

In the finite element method, all the structural and geotechnical parts of the analyzed system, are modeled using finite elements. The non – linear behavior of all materials can be included in the model. The analysis performed via finite element method can be investigated in two categories:

- ✓ Plane Strain (2 – D) Analysis :

In this type of analysis, the system is modeled in two dimensions assuming the third dimension is infinite. Plane strain analysis is suitable and widely used for practical engineering purposes. However, since the model is constructed in two dimensions, only regular loadings can be modeled. Moreover, torsional moments of the raft can not be obtained by using this analysis. In recent years, there is much effort on developing modeling methods for the optimization of plane – strain solutions (Prakoso & Kulhawy, 2001).

- ✓ Three Dimensional (3 – D) Analysis :

Three dimensional finite element analysis is the most appropriate way of analysis for piled rafts, since the complex behavior of the system can be fully modeled. It is sure to obtain the most accurate and precise results by using this type of analysis, provided that the necessary parameters are assigned appropriately. Ta & Small (1996), Katzenbach et al (1998), Reul & Randolph (2003), Reul & Randolph (2004) are some of the researches that were based on three – dimensional finite element analysis of piled rafts. Through the researches discussed above, the Reul & Randolph (2003) and Reul & Randolph (2004) are one step forward beyond others for this thesis, since they constitute the base for this thesis which will be described in the following chapters.

iii. Combination of Boundary Element and Finite Element Methods :

The boundary element and finite element methods were combined by Hain & Lee in 1978. In this combined analysis, the boundary element method is employed to estimate the pile behavior whereas the finite element method is used to model the rafts as thin plate finite elements.

2.3.1.3 Design Procedure Recommended by Poulos (1980 – 2001)

The fundamentals of the method that will be described below was established by Poulos (1980) and with new developments during years, it was revised as the latest version in 2001. A three – staged design procedure is recommended by Poulos (2001), which can be handled mostly by simple calculations and needs more detailed computer analysis only at the final stage. The defined stages are as follows :

a) a preliminary stage to assess the feasibility of using a piled raft, and the required number of piles to satisfy design requirements
b) a second stage to assess where piles are required and the general characteristics of the piles
c) a final detailed design stage to obtain the optimum number, location and configuration of the piles, and to compute the detailed distributions of settlement, bending moment and shear in the raft, and the pile loads and moments

a) Preliminary Design Stage :

In this stage, first of all the vertical bearing capacity of the raft alone should be determined. If the raft alone can provide a small portion of the necessary bearing capacity, then the foundation will need to be designed with the conventional philosophy. On the contrary, if the raft alone can provide all or most of the necessary bearing capacity, then the piles will be added only to boost the performance of the raft.

After determining whether a piled raft system is suitable for the situation or not, the vertical capacity should be determined for a preliminary piled raft system. From now on, the procedure needs back and forward calculations to optimize the pile group and load share between the raft and the piles. The ultimate bearing capacity of a piled raft can be taken as the lesser of the following two values :

- ✓ the sum of the ultimate capacities of the raft plus all the piles
- ✓ 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 piles

For estimating the load – settlement behavior and the load sharing mechanism of the piled rafts, at first the method described by Poulos & Davis (1980) as employed. However, in the latest edition, the method developed by Randolph (1994) was adopted by Poulos to the design procedure. The definition of the pile problem considered by Randolph is shown in Fig 2.4.

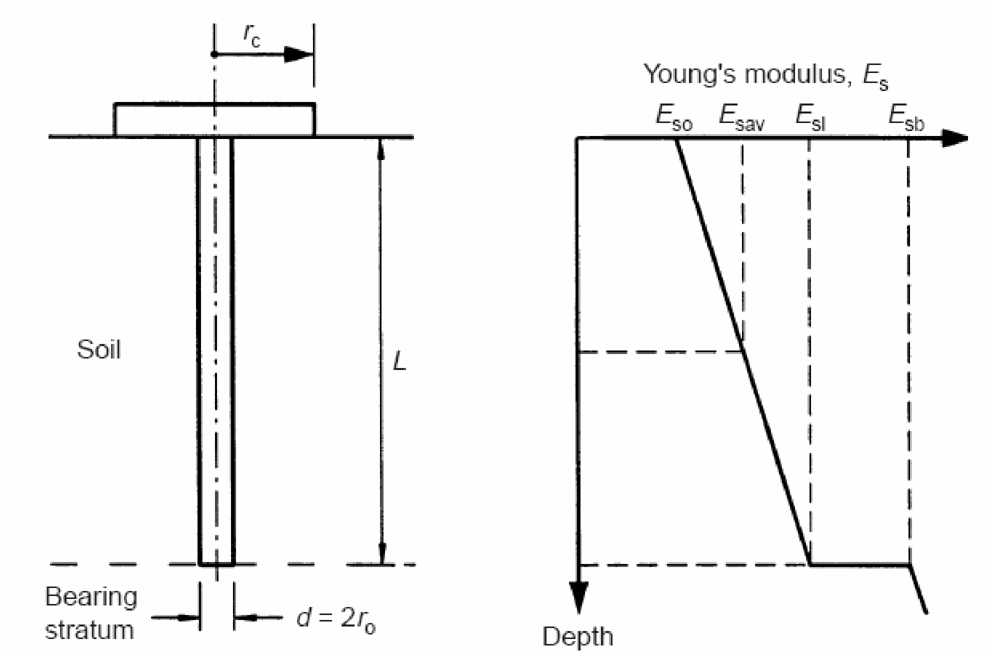


Fig 2.4 Definition of the Problem
(Poulos, 2001)

According to the Randolph (1994) the stiffness of the piled raft foundation can be estimated as follows:

$$K_{pr} = \frac{K_p + K_r * (1 - \alpha_{cp})}{1 - \alpha_{cp}^{\ 2} * K_r * K_p} \tag{2.8}$$

where,

K_{pr} : stiffness of piled raft

K_p : stiffness of the pile group

K_r : stiffness of the raft alone

α_{cp} : raft – pile interaction factor

Both K_r and K_p can be estimated via elastic theory. For estimating K_r, solutions of Fraser & Wardle (1976) or Mayne & Poulos (1999) may be employed. Similarly, solutions of Poulos & Davis (1980), Fleming et al. (1992) or Poulos (1989) may be used to estimate the K_p.

The proportion of the total applied load carried by the raft is :

$$\frac{P_r}{P_t} = \frac{K_r * (1-\alpha_{cp})}{K_p + K_r * (1-\alpha_{cp})} = X \qquad (2.9)$$

where,

P_r : load carried by the raft

P_t : total applied load

The raft – pile interaction factor, α_{cp}, can be estimated as follows :

$$\alpha_{cp} = 1 - \frac{\ln(r_c / r_o)}{\zeta} \qquad (2.10)$$

where,

r_c : average radius of the pile cap (corresponding to an area equal to the raft area divided by number of piles)

r_o : radius of pile

ζ : $\ln(r_m/r_o)$

r_m : $0.25 + \xi[2.5\rho(1-\upsilon)-0.25]*L$

ξ : E_{sl} / E_{sb}

ρ : E_{sav} / E_{sl}

υ : Poisson's ratio of soil

L : pile length

E_{sl} : soil Young's modulus at level of pile tip

E_{sb} : soil Young's modulus of bearing stratum below pile tip

E_{sav} : average soil Young's modulus along pile shaft

The equations described above can be used to establish a tri-linear load – settlement curve as shown in Fig2.5. First, the stiffness of the piled raft system should be computed using equation (2.8). This stiffness will remain operative until the pile capacity is full reached. Making the simplifying assumption that the pile load mobilization occurs simultaneously, the total applied load , P_1 , at which the pile capacity is reached can be obtained by :

$$P_1 = \frac{P_{up}}{1-X} \qquad (2.11)$$

where,

P_{up} : ultimate load capacity of the piles in the group

X : proportion of the load carried by the raft

As it can be clearly seen from Fig 2.5, beyond point A (where the pile capacity is fully mobilized) , the system's stiffness is equal to that of the raft alone until point B, where the raft capacity is also fully utilized. Beyond point B, the relationship becomes horizontal.

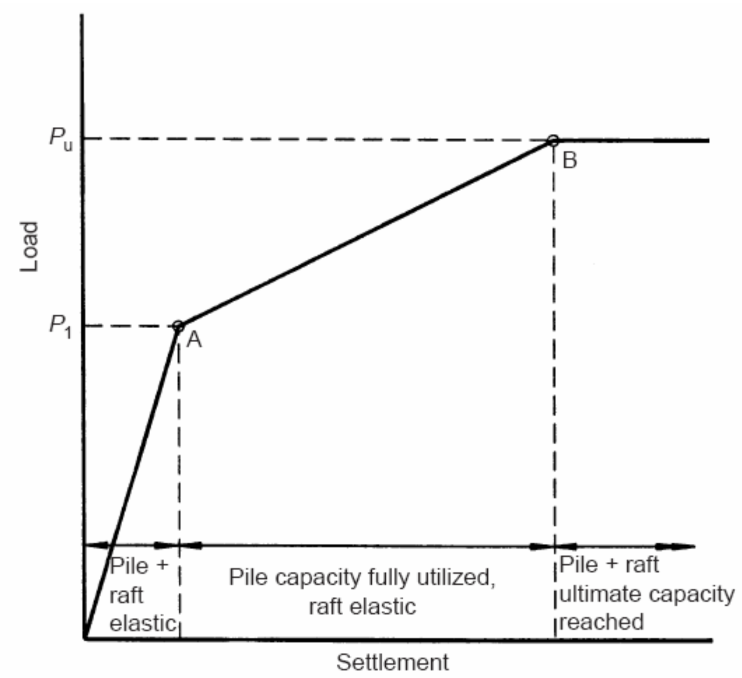


Fig 2.5 Simplified Load – Settlement Cvurve for Preliminary Analysis (Poulos, 2001)

b) Second Stage of Design : Assessment of piling requirements

Most of the available methods in the literature for piled foundation design, considers a uniform load distribution for the loads acting on the raft. This may be an adequate assumption for the preliminary stage, however, it is not adequate for locating the piles beneath the raft. The piles should be located according to the concentration of the column loadings. In this stage, a methodology developed for locating the piles according to the column loadings will be described.

The model studied for this methodology is presented below in Fig 2.6. The model involves a circular column with a radius of “c” and a concentrated load of “P”. The raft ,which is modeled as a semi – infinite elastic element, has a thichness of “t”, a Young’s modulus of E_r and a poisson’s ratio of $\upsilon_{r.}$ The soil is modeled as an elastic layer with great depth having a Young’s modulus of E_s and a poisson’s ratio of υ_s.

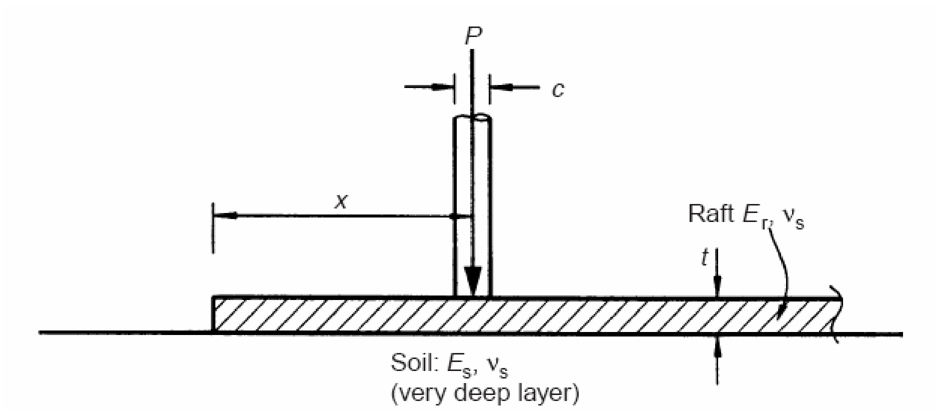


Fig 2.6 Definition of Problem for an Individual Column Load (Poulos,2001)

There are four circumstances in which piles may be needed to be added below a column :

i. if the maximum moment in the raft below the column exceeds the allowable value for the raft

ii. if the maximum shear in the raft below the column exceeds the allowable value for the raft

iii. if the maximum contact pressure below the raft exceeds the allaowable design value for the soil

iv. if the local settlement below the column exceeds the allowable value

i. Maximum Moment Criterion :

The maximum moments M_x and M_y below a column of radius c acting on a semi – infinite raft are given by :

$$M_x = A_x*P \tag{2.12}$$

$$M_y = B_y*P \tag{2.13}$$

where,

$A_x = A - 0.0928\ln(c/a)$

$B_y = B - 0.0928\ln(c/a)$

A, B : coefficients depending on δ/a

δ : distance of the column center line from the raft edge

$a = t*[E_r(1 - \upsilon_s^2)/6E_s(1 - \upsilon_r^2)]^{1/3}$ (characteristic length of the raft)

* The coefficients A and B are plotted as a function of the relative distance x/a in Fig 2.7 :

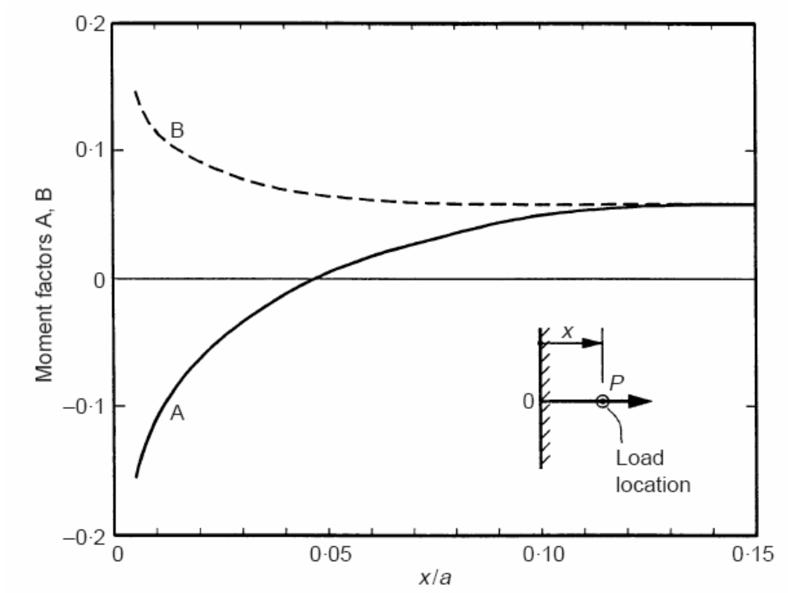


Fig 2.7 Moment Factors A, B for Circular Column
(Poulos, 2001)

The maximum column load ,P_{c1}, that can be carried without exceeding the allowable moment is then given by :

$$P_{c1} = M_d \ / \ \text{larger of } A_x \text{ and } B_y \tag{2.14}$$

where,

M_d : design moment capacity of the raft

ii. Maximum Shear Criterion :

The maximum shear, V_{max} , below a column is :

$$V_{\max} = \frac{(P - q\pi c^2)c_q}{2\pi c} \tag{2.15}$$

where,

q : contact pressure below raft

c_q : shear factor (plotted in Fig 2.8 as a function of x/a)

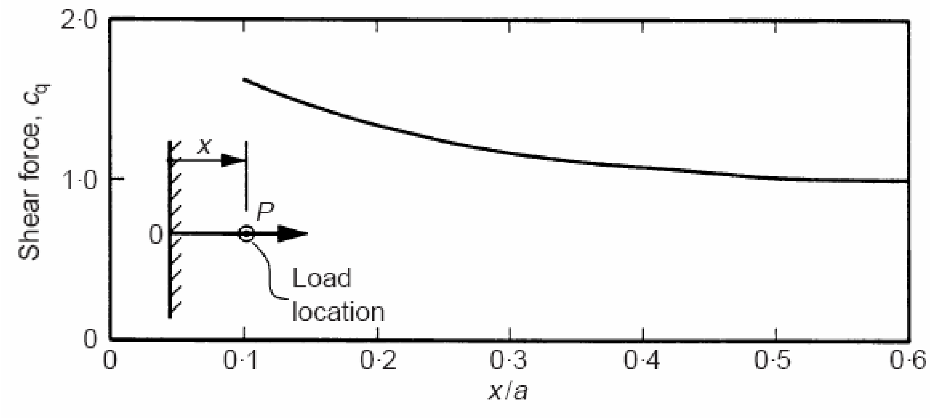


Fig 2.8 Shear Factor ,c_q, for Circular Column

(Poulos, 2001)

Then, the maximum column load that can be applied without exceeding the design shear capacity of the raft is:

$$P_{c2} = \frac{V_d 2\pi c}{c_q} + q_d \pi c^2 \tag{2.16}$$

where,

V_d : design shear capacity of the raft

q_d : design allowable bearing pressure below raft

iii. Maximum Contact Pressure Criterion :

The maximum contact pressure on the base of the raft, q_{max}, can be estimated as:

$$q_{\max} = \frac{\bar{q}\ P}{a^2} \tag{2.17}$$

where,

$\bar{q}$: factor plotted in Fig 2.9

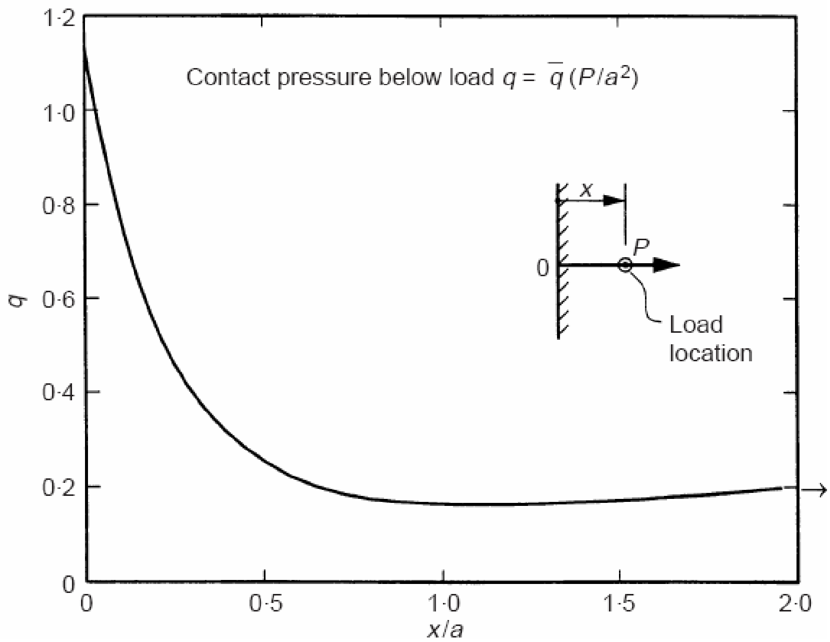


Fig 2.9 Contact Pressure Factor, q

(Poulos, 2001)

Then, the maximum column load that can be applied without exceeding the allowable contact pressure is:

$$P_{c3} = \frac{q_u a^2}{F_s \bar{q}} \tag{2.18}$$

where,

q_u : ultimate bearing capacity of soil below raft

F_s : factor of safety for contact pressure

iv. Local Settlement Criterion :

The settlement below a column is obtained as:

$$S = \frac{w(1-\upsilon_s^{\,2})P}{E_s.a} \tag{2.19}$$

where,

w: settlement factor plotted in Fig 2.10

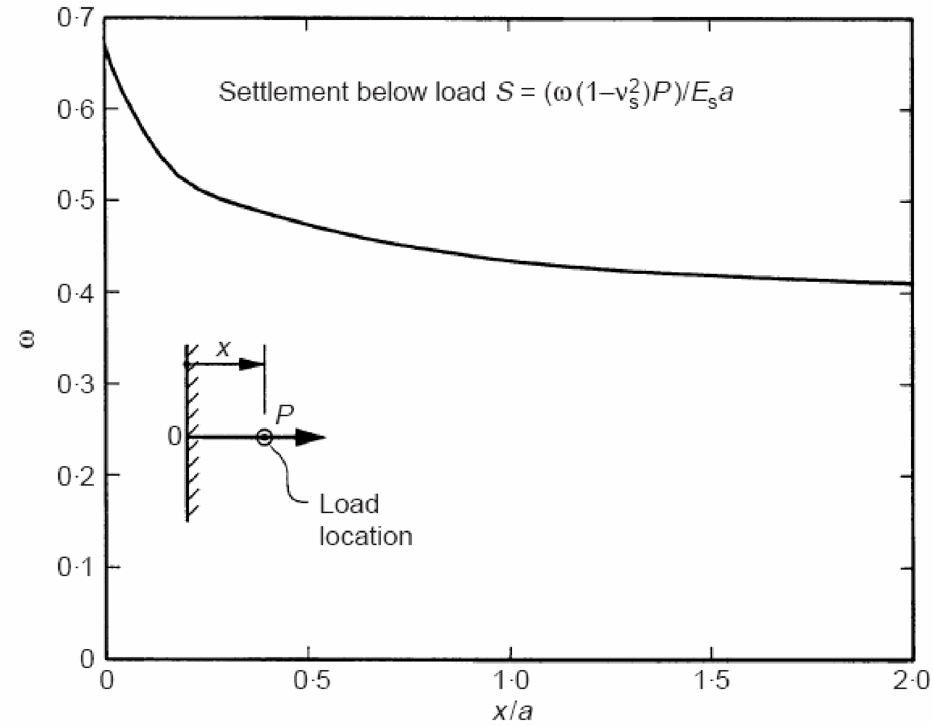


Fig 2.10 Settlement Factor, w
(Poulos, 2001)

If the allowable local settlement is S_a, then the maximum column load, P_{c4}, so as not to exceed this value is :

$$P_{c4} = \frac{S_a E_s a}{w(1-\nu_s^2)} \tag{2.20}$$

- Assessment of Pile Requirements for a Column Location :

If the applied column load, P_c, exceeds the P_{crit} value, which is equal to the minimum of P_{c1}, P_{c2}, P_{c3}, P_{c4}, then piles need to be added beneath the raft at that location. That is ;

$$\text{If } P_c > P_{crit} \Rightarrow \text{piles need to be added} \qquad (2.21)$$

If the critical criterion is maximum moment, shear or contact pressure, then the piles are to be added to provide additional bearing capacity. Since, 90% of the ultimate capacity of the piles is mobilized according to Burland (1995), the ultimate load capacity of the piles can be obtained as follows:

$$P_{ud} = 1.11 * F_p * (P_c - P_{crit}) \qquad (2.22)$$

where,

F_p : factor of safety for piles (can be taken as unity for piles designed as settlement reducers)

If the critical criterion is the local settlement, then the piles are to be added to provide additional stiffness to the foundation system. The target stiffness of K_{cd}, for an allowable settlement of S_a is:

$$K_{cd} = P_c / S_a \qquad (2.23)$$

The stiffness of the needed pile group ,K_p, can be obtained by solving equation (2.24), as a first approximation.

$$K_p^2 + K_p[K_r(1 - 2\alpha_{cp}) - K_{cd}] + \alpha_{cp}^2 K_r K_{cd} = 0 \qquad (2.24)$$

where,

K_r: stiffness of the raft around the column (can be estimated as the stiffness of a circular foundation having a radius equal to the characteristic length provided that this does not lead to a total raft area that exceeds the actual area of the raft.

d) Final Stage of Design (Optimization via Computerized Methods) :

The final stage of the recommended procedure contains the optimization of the piled raft system initialized by the first two stages, via a suitable software, by changing the locations, lengths and configurations of the piles.

Poulos (2001) had presented a comparison of the methods discussed above by solving a hypothetical raft with 9 piles. Here, the solutions for average and differential settlements, moment and % load on piles are presented. In the Fig 2.11, "Poulos&Davis" corresponds to the method founded by Poulos&Davis in 1980; "Randolph" to the method of Randolph (1994); "Strip" to the method of strip on springs developed by Poulos (1991), "Plate" to the method of plate on springs developed by (Poulos 1994), "FE Ta & Small" to the finite element method developed by Ta & Small (1996), " FE + BE Sinha" to the combined method developed by Sinha (1996). Also, there is a table provided by Poulos (2001) in which the capabilities of various methods are summarized. The table is provided below as Table 2.1

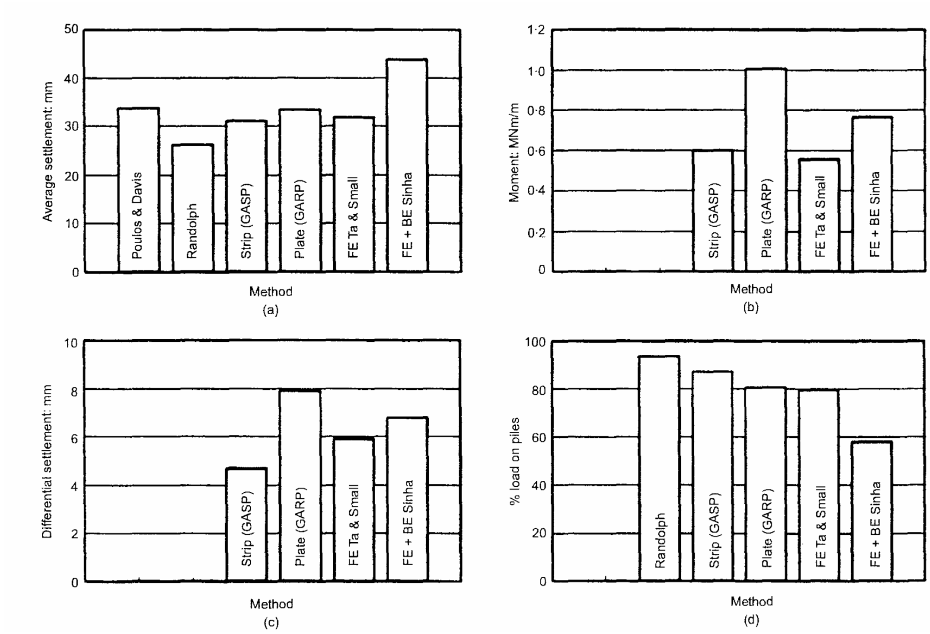


Fig 2.11 Comparative Results for Hypothetical Example (raft with 9 piles, total load=12 MN) (Poulos, 2001)

2.3.2 Randolph Approach

Randolph is one of the main researchers who established the fundamentals of the piled raft concept. The basic methodology recommended by Randolph is very similar to that recommended by Poulos which was described above. In fact, some of the equations like 2.7, 2.8 of Poulos method are the ones which are directly developed by Randolph (1994). However, there are some differences in the approach of Randolph in categorizing the design philosophies. Correspondingly, in this part of the thesis, the Randolph's classification system and some basic ideas of him related with the design of the piled rafts will be discussed rather than giving a full design procedure like did in the last part. Also, it should be mentioned that, Randolph suggests a detailed finite element analysis in his most recent researches, especially in the ones with Reul, for the design of piled rafts rather than an analytical solution.

Table 2.1 Comparison of Various Methods

(Poulos, 2001)

Method	Response Characteristics					Problem Modelling			
	Settlement	Differential settlement	Pile loads	Raft bending moment	Torsional shear	Non-linear soil	Non-linear pile	Non-uniform soil	Raft flexibility
Poulos & Davis (1980)	✓						✓		
Randolph (1983)	✓		✓						
Van Impe & Clerq (1995)	✓	✓							
Equivalent Raft (Poulos, 1994)	✓	✓							
Brown & Wiesner (1975)	✓	✓	✓	✓					✓
Clancy & Randolph (1993)	✓	✓	✓	✓	✓			✓	✓
Poulos (1994)	✓	✓	✓	✓	✓	✓	✓	✓	✓
Kuwabara (1989)	✓		✓						
Hain & Lee (1978)	✓	✓	✓	✓	✓		✓		✓
Sinha (1997)	✓	✓	✓	✓	✓	✓	✓		✓
Franke et al (1994)	✓	✓	✓	✓	✓		✓		✓
Hooper (1973)	✓	✓	✓	✓		✓		✓	✓
Hewitt & Gue (1994)	✓	✓	✓	✓				✓	✓
Lee et al (1993)	✓	✓	✓	✓	✓			✓	✓
Ta & Small (1996)	✓	✓	✓	✓	✓			✓	✓
Wang (1995)	✓	✓	✓	✓	✓	✓	✓	✓	✓
Katzenbach et al (1998)	✓	✓	✓	✓	✓	✓	✓	✓	✓

2.3.2.1 Randolph's Design Philosophy (Randolph, 1994)

Randolph classifies the design approaches for piled rafts in three main categories. These are as follows:

a) Conventional Design
b) Creep Piling
c) Differential Settlement Control

a) Conventional Design :

In "Conventional Design" , the foundation system is designed as a traditional pile group, spreading the piles with a regular spacing over the foundation area. The only difference from a traditional piled foundation is that, the raft is allowed to carry some part of the load. The piles, generally, are designed to carry the 60 – 75 % of the total load. The principle benefit of the system is the reduction in the number of piles without a significant increase in the average settlement of the foundation system.

A very interesting case history about this type of design was presented by Cooke et al. (1981). The foundation of a structure was designed with 350 piles installed at a rectangular grid spacing of 3.6 m. More detailed analysis by Randolph & Clancy (1993) has shown that, the same foundation system could have been designed with less than 100 piles, without any significant increase in the average settlement.

b) Creep Piling :

This method, has been proposed by Hansbo & Källström (1983) for relatively soft cohesive soils. There are two main principles behind the method:

- ✓ Each pile is designed to operate at about 70 – 80 % of their ultimate bearing capacity, at which significant creep starts to occur
- ✓ Sufficient piles are included to reduce the net contact pressure between raft and soil to below the preconsolidation pressure of the clay

The main logic behind the method is that, the foundation is designed mainly as a raft foundation and the piles are introduced to reduce the total settlement of the system. The piles are distributed uniformly beneath the raft and allowed to move plastically relative to the surrounding soil. The bending moments of the raft can be more precisely determined with this method, since the choice of creep load as the working load of each pile prevents high loads developing in piles at the edges of the foundation.

There is a case history available in the literature described by (Hansbo, 1993) ,illustrated in Fig 2.12, which compares two buildings one on conventional piles and one on creep piles. A major reduction both in number and the length of the piles can be observed with a little difference in the performance of the foundation.

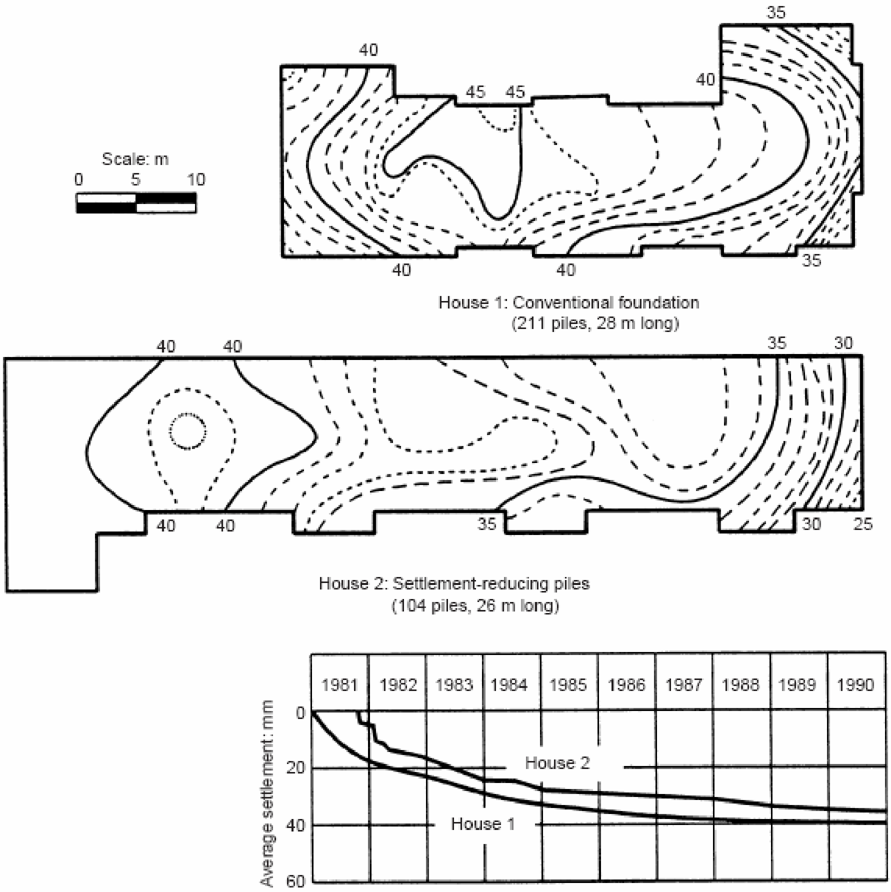


Fig 2.12 Settlement Performance of Two Buildings, one on conventional piles and one on creep piles (After Hansbo, 1993)

The method has been also investigated for the case of non-cohesive soils. It has been shown that, the transmission of the load from pile cap to the ground leads to an increase in the ultimate capacity of the piles, which prevents the piles to operate at creep loads. As a result, the foundation performance of a creep-piled foundation on non-cohesive soils can be analyzed by the conventional means described earlier.

There is also an extreme version of "creep piling" named as "extreme creep piling" in which the piles are designed to operate at their full geotechnical capacity. (Poulos, 2001)

c) Differential Settlement Control :

Differing from the two methods described above, this method does not recommend a uniform distribution of piles beneath the raft. In the two methods described, the main aim is to reduce the total settlements whereas in this method the main aim is to directly reduce the differential settlements employing a smart pile configuration, without necessarily reducing the total settlements of the system.

The figures Fig 2.13 and Fig 2.14 below, summarizes the logic behind the method. An unpiled raft shows a tendency to dish towards the center under uniform loading. So, adding a few piles in the central area of the raft, probably working at a load level close to their ultimate capacity, will reduce the dishing tendency of the raft towards the center which will result in a significant reduction in the differential settlements of the system.

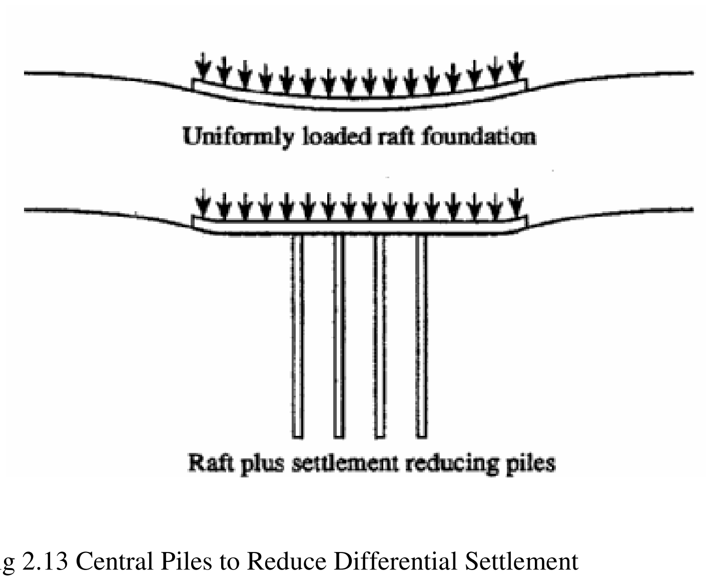


Fig 2.13 Central Piles to Reduce Differential Settlement
(Randolph, 1994)

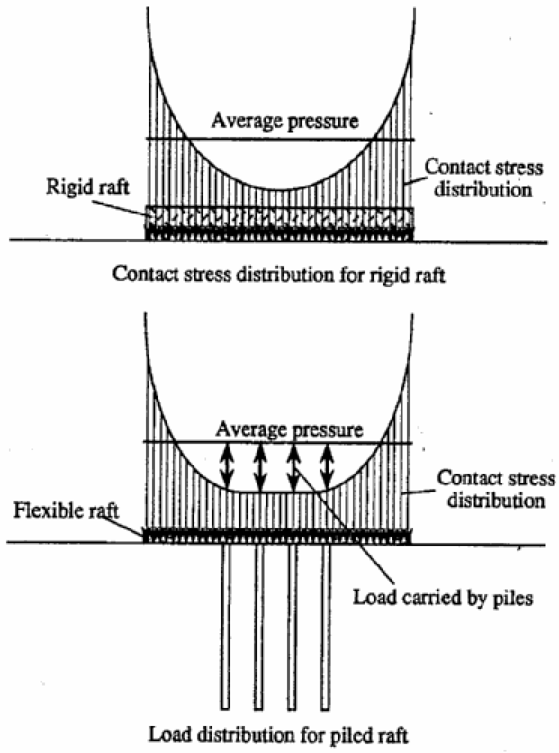


Fig 2.14 Schematic Design Approach for Settlement Reducing Piles
(Randolph, 1994)

Since the main idea is to make the raft behave as a nearly perfectly rigid raft, the piles should be designed to carry the 50 – 70 % of the structural load since the central pressure is approximately half the average applied pressure. So that, the load distribution beneath the raft will be close to a uniform distribution like in a rigid raft and the differential settlements will be minimized resultingly.

The effectiveness of the method can be clearly seen from the Fig 2.15 below. The central piles 3*3 with 1m. and 1.5m. diameter shows a much better performance in reducing the differential settlements compared to a uniformly distributed 9*9 piles and a raft alone.

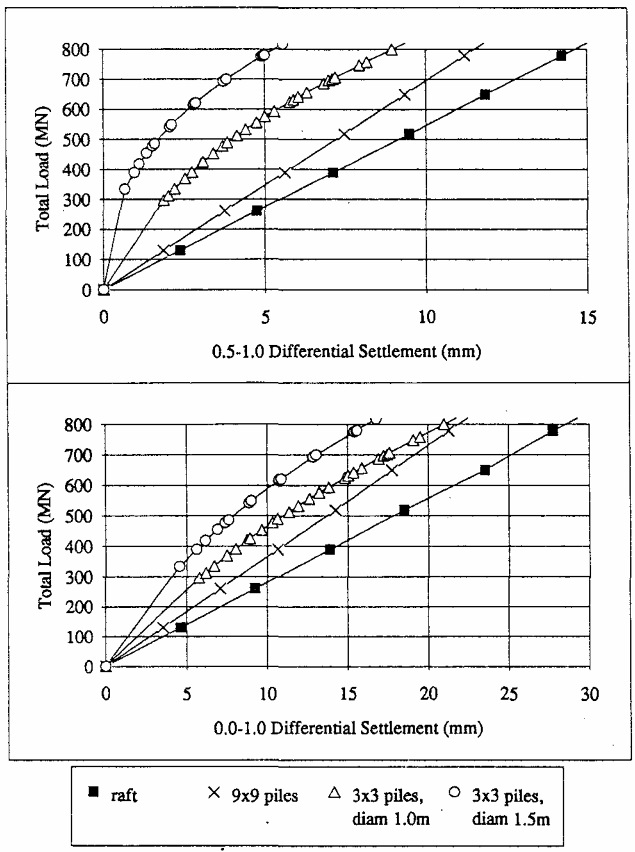


Fig 2.15 Development of Differential Settlements with Load Level (Randolph, 1994)

2.3.3 Viggiani - Mandolini Approach

In this approach, rather than classifying the design philosophies, the piled rafts are classified in two broad categories by Russo & Viggiani (1998), and investigated further in this way. The methodology, recommended by Sanctis & Mandolini (2006), operates in a fully deterministic logic and "factor of safeties" are employed to obtain an acceptable solution. The main importance of the approach is that, this approach has established the fundamentals of the new "Italian Code" which will be described in the next part of the thesis. Moreoever, it has a special importance since the analysis of the researches were performed on the typical soft clay deposits of the Italy which is , in fact, classified as an unsuitable soil type by many researchers for designing piled rafts.

2.3.3.1 Viggiani's Classification (Russo & Viggiani, 1998)

Viggiani ,in their research with Russo in 1998, grouped the piled rafts into two broad categories. These are:

a) Small Piled Rafts
b) Large Piled Rafts

a) Small Piled Rafts:
In this group of piled rafts, generally the raft alone can not provide the adequate bearing capacity and the main aim in installing the piles is to provide additional bearing capacity and to achieve a suitable safety factor against bearing failure. In " Small Piled Rafts", the raft width B_R is generally small when compared to the length of the piles L. (B_R / L < 1). Since the aspect ratio of the raft is high, it's flexural stiffness is generally high and thus, there is not a significant problem of differential settlements in the raft.

b) Large Piled Rafts:

In the case of "Large Piled Rafts", the provided bearing capacity by the raft alone is generally sufficient and the piles are installed as settlement reducers. Generally, the raft width B_R is high compared to the pile length,L (B_R / L > 1). Differential settlements may cause a significant problem in case of "Large Piled Rafts", due to the high aspect ratio and the piles may be needed to be located accordingly to minimize the differential settlements.

2.3.3.2 Main Research of Sanctis & Mandolini (2006)

This research is included in the thesis since the main findings of this research led to the corresponding recommended methodology and the "New Italian Code". In this research, piled rafts with various pile configurations were investigated for their bearing capacity on soft clay soils via a finite element code and some recommendations for the design of the piled rafts were developed resultingly.

- Terminology :

The main terminology used in the research is summarized below :

$$Q_{UR,ult} = (CF_cN_cc_u)A \tag{2.25}$$

$$Q_{G,ult} = \eta n Q_{s,ult} \tag{2.26}$$

$$Q_{PR,ult} = \alpha_{UR}Q_{UR,ult} + \alpha_G Q_{G,ult} \tag{2.27}$$

$$\beta_{PR} = \frac{Q_{PR,ult}}{Q_{G,ult}} \tag{2.28}$$

$$\xi_{PR} = \frac{Q_{PR,ult}}{Q_{UR,ult} + Q_{G,ult}} \tag{2.29}$$

where,

$Q_{ur,ult}$: ultimate bearing capacity of unpiled raft

$Q_{G,ult}$: ultimate bearing capacity of the pile group

$Q_{s,ult}$: individual pile capacity

$Q_{PR,ult}$: ultimate bearing capacity of piled raft

C, F_c, N_c : bearing capacity factors

C_u : undrained shear strength of the soil

A : base area of the raft

η : efficiency factor

n : number of piles

α_{UR} , α_G : coefficients affecting the failure load of the raft and the pile group

β_{PR} , ξ_{PR} : coefficients defined by the authors

- The Research :

A parametric study with several pile configurations were performed and even some configurations led to a value of β_{PR} = 5.85 which indicated the enormous contribution of the raft to the overall bearing capacity. The results of the parametric study is presented below in Table 2.2 .

Table 2.2 Results of the Parametric Study
(Sanctis & Mandolini, 2006)

Case	L/d	n	s/d	B_R/d	$K_{UR}/p_{ref}d$ ($\times 10^{-3}$)	$K_G/p_{ref}d$ ($\times 10^{-3}$)	$K_{PR}/p_{ref}d$ ($\times 10^{-3}$)	$Q_{UR,ult}/p_{ref}d^2$	$Q_{G,ult}/p_{ref}d^2$	$Q_{PR,ult}/p_{ref}d^2$	β_{PR}	α_{UR}
1	40	49	4	28	9.2	43.8	44.3	1,235	4,090	4,909	1.20	0.41
2	40	9	4	28	9.2	15.7	19.9	1,235	751	1,828	2.43	0.95
3	40	9	8	28	9.2	22.2	24.8	1,235	751	1,891	2.52	1.00
4	20	49	4	28	9.9	27.7	28.3	1,235	1,364	2,120	1.55	0.41
5	20	9	4	28	9.9	7.8	13.5	1,235	251	1,446	5.77	1.00
6	20	9	8	28	9.9	12.1	15.7	1,235	251	1,465	5.85	1.00
7	40	25	4	20	5.9	28.4	28.9	630	2,087	2,644	1.27	0.42
8	40	9	4	20	5.9	15.7	17.7	630	751	1,284	1.71	0.78
9	40	9	8	20	5.9	22.2	23.1	630	751	1,348	1.79	0.97
10	20	25	4	20	5.9	16.1	16.8	630	696	1,088	1.56	0.48
11	20	9	4	20	5.9	7.8	10.4	630	251	761	3.04	0.87
12	20	9	8	20	5.9	12.1	13.3	630	251	809	3.23	0.98
13	20	9	4	12	2.7	7.8	8.1	227	251	408	1.63	0.66
14	40	9	4	12	2.7	15.7	16.0	227	751	976	1.30	0.61

An approximation for the value of α_{UR}, which defines the mobilized capacity ratio of the raft in a piled raft system, was obtained as a result of the performed analysis. The resulting figure is presented below as Fig 2.16.

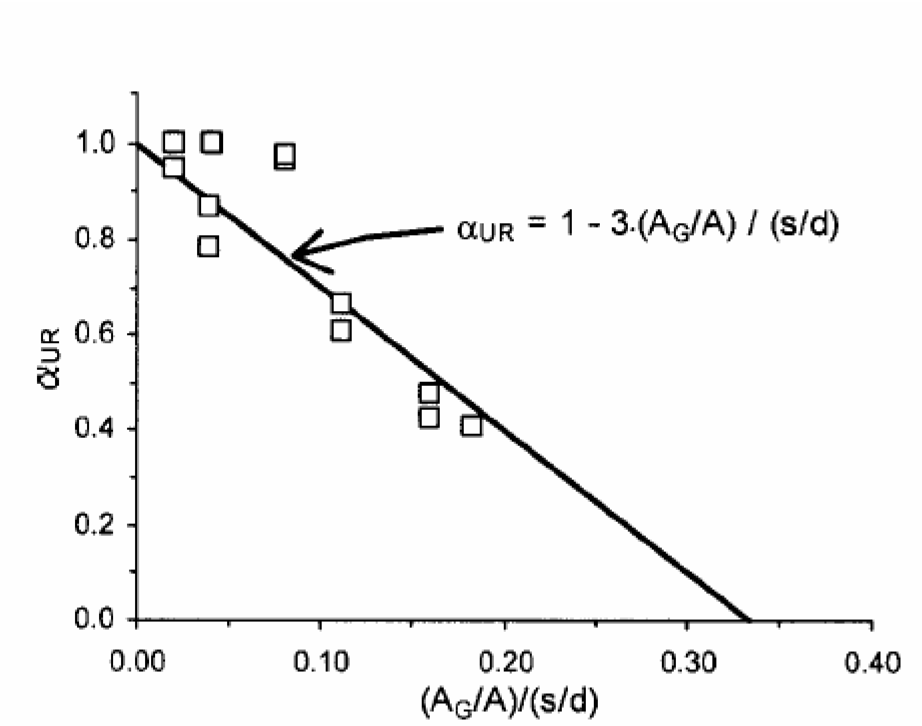


Fig 2.16 Relationship between α_{UR} and $(A_G/A)/(s/d)$

(Sanctis & Mandolini, 2006)

As it can be seen clearly from the figure above, $\alpha_{UR} = 0.5$ value is obtained for a ratio of $(A_G/A)/(s/d) = 1/6$ which may be taken as an initial design value for piled rafts where, $A_G = [(\sqrt{n}-1)s]^2$, s : spacing of piles and d : diameter of the piles .

2.3.3.3 Recommended Design Procedure

In a research by Cooke (1986) on stiff clays, it was concluded that, the total settlements of an unpiled raft (UR) designed with a FS = 3 was approximately equal to that of a piled raft designed with the same factor of safety. Moreoever, the typical piled rafts lied in the range of FS = 6 – 14 which resulted in significantly reduced settlements. The plots of the research are presented as Fig 2.17 :

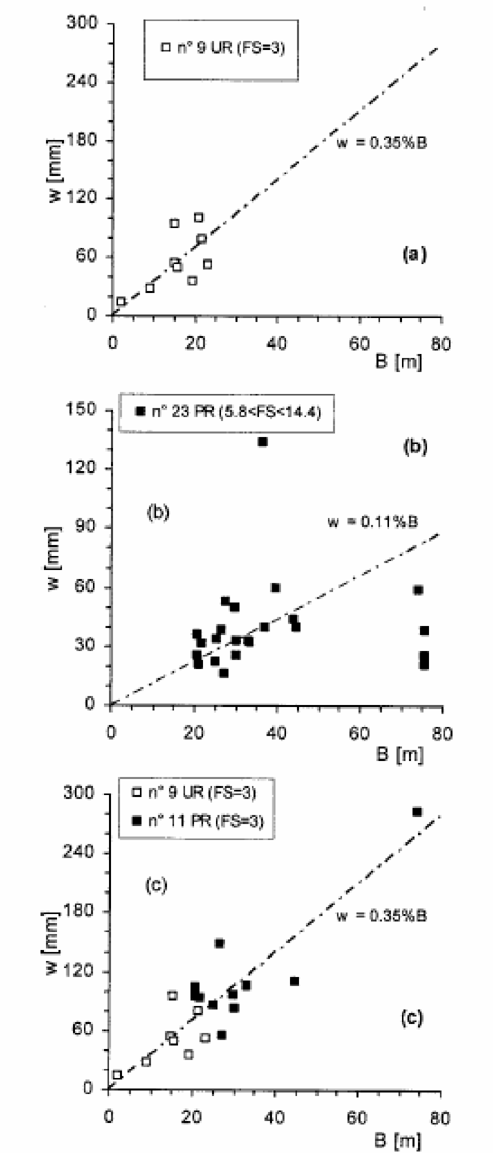


Fig 2.17 Settlement of foundation over stiff clays (a) unpiled raft, (b) piled rafts, (c) unpiled and piled rafts
(Sanctis & Mandolini, 2006)

As it can be seen from the Table 2.3 below, the results of the parametric study led to the conclusion that the ξ_{PR} value ranged between 0.82 – 1.00 which means, at least 82 % of the total capacities of the unpiled raft and pile group is mobilized in case of a piled raft for a settlement of $w = 0.35\%B_R$.

(The studied settlement level corresponds to 100 mm, 70 mm and 40 mm for configurations of B_R / d = 28, 20 and 12 respectively, since the pile diameter equals to d = 1 m.)

Table 2.3 Safety Factors corresponding to a settlement of w = 0.35%B_R
(Sanctis & Mandolini, 2006)

Case	L/d	n	s/d	B_R/d	FS_{UR}	FS_G	FS_{PR}	ξ_{PR}
1	40	49	4	28	1.95	6.46	7.76	0.92
2	40	9	4	28	1.95	1.19	2.89	0.92
3	40	9	8	28	1.95	1.19	2.99	0.95
4	20	49	4	28	1.95	2.15	3.35	0.82
5	20	9	4	28	1.95	0.40	2.28	0.97
6	20	9	8	28	1.95	0.40	2.32	0.99
7	40	25	4	20	2.11	6.98	8.84	0.97
8	40	9	4	20	2.11	2.51	4.29	0.93
9	40	9	8	20	2.11	2.51	4.51	0.98
10	20	25	4	20	2.11	2.33	3.64	0.82
11	20	9	4	20	2.11	0.84	2.54	0.86
12	20	9	8	20	2.11	0.84	2.70	0.92
13	20	9	4	12	2.26	2.49	4.06	0.86
14	40	9	4	12	2.26	7.47	9.71	1.00

After some simple calculations the defined ratio for ξ_{PR} can be extended as :

$$\xi_{PR} = \frac{Q_{PR,ult}}{Q_{UR,ult} + Q_{G,ult}} = \frac{FS_{PR}}{FS_{UR} + FS_G} \tag{2.30}$$

Since the values of ξ_{PR} lies between 0.82 – 1.00, the design ξ_{PR} value can be accepted as 0.8 as a safe value. Then the main design principle can be obtained as:

$$FS_{PR} = 0.8*(FS_{UR} + FS_G) \tag{2.31}$$

Applying individual factor of safeties for both unpiled raft and pile group and determining the FS_{PR} from equation (2.31) will lead to a total settlement of $w = 0.35\%B_R$ at maximum.

2.4 Codes for Piled Rafts

2.4.1 German Code

A code that regulated the design criterion of piled rafts was developed by a group of experts who were sponsorized by the "German Institute for Building Research". The regulations of the code were based on both the experimental and analytical studies and extensive monitoring of the case histories.

2.4.1.1 Basic Requirements

For an acceptable calculation according to the German code, the model used in the analysis should be able to take into account all the interaction that occur between pile soil and raft. Moreover, the model should be calibrated by the back analysis of a single pile and an investigated existing foundation with similar conditions. Additionally, the model should be able to predict the following issues correctly:

- The bearing behavior of individual piles in a pile group depending on their location
- The ratio of the carried load between piles and raft as a function of the settlement of the piled raft
- The load – settlement behavior of the piled raft even up to ultimate loads

- The bending moments and internal forces in the raft for the appropriate structural design

2.4.1.2 The Recommended Design Philosophy & Procedure

The code is established on a logic that operates by limit state design philosophy. As described in Katzenbach et al. (2005), in the limit state design method, at first the set of limits beyond which the structure fails to satisfy fundamental requirements, should be determined. Then, the performance of the structure should be evaluated according to these limits. A scheme summarizing the limit state concept is presented in Fig 2.18 below.

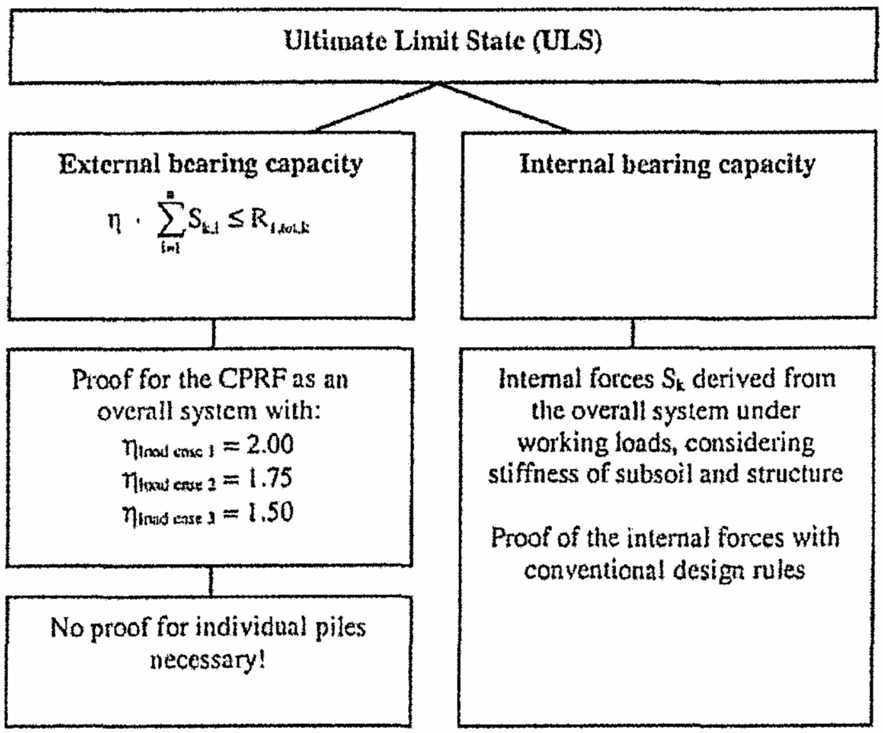


Fig 2.18 Ultimate Limit State Approach
(Katzenbach et al., 2005)

As seen in the Fig 2.18, in the limit state concept, the bearing capacities are studied in two different categories named as “external bearing capacity” and “internal bearing capacity” . The external bearing capacity is the bearing capacity of the subsoil and the internal bearing capacity is the structural bearing capacity of the piles and the raft. The main difference of the code for piled rafts from the conventional design codes is that, there is no need to prove the adequacy of an individual pile for the external bearing capacity. Rather, it is necessary to show that the piled raft system has an adequate safety in terms of the external bearing capacity. Moreover, for regularly spaced pile groups with a centrically loaded raft underlien by homogenous subsoil, the external bearing capacity of the raft may be calculated as the ultimate capacity of an unpiled equivalent raft. After showing that the system is adequate in terms of the external bearing capacity, the internal forces of the system have to be calculated under working loads. Then the structural elements of the system should be designed to provide adequate safety against these internal forces.

There are some more regulations in the code about piled rafts. To illustrate, the design of a piled raft should be supervised by an expert in the field of soil mechanics and foundation design and the observational method described in Eurocode 7 should be employed. Moreover, the integrity of the piles and raft should be guaranteed by a quality assurance concept.

2.4.2 Italian Code

“Looking at the codes and regulations all over the world, one realizes that the capacity based approach, neglecting the contribution of the raft, is still dominant. As a consequence of this overconservative assumption, the total and differential settlements of piled foundations are usually very small, and probably in most cases unnecessarily small. ” says the famous Italian researchers Sanctis, Mandolini, Russo & Viggiani in 2002.

Based on the thought described above, a new Italian code that regulates the design criterion of the piled rafts has been developed in the recent years. The code is still under evaluation. This code uses mainly the logic of the Italian approach that was described. Factor of safeties are employed for the initialization of an appropriate piled raft design. The requirements of the Italian Code are summarized below:

The Code classifies the piled rafts into two as described in the former “Italian Approach” part:

a) Small Piled Rafts
b) Large Piled Rafts

a) For Small Piled Rafts :

An analysis of the pile – raft – soil interaction is preliminarily carried out. After determining the amount of the load carried by the piles and the raft, each of them is individually checked against bearing capacity by showing that each element individually satisfies the corresponding factor of safety that is required for that structural element. The recommended factors of safety are 3 and 2.5 for raft and piles respectively. The FS for piles may be reduced to 2 in case the bearing capacity is determined by load tests to failure.

b) For Large Piled Rafts :

If the piles are needed namely to reduce total and/or differential settlements like in the case of large piled rafts, the unpiled raft should be able to provide enough bearing capacity with a factor of safety of 3. In this concept, the piles are allowed to be loaded near to their ultimate capacity.

CHAPTER 3

THE MATERIALS AND THE METHOD USED IN THE FINITE ELEMENT ANALYSIS

In this chapter, the properties of the materials used in the analysis and the solution method employed, will be described in detail. Also the reference studies of the thesis will be summarized at the beginning of the chapter.

3.1 The Reference Studies

There are two studies which are taken as reference for this thesis:

a) "Piled Rafts in Overconsolidated Clay : Comparison of in-situ Measurements and Numerical Analysis" by O. Reul & M. F. Randolph (Géotechnique, 2003)
b) "Design Strategies for Piled Rafts Subjected to Nonuniform Vertical Loading" by O. Reul & M. F. Randolph (Journal of Geotechnical and Geoenvironmental Engineering, 2004)

As it can be seen above, both of the studies belong to the same researchers. The first study is a comparison of the in-situ measurements and results of the finite element analysis for piled rafts constructed on Frankfurt Clay. The second study is a parametric study based on a finite element model developed in "ABAQUS" finite element software which was calibrated by the help of the results of the first study.

3.1.1 A Comparison of the In-Situ Measurements and Results of the Finite Element Analysis for Piled Rafts Constructed on Frankfurt Clay by O. Reul & M. F. Randolph (2003) [1]

This study is a result of the long observations of the authors Reul & Randolph, on the high – rise buildings constructed on Frankfurt Clay. (The observations made, are given in detail in Table 3.1) Detailed back – analysis of the three instrumented and observed piled raft foundations, using three dimensional finite element analysis are presented throughout the study. The investigated three structures are :

a) Westend 1
b) Messeturm
c) Torhaus

[1] ➔ Refers to the study in 3.1.(a)

Table 3.1 Piled Rafts in Frankfurt, Germany

(Reul & Randolph, 2003)

Building	References	H: m	P_{eff}: MN	A: m^2	t_r: m	z_r: m	n	L_p: m	D_p: m	n_{ip}	P_p: MN	s: mm	t: years
American Express	Rollberg & Gilbert (1993); Reul (2000)	75	723	3575	2·0	14·0	35	20·0	0·9	6	2·7–5·1	55	1·0†
Congress Centre	Barth & Reul (1997); Reul (2000)	52	1440	10 200	2·7	14·2	141	12·5–34·5	1·3	12	2·4–5·9	58	0
Eurotheum	Katzenbach *et al.* (1998); Moormann (2000)	110	425	1893	2·5	13·0	25	25·0–30·0	1·5	4	2·6–4·7	29	1·0†
Forum-Kastor	Lutz *et al.* (1996); Ripper & El Mossallamy (1999)	95	750	2830	3·0	13·5	26	20·0–30·0	1·3	3	5·0–12·6	55	0†
Forum-Pollux	Lutz *et al.* (1996); Ripper & El Mossallamy (1999)	130	760	1920	3·0	13·5	22	30·0	1·3	3	7·4–11·7	70	0†
Japan Centre	Lutz *et al.* (1996); Ripper & El Mossallamy (1999)	115	630	1920	3·5	15·8	25	22·0	1·3	6	7·9–13·8	65	0·5†
Main Tower	Katzenbach *et al.* (1998); Moormann (2000)	199	1470	3800	3·8	21	112	30·0	1·5	17	1·4–8·0	25	0
Messeturm	Sommer *et al.* (1990, 1991); Sommer & Hoffmann (1991a, b); Sommer (1993); Reul (2000)	256	1570	3457	6·0	14·0	64	26·9–34·9	1·3	12	5·8–20·1	144	8
Torhaus	Sommer (1986, 1991); Sommer *et al.* (1984, 1985)	130	2 × 200	2 × 429	2·5	3·0	2 × 42	20·0	0·9	6	1·7–6·9	140	2
Westend 1	Franke & Lutz (1994); Lutz *et al.* (1996); Wittmann & Ripper (1990)	208	950	2940	4·7	14·5	40	30·0	1·3	6	9·2–14·9	120	2·5†
Haus der Wirtschaft, Offenbach*	Reul (2000)	68	605	5120	2·0	8·5	47 6	25·0 37·5–41·0	1·2	6	1·4–3·1	25	0

H, height of the building; P_{eff}, effective load (settlement-inducing total load minus uplift); A, area of raft; t_r, maximum thickness of raft; z_r, maximum depth of raft below ground level; n, number of piles L_p, pile length; D_p, pile diameter; n_{ip}, number of instrumented piles; P_p, measured pile load resistance; s, maximum measured settlement; t, time of settlement measurement after completion o construction of building.

* Foundation in Rupel clay.

† Indicates completion of shell only.

3.1.1.1 Westend 1

3.1.1.1.1 The General Properties

The Westend 1 is a 90 m*100 m office building which was constructed between 1990 and 1993. There is a high – rise section with 208 m. height and a low – rise section with a height of 60 m. The high – rise part is constructed on a piled raft foundation which has a varying thickness of 4.7 m. in the core and 3.85 m. at the corners. The piled raft has dimensions of 47 m. * 62 m. and consists of 40 bored piles with a length of 30 m. and a diameter of 1.3 m. The general view of the structure is given in Fig 3.1

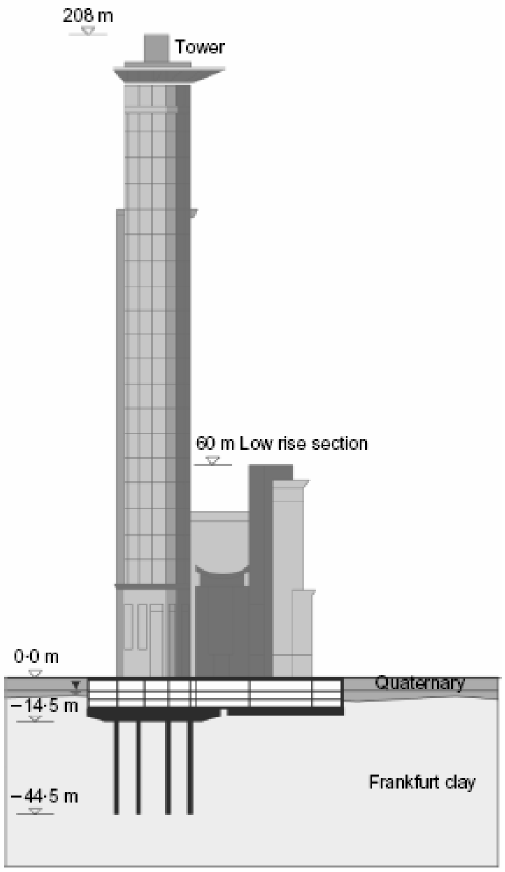


Fig 3.1 General View of the Westend 1
(Reul & Randolph, 2003)

3.1.1.1.2 The Instrumentation

In the instrumentation of the building, 6 instrumented piles, 13 contact pressure cells, 5 pore pressure cells, 1 multi – point borehole extensometer and 2 combined inclinometers / multi – point borehole extensometers were employed. The layout of the instrumentation is illustrated in Fig 3.2.

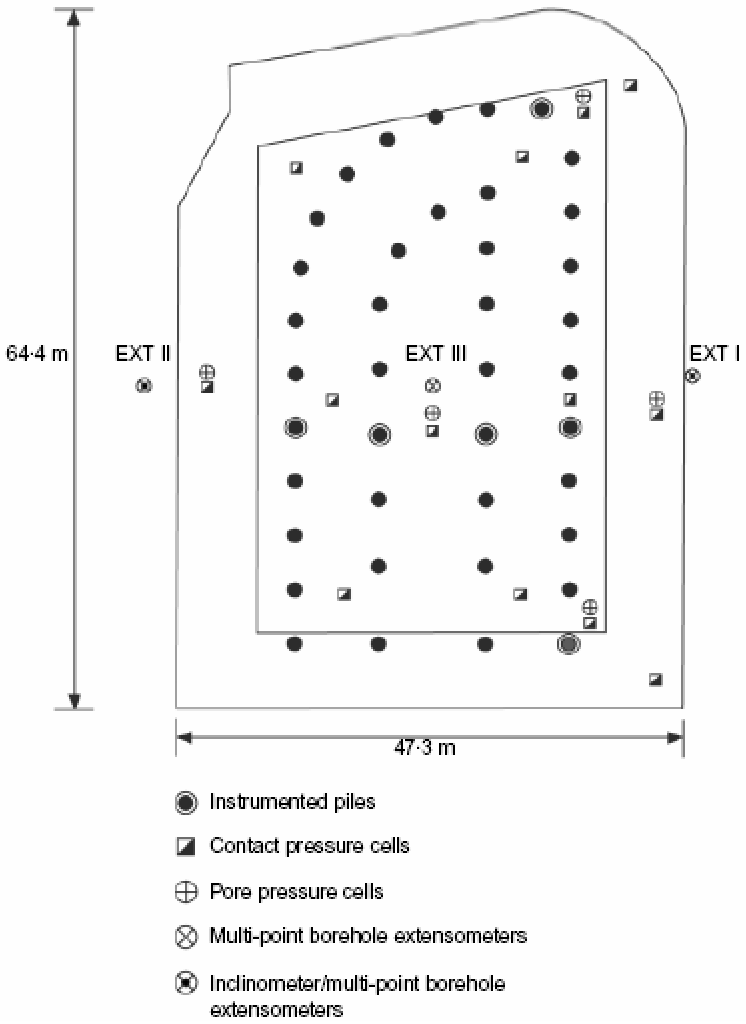


Fig 3.2 Instrumentation Layout of Westend 1
(Reul & Randolph, 2003)

3.1.1.1.3 The Subsoil Conditions

The subsoil consists of 8.5 m. thick quaternary layer at the top which is underlien by a layer of 63m. thick Frankfurt clay. The groundwater is at 7m. depth from ground level. Below Frankfurt clay, the Frankfurt limestone lies, which is classified as incompressible and modeled so as not to affect the results of the analysis. The building is founded directly on Frankfurt clay since the foundation depth equals to 14.5 m.

3.1.1.1.4 The Finite Element Analysis

In the analysis, only the soil below the foundation level was modeled and the overlying soil was given as a surcharge on the model. The long – term behavior of the Frankfurt clay was modeled by using drained shear parameters c^1 and ϕ^1. The non – linear material behavior of the soil has been modeled with a cap model which consists of three yield surface segments: the pressure dependent, perfectly plastic shear failure surface, F_s; the compression cap yield surface, F_c; and the transition yield surface, F_t. Changes of stress inside the yield surfaces cause elastic deformations, whereas changes of stress on the yield surfaces cause plastic deformations. Plastic flow is defined by the non – associated flow potential, G_s, of the shear surface and the associated flow potential G_c, of the cap. The parameters β and d can be derived from the c^1 and ϕ^1 of the soil. The yield surface used for the analysis and the corresponding p – t graph are provided below in Fig 3.3.

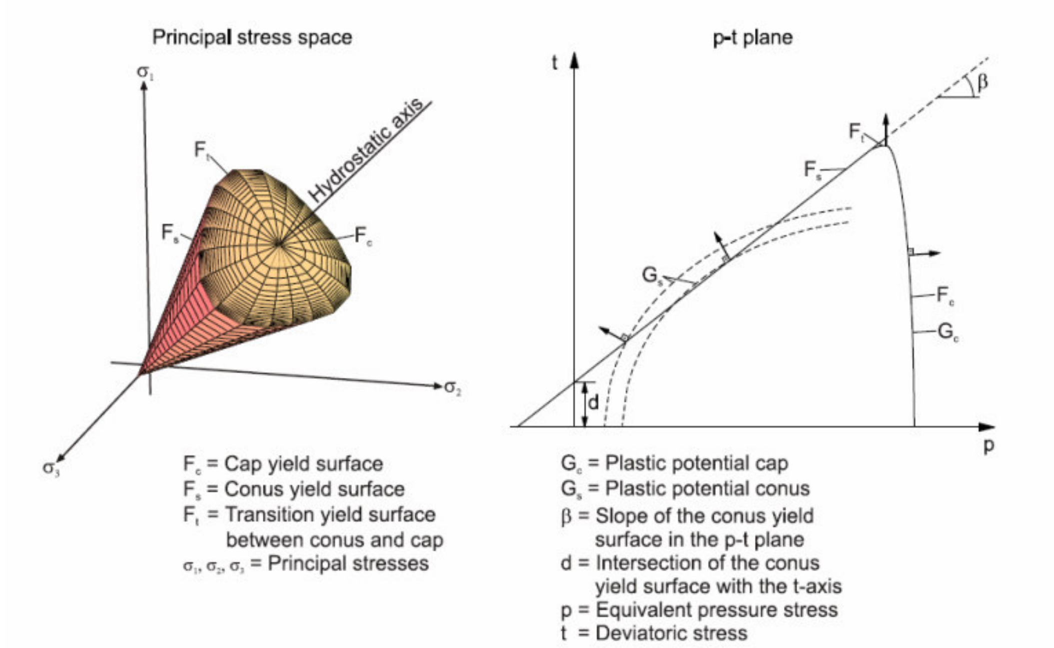


Fig 3.3 Cap Model : Yield Surfaces in Principal Stress Space and p – t plane
(Reul & Randolph, 2004)

The piled raft is completely modeled since there is no symmetry axis available. The finite element mesh of the system and the piled raft is shown in Fig 3.4.

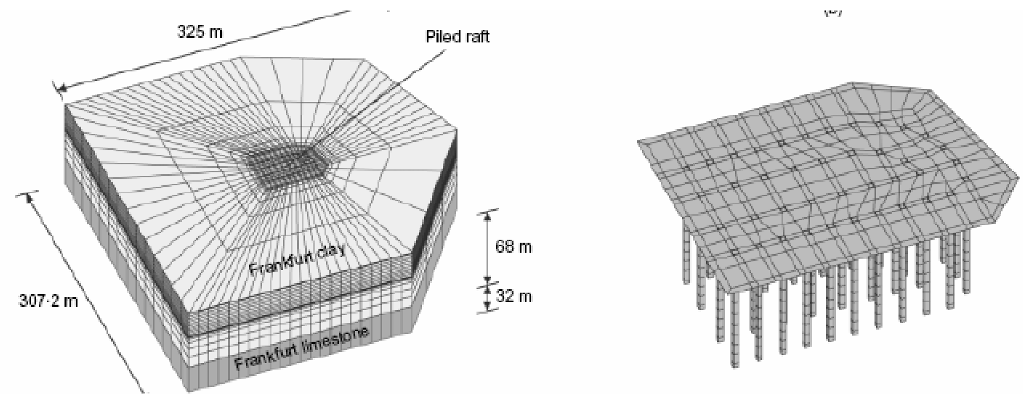


Fig 3.4 The Finite Element Mesh of the System and the Piled Raft for Westend 1
(Reul & Randolph, 2003)

3.1.1.2 Messeturm

3.1.1.2.1 The General Properties

The 256 m. high Messeturm building consists of 64 bored piles and a square raft with an edge length of 58.8 m. The 1.3 m. diameter piles are constructed to form three rings from outer core through the inner core. The piles vary in length from 26.9 m. at outer ring through 30.9 m. at the middle ring and to 34.9 m. at the inner ring. A general view of the building is illustrated in Fig 3.5 (a)

3.1.1.2.2 The Instrumentation

The piled raft of the structure was monitored for 7 years after the building was finished in 1991. In the monitoring study; 12 instrumented piles, 13 contact pressure cells, 1 pore pressure cell and 3 multi – point borehole extensometers were employed. The plan of the instrumentation can be seen in Fig 3.5 (b)

3.1.1.2.3 The Subsoil Conditions

The subsoil below Messeturm building consists of fill and quaternary sand and gravel up to a depth of 10 m. below ground level which is followed by the Frankfurt clay at least up to a depth of 70 m. The groundwater level is at 4.5 – 5 m. depth from ground level.

3.1.1.2.4 The Finite Element Analysis

The logic in modeling and analysis is just the same with the one in Westend 1. The mesh of the system and the piled raft are given in Fig 3.5 (c) & (d). One eight of the total system is modeled for the analysis of this structure.

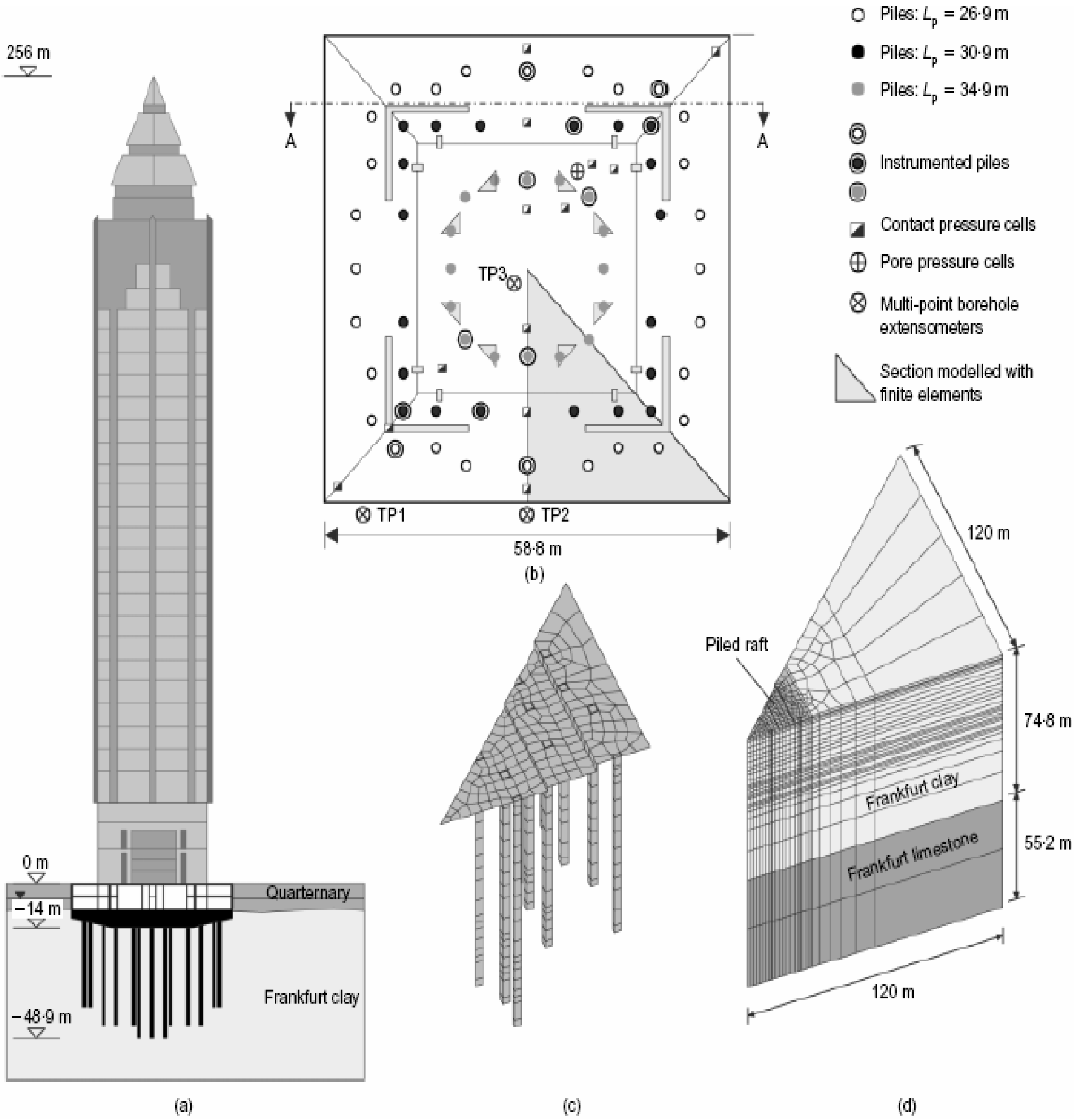


Fig 3.5 (a) General View, (b) Instrumentation Layout, (c) Finite Element Mesh of the Piled Raft, (d) Finite Element Mesh of the System

(Reul & Randolph, 2003)

3.1.1.3 Torhaus

3.1.1.3.1 The General Properties

The "TORHAUS DER MESSE" building was the first building in Germany designed with a piled raft when constructed between 1983 and 1986. The building is 130 m. high and has a very interesting foundation design since the structure lies on two separated symmetric piled rafts. These 17.5 m.* 24.5 m. rafts contain 84 bored piles, 20 m. in length and 0.9 m. in diameter oriented in rectangular manner. The distance between two rafts is 10 m. The general view of the building can be seen in Fig 3.6 (a).

3.1.1.3.2 The Instrumentation

The piled rafts of the structure were instrumented by 6 instrumented piles, 11 contact pressure cells and 3 multi – point borehole extensometers. The layout of the instrumentation is illustrated in Fig 3.6 (b)

3.1.1.3.3 The Subsoil Conditions

The soil consists of 2.5 m. thick quaternary sand and gravel below ground level which is followed by Frankfurt clay. The 2.5 m. thick raft lies just 3 m. below the ground level and the groundwater is just below the raft.

3.1.1.3.4 The Finite Element Analysis

The logic in modeling and analysis is just the same with the ones above in Westend 1 and Messeturm buildings. However one quarter of the raft was modeled this time considering the two symmetry planes in the raft. The mesh of the system and the piled raft are given in Fig 3.6 (c) & (d)

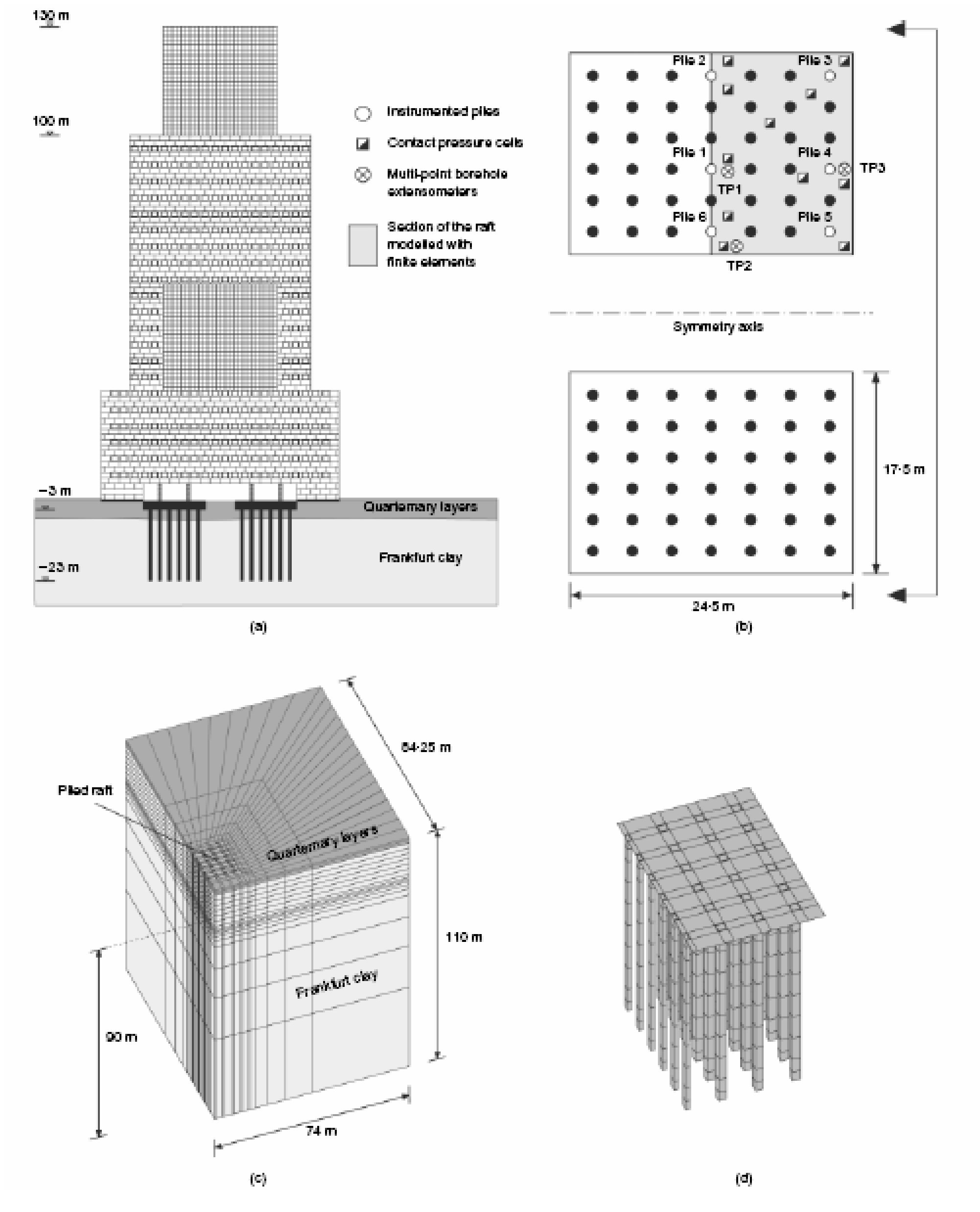


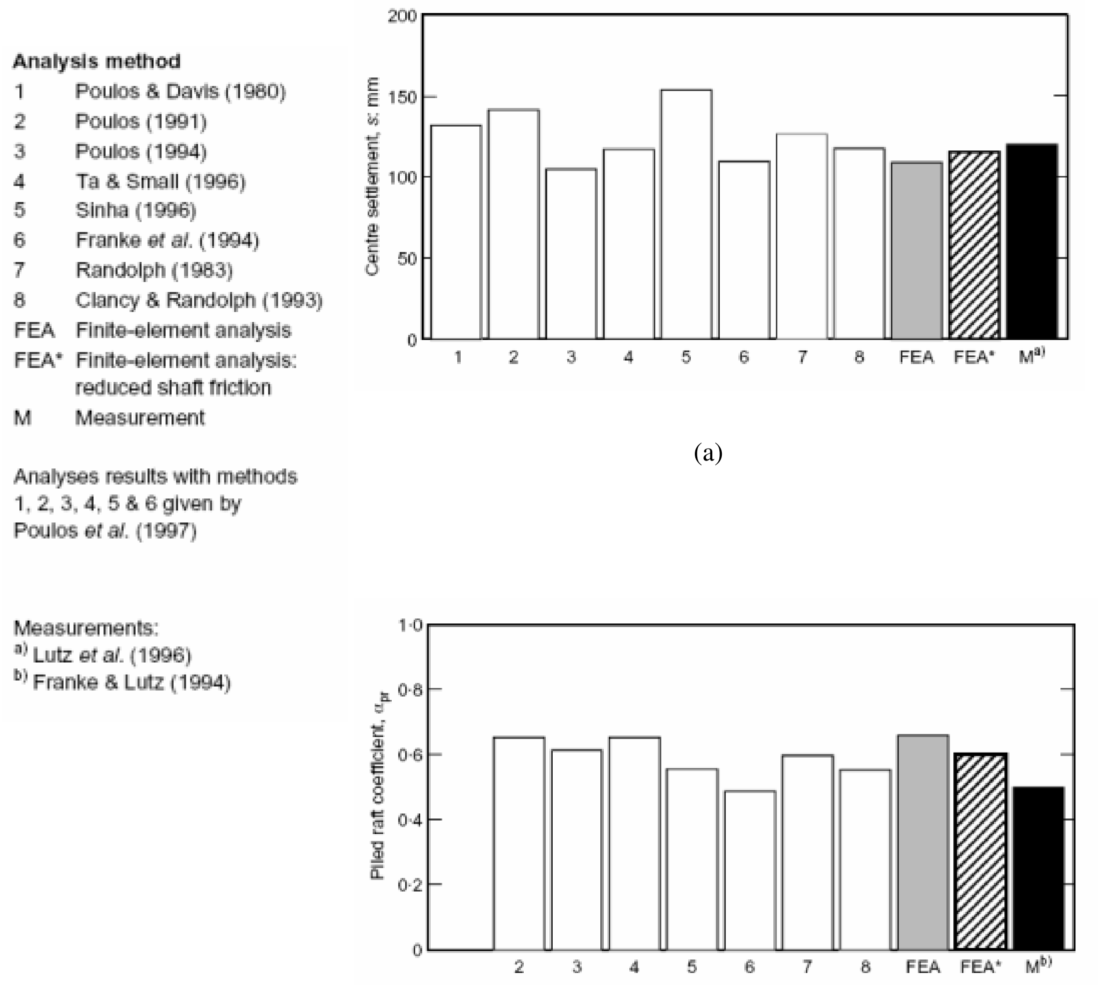
Fig 3.6 (a) Profile View of the Building, (b) Ground Plan of Raft, (c) Finite Element Mesh of System, (d) Finite element mesh of Piled Raft

(Reul & Randolph, 2003)

3.1.1.4 The Conclusions and Recommendations of the Authors

As described before; in the study discussed above, the detailed back analysis of three monitored piled rafts were performed. At the end of the analysis, it was observed that the results of the three – dimensional finite element analysis were in very good accordance with the in – situ measurements. However, there is a general tendency of the finite element analysis to overestimate the piled raft coefficient and correspondingly to underestimate the settlements compared to the measured values. A comparison of the finite element method with other methods and with the measured values for Westend 1 building is provided below in Fig. 3.7 (a) & (b) for central settlements and piled raft coefficient respectively. Here it should be emphasized that, the finite element analysis with reduced shaft friction (c = 0 kPa, represented by FEA* in Fig. 3.7) shows a better performance in matching with the real values which indicates that even some settlement is allowed in the foundation the shaft friction does not fully mobilize along the pile shafts.

The authors also emphasize that; although an elasto – plastic cap model was used for modeling the behavior of the Frankfurt clay in this study, an elastic model would yield to very similar results since the Frankfurt clay is an overconsolidated clay and as a result, the behavior is dominated by the soil stiffness rather than soil strength. This idea is the base of this thesis since elastic material properties will be used in the two – dimensional and three – dimensional finite element analysis of the thesis



(a)

(b)

Fig 3.7 Comparison of Measurements and Analysis; (a) Central Settlement, (b) Piled Raft Coefficient
(Reul & Randolph, 2003)

3.1.2 A Parametric Study by O. Reul and M. F. Randolph (2004) for Piled Rafts Under Non-Uniform Vertical Loading [2]

3.1.2.1 General View of the Study

In this research, a parametric study is carried out for 259 different piled raft configurations. For the analysis, an imaginary model is created which represents the characteristic properties of both the subsoil conditions and piled rafts in the Frankfurt region and calibrated according to the results of the previous study. In the model, the foundation level is set 14 m. below the ground level with a groundwater level of 7 m. from ground surface. Like mostly observed in the region, the quaternary layer was taken as 7 m. thick and the Frankfurt clay as 69 m. thick. underlying the quaternary layers. Similar to the past research, the soil above the foundation level is modeled as an external surcharge and only the soil below the foundation level is modeled with finite elements. Moreover, the Frankfurt limestone layer is not included in the model this time since it is regarded as incompressible and has no effect on the results of the analysis. Only one eight of the total system is modeled in all analysis owing to the threefold symmetry in the system. The model extends to 190 m. in both directions and constrained against lateral movement along the boundaries. The model and the mesh used in the finite element analysis are visualized in Fig 3.8 below.

[2] ➔ Refers to the Study in 3.1. (b)

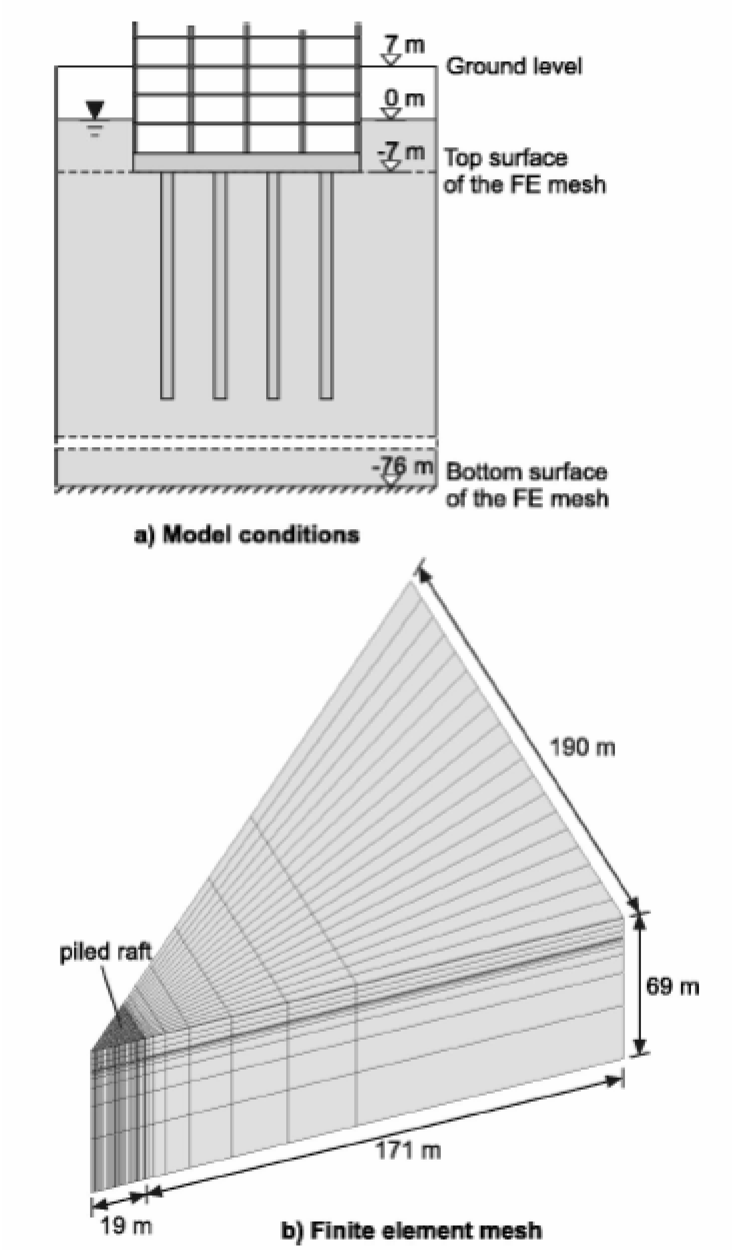


Fig 3.8 Model Conditions in Parametric Study and Finite Element Mesh
(Reul & Randolph, 2004)

The piles are represented by first – order solid finite elements of brick and wedge shape and the raft is modeled as first – order shell elements of square and triangular shape with reduced integration. Both the piles and the raft are modeled to behave as linear – elastic materials.

3.1.2.2 System Configurations

In this study, square piled rafts with an edge length of B = 38 m. have been considered. Mainly there are 3 pile configurations concerned which are named as "Pile Configuration 1,2&3". In Pile Configuration 1, the piles are uniformly distributed beneath the raft. In Pile Configuration 2, the piles are placed in the central area of the raft. Finally in Pile Configuration 3, piles are placed at the edges of the raft as well as the central area of the raft. Number of piles varied from n = 9 to n = 169 and the length of piles ranged between L_p = 10 – 50 m. The pile spacings are taken either as s = 3 m. or s = 6 m. for a constant pile diameter of 1 m. for all configurations. The pile configurations are summarized schmetically in Fig 3.9 (a)

3.1.2.3 Load Configurations

In the parametric study, there are four different load configurations employed for the analysis which are named as Loadtype I, II, III & IV. Loadtype I is the most realistic loading type beyond others since it represents the loading of a typical tall building. In this loadtype, half of the total load is applied in the core area of the raft which equals 25 % of the total raft area and the other half of the load is applied at the edges of the raft. In loadtypes 2 & 3, the whole load is applied only either on the core or on the edges of the raft respectively. Finally, in the Loadtype IV, the load is applied as uniform loading on the whole raft which is actually the basic assumption of the several methods available in the literature. The load configurations used in the analysis are illustrated in Fig 3.9 (b).

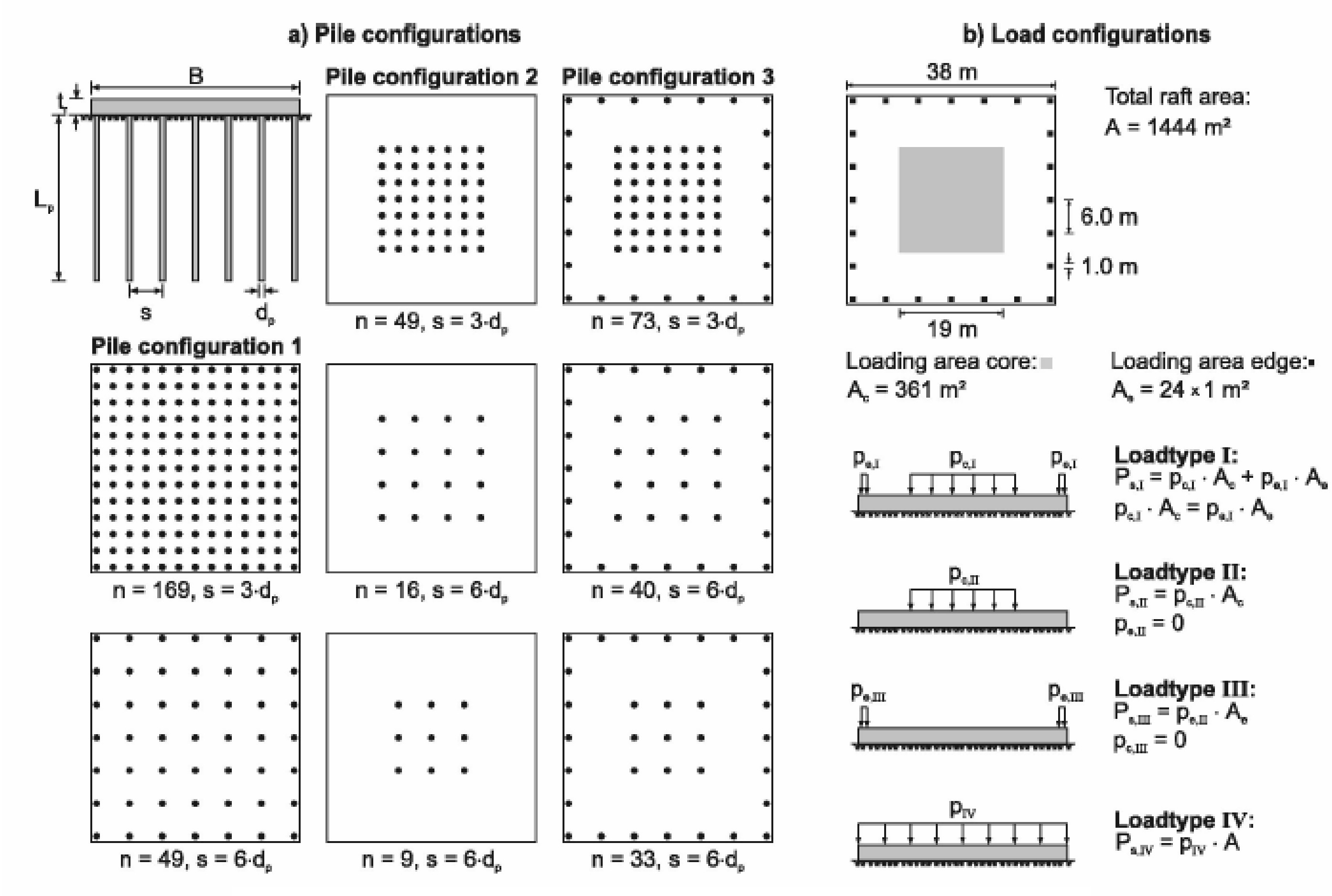


Fig 3.9 System Configurations and Load Configurations for Parametric Study

(Reul & Randolph, 2004)

The maximum load applied to the raft, including the weight of the raft and the uplift due to pore pressures equals to P_{eff} = 721.7 MN, which is approximately 20 % of the ultimate capacity of an equivalent unpiled raft under drained conditions. The applied load is within the range of the observed structures as shown in Table 3.2 below.

Table 3.2 Piled Rafts in Germany
(Reul & Randolph, 2004)

Building	H (m)	P_{eff} (MN)
Japan Center, Frankfurt	115	630
Messeturm, Frankfurt	256	1570
Torhaus, Frankfurt	130	2×200
Westend 1, Frankfurt	208	950
Haus der Wirtschaft, Offenbach	68	605
Treptowers, Berlin	122	632

The results of this study will not be discussed in this section, since the results of this study will be presented and compared with the results of this thesis in the following chapters in detail.

3.2 The Materials used in the Analysis

In this part of the chapter, the properties of the materials used in the analysis will be described in detail. Mainly, there are 3 different materials used in the three dimensional finite element analysis. These are:

a) The Frankfurt Clay
b) The Raft
c) The Piles

The material properties of these three elements above and all the assumptions made about these materials in order to facilitate the analysis are described below:

3.2.1 The Frankfurt Clay

3.2.1.1 The General Discussion

In comparison with the other developed cities in the world, Frankfurt experiences a main disadvantage of its highly plastic and very deep Frankfurt clay, which turns the construction of each high – rise building to a challenge. On the other hand, this situation provides an enormous data for geotechnical experts and a new research area for academicians.

The first researches about the Frankfurt clay started in the late 1960's after heavy tilting and settlements up to 300 mm was experienced in the newly constructed high – rise buildings and the findings up to now are as follows :

3.2.1.2 The Geological Properties

The Frankfurt clay is an overconsolidated clay which developed 2 to 10 million years ago as a result of the sedimentation in the Tertiary sea in the Mainz basin. In the central Frankfurt, the thickness of the clay layer reaches up to 100 meters and includes lignite coal lenses, limestone banks and layers of calcerous sand. The groundwater level is generally just above the clay layer and seeps through the fissured limestone banks and sand lenses resulting in different confined aquifer pressures. (El – Mossallamy, 2002)

The clay is geologically overconsolidated through older, already eroded sediments and volcanic rock which results in highly horizontally stressed subsoil. Moreover, this situation affects all the stress – strain and failure behavior of the Frankfurt clay.

3.2.1.3 The Geotechnical Properties

As a result of the continuous site observations and laboratory test programs performed through the past decades, much of the behavior of the Frankfurt clay is understood and the main geotechnical parameters of the clay are determined.

The Frankfurt clay has a unit weight of γ_{fc} = 19 kN/m^3 and shows a tendency to have a high K_0 (coefficient of lateral earth pressure) depending on the depth of the soil below surface of tertiary layers as a result of being subjected to a high erosion in the past. At the end of the long laboratory tests, the drained shear parameters c^1 and ϕ^1 of Frankfurt clay were determined as c^1 = 20 kPa and $\phi^1 = 20^0$.

The Young's modulus of the Frankfurt clay increases with depth. The distribution of the Young's Modulus with depth, is described by the Eq 3.1 below, which is an empirical formulation based on the back analysis of boundary value problems in Frankfurt clay.(Reul, 2000) . Also, the Frankfurt clay has a Poisson's ratio of 0.15 indicating a very stiff material.

$$E = 45 + \left(\tanh\left(\frac{z-30}{15} \right) + 1 \right) 0.7z \qquad (3.1)$$

3.2.1.4 Modeling of the Frankfurt Clay in the Analysis

As mentioned before, the authors of the reference studies Reul & Randolph emphasize that, since the material behavior of the Frankfurt clay is dominated by soil stiffness rather than soil strength, an elastic model would yield to very similar results with their solutions in which an elasto – plastic cap model was used to model the Frankfurt clay. With the help of this idea, Mohr – Coulomb material behavior under drained conditions is introduced in both the two – dimensional and three – dimensional finite element analysis of this thesis. The unit weight, Poisson's ratio, c^1, and ϕ^1 of the clay are inserted to the models just like it was given in the reference study.

Since the Young's modulus of the Frankfurt clay increases with depth, the layering of the soil was an important issue since it would affect all the results of the analysis. The layering of the soil is made in convenience with the parameters given in the reference studies by dividing the 69 m. thick clay layer into three pieces concerning the rate of change in E and the change in K_o of the subsoil. A more detailed description of the layering of the analyzed models will be described in the following chapter.

3.2.2 The Raft

3.2.2.1 The Raft Modeling in the Reference Study

The raft was considered to behave as linear elastic in the analysis. The Young's modulus of the raft was selected as $E_r = 34000$ MPa which represents an average value of the in – situ measurements and hand calculations. The unit weight of the raft was taken as, $\gamma_r = 25$ kN/m^3 and the Poisson's ratio was selected as $\upsilon_r = 0.2$, which are typical values for reinforced concrete.

3.2.2.2 The Raft Modeling in the Analysis of This Thesis

The raft is modeled as linear elastic same as in the reference study. Moreover, all the material parameters given in the reference study are used in the analyses. However, some small adaptations on some of the parameters are made to make the parameters convenient with the database of the "Plaxis" software for two-dimensional finite element analysis. These adoptions will be described in detail in the following chapter.

3.2.3 The Piles

3.2.3.1 The Pile Modeling in the Reference Study

Just as the raft, the piles were also modeled to behave in linear elastic manner. Moreover, all the parameters, except for the Young's modulus, used for the raft were also utilized for the modeling of the piles since they are made of the same materials, steel and concrete, with very close reinforcement ratios. The Young's modulus of the piles was selected as E_p = 30000 MPa which is a smaller value than that of the raft, E_r. "Because, concrete samples taken from bored piles as well as in – situ integrity testing show that the Young's modulus of the piles is generally smaller than the design value obtained from samples under more simple production conditions". (Franke & Lutz, 1994; Holzhäuser, 1998; Reul, 2000)

3.2.3.2 The Pile Modeling in the Analysis of This Thesis

The piles are modeled to behave as linear elastic materials just as in the reference study. Moreover, all the parameters used in the reference study were also integrated in the analysis of this thesis with small adoptions for convenience with the "Plaxis 8.2" software which works under plane-strain conditions. These adoptions will be described in detail in the following chapter.

- All the material parameters used in the finite element analysis are summarized below in Table 3.3

Table 3.3 Material Parameters used in the Finite Element Analysis for Parametric Study
(Reul & Randolph, 2004)

Parameter		Soil	Raft	Piles
E_s , E_r , E_p	MPa	Eq. (1)	34,000	30,000
ν_s , ν_r , ν_p	—	0.15	0.2	0.2
γ'	kN/m^3	9	15	15
K_0	—	0.72 $0 \leq z < 25^a$ 0.57 $z \geq 25^a$	—	—
ϕ'	°	20	—	—
c'	kPa	20	—	—

Z^a = meters below surface of tertiary layers

3.3 The Method used in the Analysis

3.3.1 The Method used in the Reference Studies

Since the first reference study described before (Reul & Randolph, 2003) was a comparison of the in – situ measurements with the three – dimensional finite element models; a new, site – specific solution method had to be developed to illustrate the real situation as close as possible in the computer models. So a solution method was developed by the authors which follows a step – by –

step construction technique. A sample analysis performed for the Westend 1 building is shown below in Table 3.4.

Table 3.4 Step – by – Step Analysis of the Westend 1 Building
(Reul & Randolph, 2003)

Step	Applied load, P_{eff}: MN	Mean vertical effective stress at foundation level, σ'_v: kPa
1. *In situ* stress state	–	192·0
2. Excavation to a depth of 7 m below ground level	–	66·0
3. Installation of the piles	–	66·0
4. Excavation to a depth of 14·5 m below ground level	–	0
5. Application of weight of raft minus uplift due to pore pressures as uniform load on subsoil (zero stiffness of raft)	61·9	21·9
6. Installation of raft	61·9	21·9
7. Loading of raft	956·9	338·0

In this method, only the soil below the foundation level was modeled in finite elements where the soil above foundation level was given as a surcharge on the model. Also, the groundwater above the foundation level was taken into account in the model by considering its uplift effect. The groundwater level was modeled as if it starts from the foundation level. In the first reference study, the Frankfurt limestone layer below the Frankfurt clay was modeled but after it was seen that it has no effect on the system, the Frankfurt limestone was not considered in the models of the second reference study (Reul & Randolph, 2004). Also, all of the models used in the reference studies are extended to 10 times of the raft width in order to eliminate the boundary effects. Moreover, either all or one quarter or one eight of the raft is modeled depending on the location and number of the symmetry axis on the raft.

In the solution method developed by authors; just after the model is established, the surcharge due to the overlying soil is applied to the whole system as the in – situ stress state as the first step. In the second step, only the applied surcharge on the raft is decreased as it will be equal to the decrease in the mean effective vertical stress at foundation level if the soil was excavated to the half depth of the foundation level. Then the piles are installed without changing any stresses on the system. After the installation of the piles, the applied surcharge is decreased to zero which means all the soil above the foundation level is excavated. In this solution method, the raft is modeled as weightless and the weight of the raft minus 50% of the uplift due to pore pressures is applied as uniform load on the subsoil as the fourth step of the analysis. Then, the raft is installed into the model without changing any stresses. After the raft is installed, a two staged loading is applied which illustrates the loading during the construction process. As the seventh stage, half of the total load due to the superstructure and 100 % of the uplift pore pressure is applied to the model. Then further loading of the raft is made as the last stage of the step – by – step analysis.

As an important point it should be emphasized that, in the analysis, the settlements until the seventh stage is disregarded. Because, the measured settlements of the high – rise buildings which were used as the database for the first reference study were the measurements taken after the installation of the raft, since the measuring instruments can only be installed to the foundation after the construction of the raft is completely finished. So, the settlements determined here are the settlements due to only the load of the superstructure. As a result, the settlements of the model until the seventh stage are reset to zero in the analysis. The step – by – step analysis of the second reference study, which is used in this thesis, is shown below in Table 3.5.

Table 3.5 Step–by–Step Analysis of Construction Process in Finite Element Calculations for Parametric Study

(Reul & Randolph, 2004)

Step		Applied load P_{eff} (MN)	Mean vertical effective stress at foundation level σ'_v (kPa)
I	In-situ stress state	—	126
II	Excavation to a depth of 7 m below ground level	—	63
III	Installation of the piles	—	63
IV	Excavation to a depth of 14 m below ground level	—	0
V	Application of the weight of the raft minus 50% of the uplift due to pore pressures as uniform load on the subsoil (zero stiffness of the raft)	57.8	40
VI	Installation of the raft	57.8	40
VII	Loading of the raft and application of 100% uplift	360.9	250
VIII	Further loading of the raft	721.7	500

3.3.2 The Method used in This Thesis

The method used in this thesis is ,in fact, just the same with the reference studies since the aim of the thesis is to compare the solutions of two different finite element softwares. However, due to the some restrictions of the used software “PLAXIS” and due to the some logical differences between “PLAXIS” and “ABAQUS” , there are some small differences in the steps of the analysis :

As a first difference, the parameters of the materials need to be adopted to the logic of “PLAXIS 8.2” software which needs the parameters to be converted to the plane – strain conditions. Also, some idealizations were needed for inserting the parameters into the “Plaxis 3D Foundation” software. These adaptations and idealizations will be described in detail in the next chapter. Moreover, the model can be extended to 10 times of the side length of the raft only in one direction due to the restrictions of the “PLAXIS 8.2” software, whereas it can be extended in two directions in both ABAQUS and “PLAXIS 3D Foundation” softwares. Also, although the model in the second reference study ,which will be used in the analysis of this thesis, was established as one eight of the total system due to the symmetry; the model is established as half and one quarter of the total system in “PLAXIS 8.2” and “PLAXIS 3D Foundation” softwares respectively.

Beyond these differences, all the chronology of the analysis, the applied load levels, and all other things are just the same with the analysis in the reference study (Reul & Randolph, 2004)

CHAPTER 4

THE FINITE ELEMENT ANALYSIS

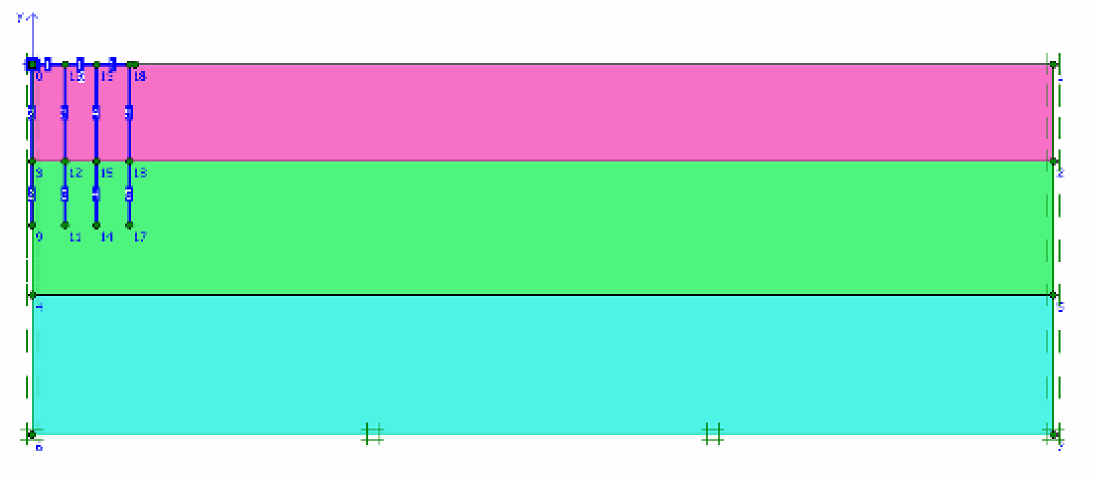
4.1 Introduction

For the parametric study performed in this thesis, a total of 100 different finite element analyses both with “Plaxis 8.2” and “Plaxis 3D Foundation” are performed. In this chapter; after the detailed description of the model and the mesh used in the two-dimensional and three-dimensional analysis together with the material parameters, the different configurations applied for the parametric study will be explained and then the results of the analysis will be discussed in detail.

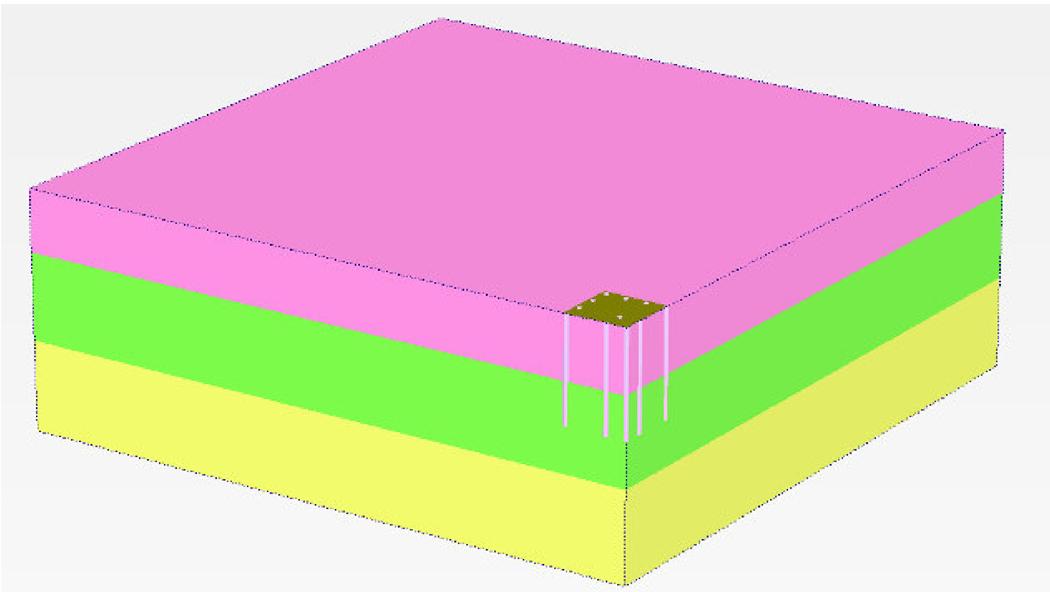
4.2 The Model Used in the Analysis

4.2.1 The General Properties of the Model

In this thesis, “Plaxis 8.2” and “Plaxis 3D Foundation” softwares are used for the two-dimensional and three-dimensional analysis respectively. Both for the two-dimensional and three-dimensional analysis, the model has a height of 69 m. which equals to the thickness of the clay layer below the foundation level. For the two dimensional analysis, half of the system is modeled whereas one quarter of the system is modeled for the three-dimensional analysis. The model is extended 10 times of the half side length of the raft (equals to 190 m.) only in the lateral direction in two-dimensional analysis due to the plane strain conditions while it is extended in both lateral and the orthogonal directions in the three-dimensional analysis. Two sample models for two-dimensional and three-dimensional analysis are given below in Fig 4.1 (a) & (b) respectively.



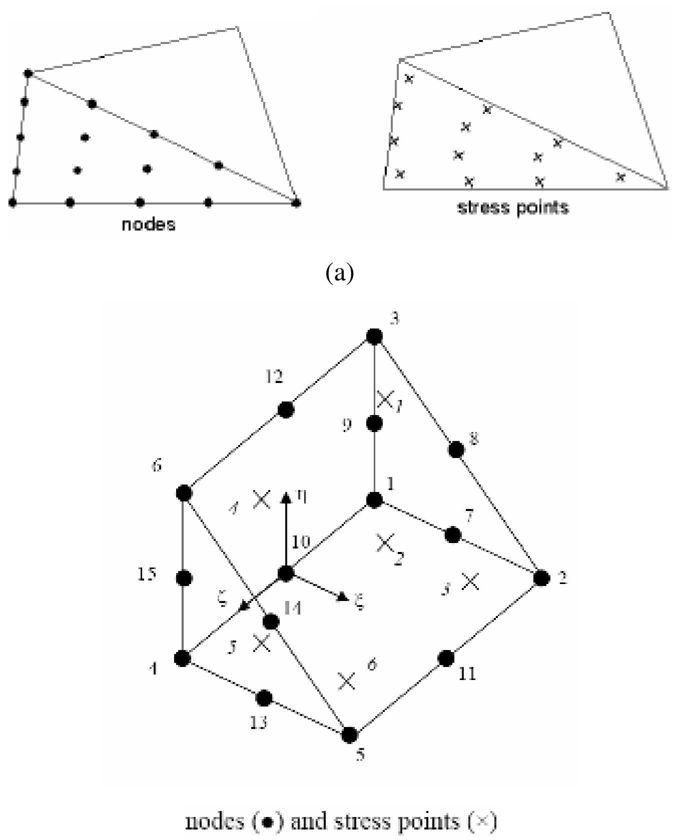
(a)



(b)

Fig 4.1 (a) Two – Dimensional Model, (b) Three Dimensional Model

The boundary conditions in all two-dimensional analysis are defined by the "Standard Fixities" provided automatically by the "Plaxis 8.2" software which allows the vertical movement while restricting the lateral movements. The boundary conditions for three-dimensional analysis are internally generated by the "Plaxis 3D Foundation" software in a similar logic with the plane strain conditions. Also, all interfaces were defined as rigid ,that is, no relative motion between the nodes of the finite elements that represent the structure and those of the finite elements that represent the soil takes place. 15 - node wedge elements are used both for the plane – strain and three-dimensional analysis. The schematic representation of the 15-node elements used in the analysis is provided below in Fig 4.2 (a) & (b).

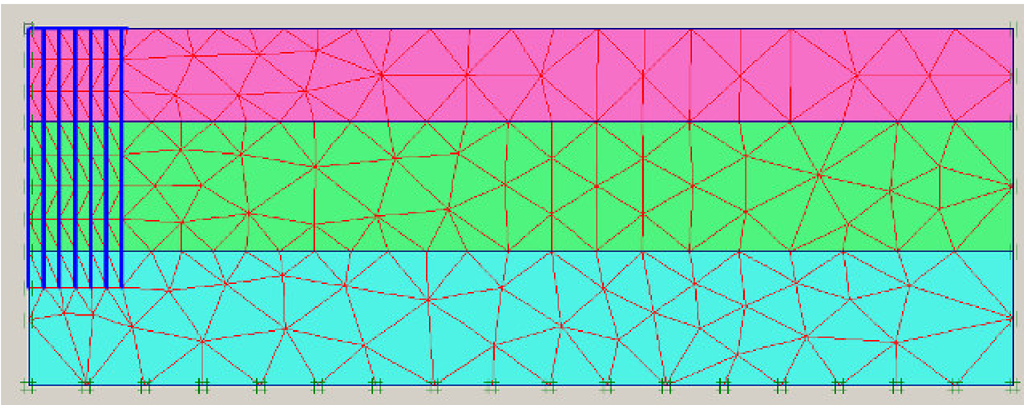


(b)

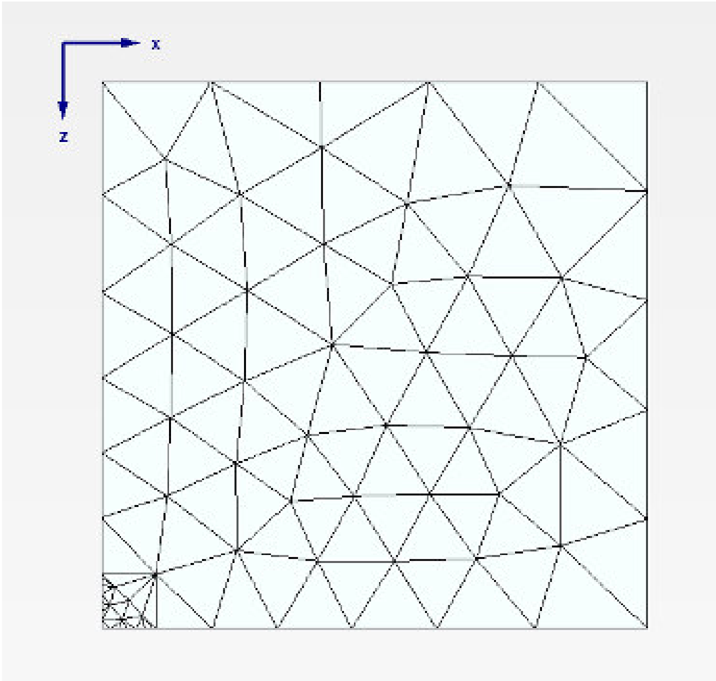
Fig 4.2 15-Node Wedge Elements (a) In 2D Analysis (b) In 3D Analysis
(Plaxis 3D Foundation Tutorial Manual, 2007)

4.2.2 The Mesh Generation

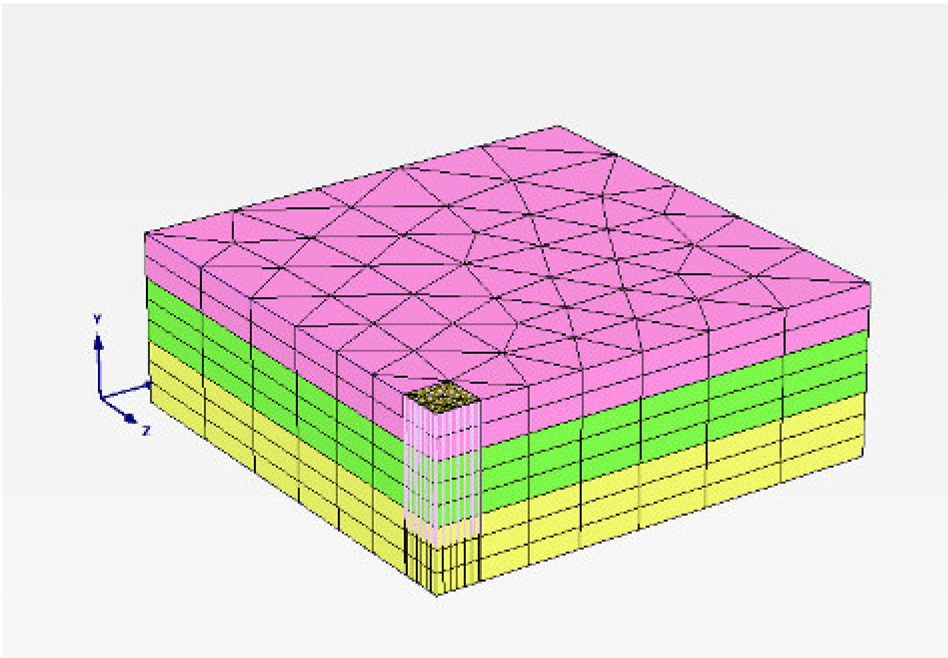
In all two-dimensional analysis, the mesh is generated in standard procedure provided automatically by the software and then refined for once globally. However, in three dimensional analysis, the logic does not work in the same manner. In all three-dimensional analysis a two-dimensional top-view mesh is generated first and refined for the cluster in which the raft is placed. Then the three-dimensional mesh is generated with the standard procedure of the software. Three sample meshes for the described conditions are provided below in Fig 4.3 (a), (b) & (c).



(a)



(b)



(c)

Fig 4.3 Sample Meshes Used in the Analysis

(a) 2D Mesh of 2D Analysis, (b) 2D Mesh of 3D Analysis, (c) 3D Mesh

4.3 The Material Parameters Used in the Analysis

Although the material parameters were introduced basically in the last chapter, some adaptations and idealizations were needed for modeling the behavior of the materials both in two-dimensional and three-dimensional analysis. The material parameters used in the analysis will be introduced in this part of the chapter together with describing the assumptions and idealizations employed for the determination of these parameters.

4.3.1 The Material Parameters Used in the 2D Analysis

4.3.1.1 The Parameters Used for the Subsoil

The Frankfurt clay was modeled to obey the Mohr-Coulomb failure criterion under drained conditions. The wet and dry unit weights were given as the same to the system and it has no effect on the solutions since the subsoil is fully saturated through the analysis. Moreover no permeability value was estimated since drained parameters are used already to analyze the systems.

Since the modulus of elasticity (E) of the Frankfurt clay increases with depth, the parameters of the soil layers are supplied for each layer, using the advanced options, for both two-dimensional and three-dimensional analyses, which allows the continuous increase of E through the soil layers. For this manner, firstly the soil thicknesses are determined in convenience with the change in Ko of the subsoil and regarding the change in the rate of increase of E per meter as the depth increases.

In determination of the thickness of the top soil layer (Layer 1), the change in Ko value was more important than the change in the rate of increase of E, since E was nearly constant for about 20 – 25 m. depth from the top. However, the Ko value was given by the reference studies as 0.72 until 25 m. depth below the surface of tertiary layers and 0.57 for below. So, the thickness of the first layer was determined as 18 m., since foundation level is at 7 m. depth below the top of the Frankfurt clay. The rate of change of E through 18 m. is taken to be constant and after determining the initial and final values of E, the initial E was given as an input parameter together with the change in E per meter through the soil layer.

For layers 2 and 3, the main concern in the determination of the layer thicknesses was the change in the rate of increase of E. In this manner, the layer thicknesses were determined as 25 m. and 26 m. for "Layer 2" and "Layer 3" respectively. The change in the rate of increase of E of the Frankfurt clay with depth is illustrated in Fig 4.4 below.

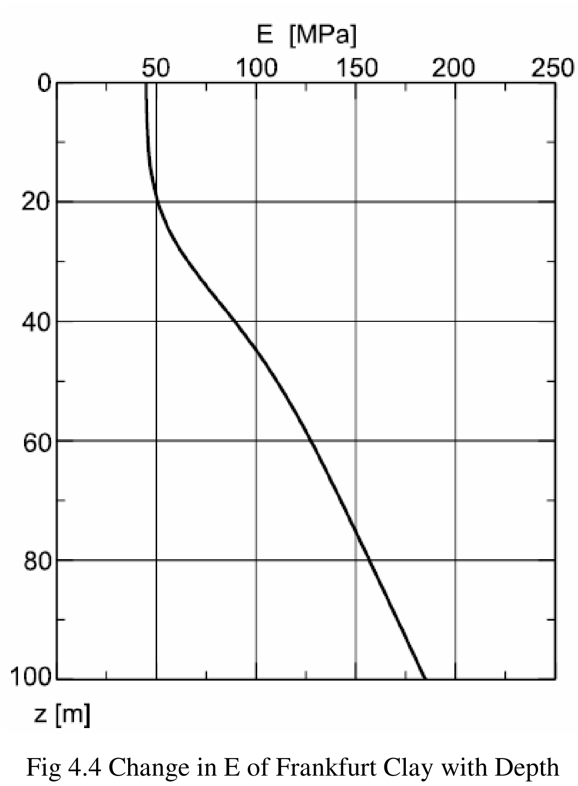


Fig 4.4 Change in E of Frankfurt Clay with Depth
(Reul, 2001)

E (MPa) : Modulus of Elasticity of Frankfurt Clay

Z (m) : Depth below surface of tertiary layers

All other parameters used in the analysis are directly taken from the reference studies described in the last chapter and are summarized below in Table 4.1.

Table 4.1 Soil Parameters used in the Analysis

	Layer 1	**Layer 2**	**Layer 3**
Thickness (m.)	18	25	26
Initial E (kPa)	45000	57000	110000
ΔE (kPa/m)	660	2138	1584
Ko	0.72	0.57	0.57
υ	0.15	0.15	0.15
c (kPa)	20	20	20
ϕ	20	20	20
γ (kN/m^3)	19	19	19

4.3.1.2 The Parameters Used for the Piles and the Raft

As mentioned in the last chapter, mainly there is no change made in the parameters of the piles and the raft which were taken from the second reference study and given in Table 3.3. However, the given parameters had to be converted into the plane-strain conditions (units / per meter).

The pile parameters are converted into the plane-strain conditions such that a row of pile provides equal rigidity per meter of the system. That is, the rigidity of a pile is multiplied with the number of piles in that row and then divided to the length of the one side of the raft (38m.). On the other hand, the raft parameters are determined by assuming a 1m. long and 3m. thick concrete block. The pile and the raft parameters used in the analysis are summarized below in Table 4.2.

Table 4.2 The Pile and The Raft Parameters used in the 2D Analysis

		EA (kN/m)	EI ($kN*m^2/m$)	υ	w (kN/m/m)
Pile Configuration - I	**n=169**	6.048E+06	5.040E+05	0.2	6
	n=49	3.250E+06	2.708E+05	0.2	6
Pile Configuration - II	**n=49**	3.250E+06	2.708E+05	0.2	6
	n=16	1.860E+06	1.547E+05	0.2	6
	n=9	1.393E+06	1.160E+05	0.2	6
Pile Configuration - III	**for 2 piles in a row**	9.284E+05	7.737E+04	0.2	6
	for 4 piles in a row	1.860E+06	1.547E+05	0.2	6
	for 5 piles in a row	2.321E+06	1.934E+05	0.2	6
	for 7 piles in a row	3.250E+06	2.708E+05	0.2	6
	for 9 piles in a row	4.170E+06	3.482E+05	0.2	6
Raft	**-**	1.020E+08	7.650E+07	0.2	0

4.3.2 The Material Parameters Used in the 3D Analysis

4.3.2.1 The Parameters Used for the Subsoil

All the parameters used for modeling the subsoil in the three-dimensional analysis are the same with the ones used in the two-dimensional analysis. Moreover, there is no change in the layer thicknesses since the same system is analyzed in both two – dimensional and three – dimensional analysis.

4.3.2.2 The Parameters Used for the Piles and the Raft

In three – dimensional analysis, the available parameters for the piles and the raft given in Table 3.3 are fully employed. However, for piles there arose a need to estimate some values which are essential to be input into the "Plaxis 3D Foundation" software.

For the piles, in addition to the material parameters, one has to input the skin resistance along the pile shaft and the base resistance of the pile. The base and total skin resistance of the piles, depending on their length, were determined by Reul (2001) for a pile head settlement of $s = 0.1d_p$ (d_p : pile diameter) (Fig 4.5) . The base resistance given in the cited study is directly used in the analysis. However, the pile shaft resistance should have to be modified since it was given as total skin resistance in the reference study and the used software database was designed such that a skin resistance value should be entered per meter. In this manner, firstly the total skin resistance of the pile with L_p=10m. was taken from the reference study. Then, assuming that the skin resistance per meter for the top 10 meters of soil is constant, the total skin resistance of the 10m. long pile is just divided to the length of the pile and this value is assumed to be equal to the both top and bottom skin resistances of the pile. For the longer piles used in the analysis (L_p = 30m. & 50m.), again the total skin resistances are directly taken from the cited study and after dividing this value to the length of the piles, the average skin resistance per meter along pile shaft is obtained for each pile length. Then, equating the top skin resistance of the longer piles to that of the 10m. long pile, the bottom skin resistance of the corresponding pile length is arranged so as to give the total skin resistance on average. The change in skin resistance along pile shaft is modeled as linear. On the other hand, there was no need to modify the material parameters used for the raft. However, the raft was assumed to behave as an isotropic material only. The parameters used for the piles and the raft in three – dimensional analysis are given below in Table 4.3.

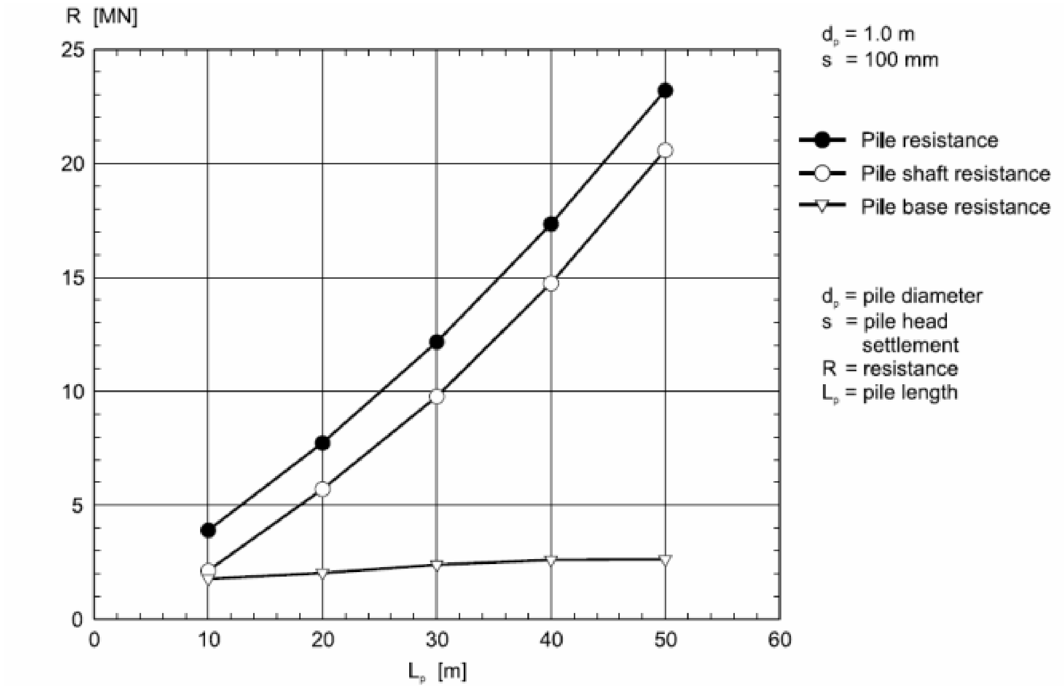


Fig 4.5 Variation of the Ultimate Pile Resistance of a Single Pile with the Pile Length (Reul, 2001)

Table 4.3 The Pile and The Raft Parameters used in the 3D Analysis

	Piles			**Raft**
	Lp=10m.	**Lp=30m.**	**Lp=50m.**	
E (MPa)	30000	30000	30000	34000
υ	0.2	0.2	0.2	0.2
Top Skin Resistance (kN/m)	210	210	210	-
Bottom Skin Resistance (kN/m)	210	430	610	-
Base Resistance (kN)	1900	2250	2500	-

4.4 The Parametric Study

In this study, a total of 100 analyses were performed which is equally shared between two – dimensional and three – dimensional analysis. In the parametric study, square unpiled and piled rafts with a side length of 38 m. have been considered. For the piled raft analysis, mainly three pile configurations were investigated. In "Pile Configuration 1", the piles were uniformly distributed under the whole raft area whereas in "Pile Configuration 2" the piles were concentrated in the central region of the raft. In "Pile Configuration 3" piles were located at the edges as well as in the central region of the raft. The analyses were performed for three different pile lengths of L_p = 10m. , 30m. & 50m. and the number of piles varied between n = 9 and n = 169. Moreover, the pile spacing varied between s = $3d_p$ and s = $6d_p$ while the pile diameter was held constant at d_p = 1.0 m. The raft thickness was taken as t_r = 3m. for all cases.

The analyses were performed for 2 different load levels, applied as uniform load on the whole model, in order to investigate the effect of the load level on the performance of the piled rafts. The first load level was defined to be equal to the 20% of the ultimate capacity of an equivalent unpiled raft (V_{ult} / P_{eff} = 5) reaching to a maximum level of P_{eff} = 721.7 MN. The second load level was defined as the 5% of the ultimate capacity of an equivalent unpiled raft and was equal to the one-fourth of the applied load of the first load level. A summary of the performed analysis are given below in Table 4.4:

Table 4.4 Summary of the Performed Analysis for the Parametric Study*

Pile Configuration	B (m.)	s/d_p	n	Lp (m)	n*Lp (m)	tr (m)	V_{ult} / P_{eff}
Unpiled Raft	38	-	-	-	-	3	5
Unpiled Raft	38	-	-	-	-	3	20
Pile Configuration 1	38	3	169	10	1690	3	5
Pile Configuration 1	38	3	169	10	1690	3	20
Pile Configuration 1	38	3	169	30	5070	3	5
Pile Configuration 1	38	3	169	30	5070	3	20
Pile Configuration 1	38	3	169	50	8450	3	5
Pile Configuration 1	38	3	169	50	8450	3	20
Pile Configuration 1	38	6	49	10	490	3	5
Pile Configuration 1	38	6	49	10	490	3	20
Pile Configuration 1	38	6	49	30	1470	3	5
Pile Configuration 1	38	6	49	30	1470	3	20
Pile Configuration 1	38	6	49	50	2450	3	5
Pile Configuration 1	38	6	49	50	2450	3	20
Pile Configuration 2	38	3	49	10	490	3	5
Pile Configuration 2	38	3	49	10	490	3	20
Pile Configuration 2	38	3	49	30	1470	3	5
Pile Configuration 2	38	3	49	30	1470	3	20
Pile Configuration 2	38	3	49	50	2450	3	5
Pile Configuration 2	38	3	49	50	2450	3	20
Pile Configuration 2	38	6	16	10	160	3	5
Pile Configuration 2	38	6	16	10	160	3	20
Pile Configuration 2	38	6	16	30	480	3	5
Pile Configuration 2	38	6	16	30	480	3	20
Pile Configuration 2	38	6	16	50	800	3	5
Pile Configuration 2	38	6	16	50	800	3	20
Pile Configuration 2	38	6	9	10	90	3	5
Pile Configuration 2	38	6	9	10	90	3	20
Pile Configuration 2	38	6	9	30	270	3	5
Pile Configuration 2	38	6	9	30	270	3	20
Pile Configuration 2	38	6	9	50	450	3	5
Pile Configuration 2	38	6	9	50	450	3	20
Pile Configuration 3	38	3	73	10	730	3	5
Pile Configuration 3	38	3	73	10	730	3	20
Pile Configuration 3	38	3	73	30	2190	3	5
Pile Configuration 3	38	3	73	30	2190	3	20
Pile Configuration 3	38	3	73	50	3650	3	5
Pile Configuration 3	38	3	73	50	3650	3	20
Pile Configuration 3	38	6	40	10	400	3	5
Pile Configuration 3	38	6	40	10	400	3	20
Pile Configuration 3	38	6	40	30	1200	3	5
Pile Configuration 3	38	6	40	30	1200	3	20
Pile Configuration 3	38	6	40	50	2000	3	5
Pile Configuration 3	38	6	40	50	2000	3	20
Pile Configuration 3	38	6	33	10	330	3	5
Pile Configuration 3	38	6	33	10	330	3	20
Pile Configuration 3	38	6	33	30	990	3	5
Pile Configuration 3	38	6	33	30	990	3	20
Pile Configuration 3	38	6	33	50	1650	3	5
Pile Configuration 3	38	6	33	50	1650	3	20

*The Summarized Analysis are Performed Both in 2D and 3D

4.5 Results

4.5.1 The Definitions

Following parameters are defined to be used for the interpretation of the results of the parametric study.

$$S_{average} \approx \frac{1}{3}\left(2S_{centre} + S_{corner}\right) \tag{4.1}$$

where ;

$S_{average}$: average settlement of the foundation under uniform loading

S_{centre} : the settlement at the centre of the raft

S_{corner} : the settlement at the corner of the raft

Δs : differential settlement defined as the difference between the centre and mid-side settlements

$$\xi_s = \frac{s_{average}}{s_{average,r}} \tag{4.2}$$

where ;

ξ_s : coefficient for the average settlement

$s_{average}$: the average settlement of the investigated case

$s_{average,r}$: the average settlement of the unpiled raft

$$\xi_{\Delta s} = \frac{\Delta s}{\Delta s_r} \tag{4.3}$$

where ;

$\xi_{\Delta s}$: coefficient for the differential settlement

Δs : differential settlement of the investigated case

Δs_r : differential settlement of the unpiled raft

$$R_{s-1} = \frac{s_{avg,2D}}{s_{avg,3D(Abaqus)}} \tag{4.4}$$

$$R_{s-2} = \frac{s_{avg,3D(Plaxis)}}{s_{avg,3D(Abaqus)}} \tag{4.5}$$

$$R_{s-3} = \frac{s_{avg,2D}}{s_{avg,3D(Plaxis)}} \tag{4.6}$$

where,

$s_{avg,2D}$: the average settlements calculated by the plane – strain "Plaxis" analysis

$s_{avg,3D(Plaxis)}$: the average settlements calculated by the three - dimensional "Plaxis" analysis

$s_{avg,3D(Abaqus)}$: the average settlements calculated in the reference study

$$R_{\Delta s-1} = \frac{\Delta s_{2D}}{\Delta s_{3D,Abaqus}} \tag{4.7}$$

$$R_{\Delta s-2} = \frac{\Delta s_{3D,Plaxis}}{\Delta s_{3D,Abaqus}} \tag{4.8}$$

$$R_{\Delta s-3} = \frac{\Delta s_{2D}}{\Delta s_{3D,Plaxis}} \tag{4.9}$$

where,

Δs_{2D} : the differential settlements calculated by the plane – strain "Plaxis" analysis

$\Delta s_{3D,Plaxis}$: the differential settlements calculated by the three - dimensional "Plaxis" analysis

$\Delta s_{3D,Abaqus}$: the differential settlements calculated in the reference study

4.5.2 The Results for Pile Configuration – 1

As discussed before, in Pile Configuration – 1, the piles are located as uniformly distributed under the whole raft area and the analysis were carried out for n = 169 and n=49 piles for s = $3d_p$ and s = $6d_p$ respectively. The results will be presented below in two categories as "Average Settlements" and "Differential Settlements" both for two – dimensional and three – dimensional analysis:

4.5.2.1 The Results of the Plane – Strain Analysis

Since the subject of the thesis is a comparison between different analyses, the results of the plane – strain analysis will be compared with the original solutions given in the reference study and with the three – dimensional analysis performed in this thesis.

4.5.2.1.1 Average Settlements

- For V_{ult} / P_{eff} = 5 : The average settlements calculated by the plane – strain analysis were always higher than the original values. Moreover, the calculated values were approximately two times of the original values given in the reference study, R_{s-1} being between 1.575 – 2.790, with an average value of 2.191. This is illustrated in Fig 4.6 below, which represents the distribution of this ratio over total pile length (n*L_p (m.)).

The R_{s-3} values which refers to the ratio between the two – dimensional and three – dimensional "Plaxis" analyses, accumulated in the 1.6 – 1.9 band with an average value of 1.752, which shows the overestimation of the plane – strain solutions compared to the three – dimensional ones.

Expected decrease of the average settlement of the foundations with the introduction of piles (indicated by ξ_s) is in very good agreement with the values presented in the original study, having only 0.2 % underestimated values on average as compared to the reference study. A comparative graph is provided below as Fig 4.7

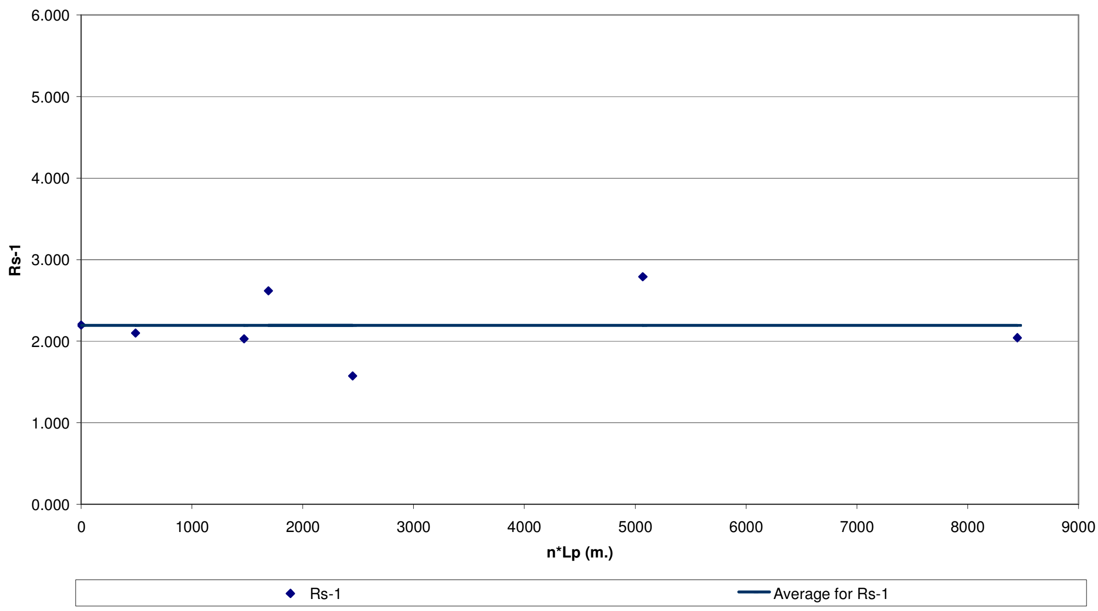


Fig 4.6 R_{s-1} vs. Total Pile Length (n^*L_p) for Pile Configuration - 1 & V_{ult} / P_{eff} = 5

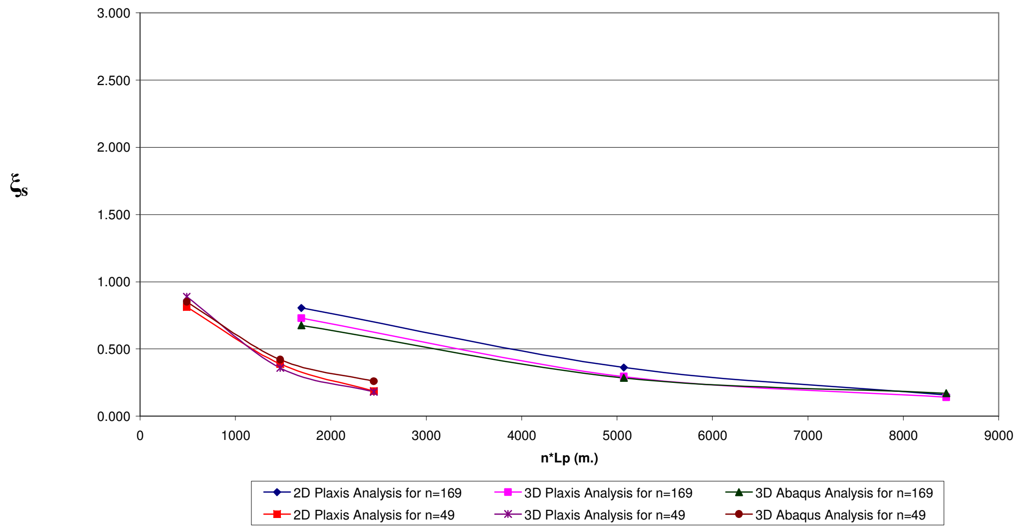


Fig 4.7 ξ_s vs. n*L_p for Pile Configuration – 1 & $V_{ult} / P_{eff} = 5$

- For $V_{ult} / P_{eff} = 20$: The solutions for this load level were more close to the original values of the reference study which is indicated by the smaller R_{s-1} ratios. The R_{s-1} ratios varied between 1.305 – 1.634 with an average value of 1.479.

The R_{s-3} value obtained for this load level changes between 1.422 – 1.596 with an average value equal to 1.519.

The ξ_s calculated for this load level is in good accordance with the values presented in the original study with an only 1.5 % underestimation on the average as compared to the reference study.

4.5.2.1.2 Differential Settlements

- For V_{ult} / P_{eff} = 5 : The calculated differential settlements for the analyzed cases were generally slightly larger than the original values with $R_{\Delta s-1}$ values close to the unity, excluding a marginal value of 0.518 which was obtained for n=169 & L_p =10 m. The $R_{\Delta s}$ values varied between 0.518 – 1.400 with an average value of 1.064. A graph illustrating the change of $R_{\Delta s-1}$ with total pile length (n*L_p) is provided below in Fig 4.8.

  The $R_{\Delta s-3}$ values calculated for this load level shows a deviation of 11 % from the original values on average. But this value does not give an opportunity to make a generalization since there occurs a high standard deviation in the $R_{\Delta s-3}$ values.

  Unfortunately, the $\xi_{\Delta s}$ values obtained for this load level can not be said to be close to the values in the reference study. The obtained values show 68 % deviation from the original values on average. A comparative graph (including the results of the three – dimensional analysis), created for n=49 piles, is provided in Fig 4.9 below.

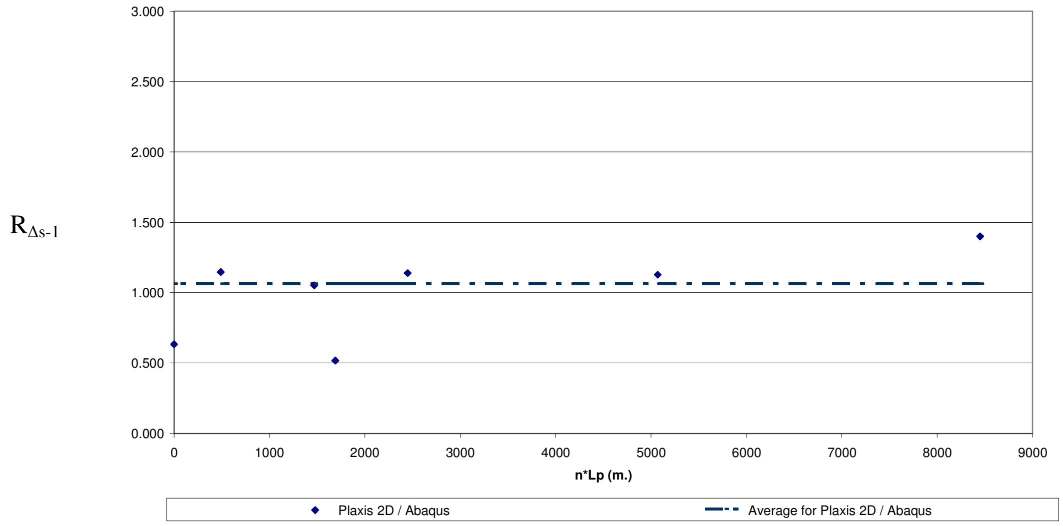


Fig 4.8 $R_{\Delta s\text{-}1}$ vs. n^*L_p for Pile Configuration – 1 & V_{ult} / P_{eff} = 5

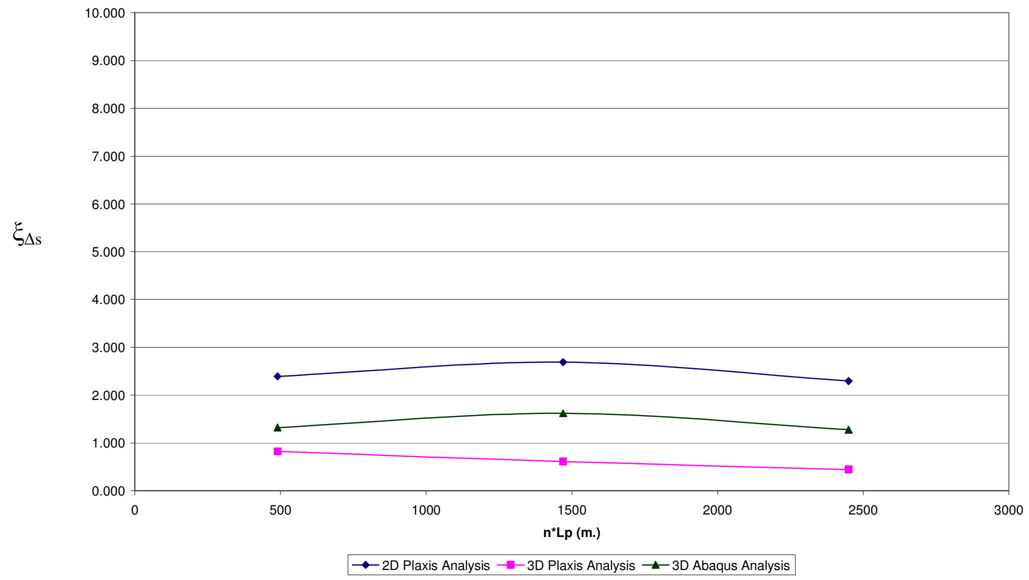


Fig 4.9 $\xi_{\Delta s}$ vs. n^*L_p for Pile Configuration – 1, n=49 & V_{ult} / P_{eff} = 5

- For V_{ult} / P_{eff} = 20 : At this load level, the obtained results were readily comparable with the original values. The calculated differential settlements were generally larger than the ones in the reference study. But, the $R_{\Delta s\text{-}1}$ values showed a small standard deviation concentrating on 1.300 – 1.500 narrow band with an average value of 1.341. The change of $R_{\Delta s\text{-}1}$ with total pile length n^*L_p is presented below as Fig 4.10.

Although the calculated $R_{\Delta s\text{-}3}$ values showed a higher deviation (24.8%) on the average compared to the one for the load level V_{ult} / P_{eff} = 5, the results were better since they concentrated between 1.1 and 1.4.

Opposite to the situation in the last discussion, the calculated $\xi_{\Delta s}$ values were very close to the ones given in the reference study. The calculated $\xi_{\Delta s}$ values showed only a 4.5 % difference compared to the original values. The case is illustrated in Fig 4.11 together with the $\xi_{\Delta s}$ plot of the three – dimensional analysis.

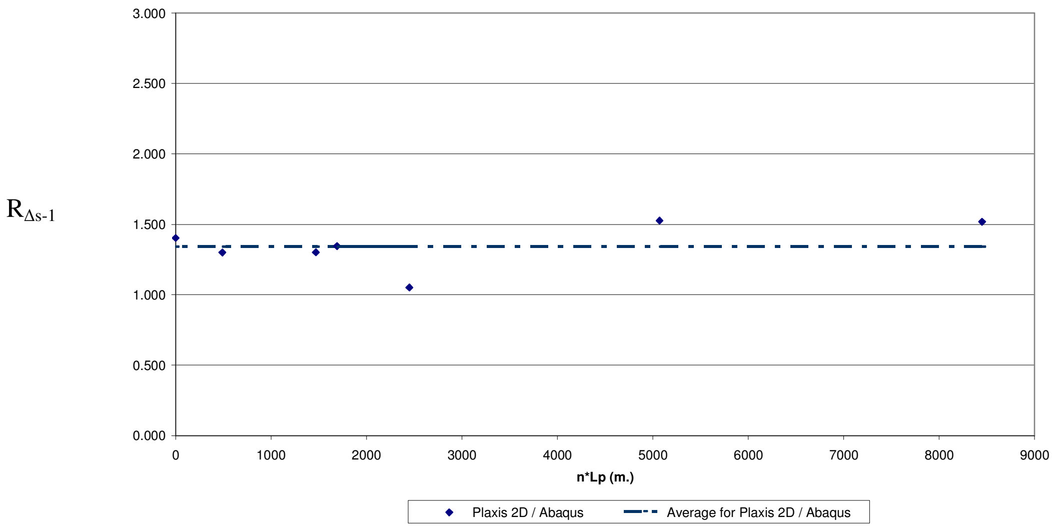


Fig 4.10 $R_{\Delta s\text{-}1}$ vs. n^*L_p for Pile Configuration – 1 & V_{ult} / P_{eff} = 20

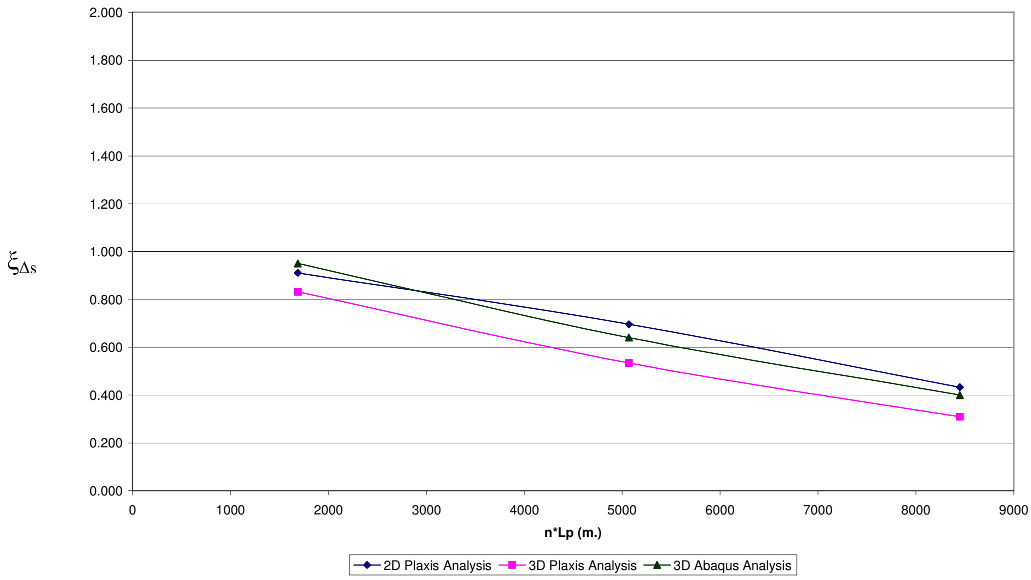


Fig 4.11 $\xi_{\Delta s}$ vs. $n{*}L_p$ for Pile Configuration – 1, n=169& $V_{ult} / P_{eff} = 20$

4.5.2.2 The Results of the Three – Dimensional Analysis

In this section of the chapter, the results obtained by the "Plaxis 3D Foundation" software in the analysis of this thesis will be compared with the solutions of "ABAQUS" software which was employed in the reference study.

4.5.2.2.1 Average Settlements

- For $V_{ult} / P_{eff} = 5$: The average calculated settlements were all higher than the original values except for n=49 piles with L_p=50m. As a result, the $R_{s\text{-}2}$ values were all found above unity except for the mentioned analysis. The $R_{s\text{-}2}$ values fall between 0.941 and 1.462 with an average of 1.249, implying 25% overestimation as compared to the values given in the reference study.

The calculated ξ_s values are in accordance with the ones given in the reference study. Order of magnitude of these values and the general trend of the curve had similar characteristics with the original study. Correspondingly, the calculated ξ_s values showed a small deviation of -7.9% from the original values on the average.

- For V_{ult} / P_{eff} = 20 : The average settlements calculated for this load level were very close to the ones given in the cited study. As a result, nearly all the $R_{s\text{-}2}$ values were slightly less or more than the unity. The average $R_{s\text{-}2}$ was determined to be equal to 0.974 indicating only a -2.6% deviation from the original values on average.

4.5.2.2.2 Differential Settlements

- For V_{ult} / P_{eff} = 5 : Although, the error in the differential settlements calculated were in tolerable limits, unfortunately the calculated result did not show a general trend of underestimation and overestimation. However, it can still be said that the results were comparable with the original values with an average $R_{\Delta s\text{-}2}$ of 1.231 varying between 0.9 – 1.6.

  The $\xi_{\Delta s}$ values calculated by the "Plaxis 3D Foundation" were not in good agreement with original values. Moreover, the calculated values were approximately 40% of the original values.

- For V_{ult} / P_{eff} = 20 : Interestingly, the differential settlements calculated for this load level were close to the values obtained in the reference study. Although the calculated $R_{\Delta s\text{-}2}$ values varied in the range of 0.8 – 1.2, the values were concentrated near unity and average $R_{\Delta s\text{-}2}$ value was equal to 1.077 indicating a 7.7% overestimation of the "original" differential settlement on average only.

On the other hand, the $\xi_{\Delta s}$ values were about 77% of the original ones within a small margin of 61% - 82%.

4.5.3 The Results for Pile Configuration – 2

As mentioned before, the piles are located only in the central region of the raft in Pile Configuration – 2. The analysis were carried out for n=49, n=16 & 9 piles for s = 3d_p and s = 6d_p respectively for the prescribed conditions. When the piles are concentrated in the central region of the raft, the corner of the raft tends to settle more than the central region in contrast with the general settlement behavior. This phenomenon is called "hogging" and illustrated below in Fig 4.12. The results will be presented below in two categories as "Average Settlements" and "Differential Settlements" both for two – dimensional and three – dimensional analysis:

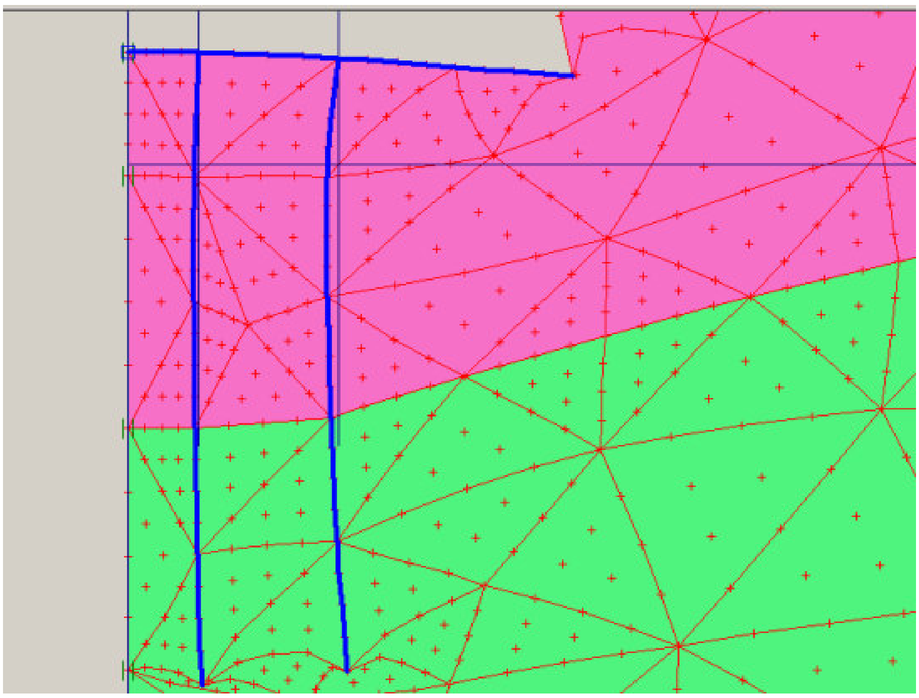


Fig 4.12 "Hogging" Phenomenon

4.5.3.1 The Results of the Plane – Strain Analysis

4.5.3.1.1 Average Settlements

- For V_{ult} / P_{eff} = 5 : For this load level, the average settlements calculated by the plane – strain solutions are generally larger than the original values. Moreover, the R_{s-1} values generally fall in a band close to 2, similar to the situation for "Pile Configuration – 1", with an average value of 1.922.

  The R_{s-3} values obtained for this pile configuration, concentrates between 1.1 – 1.5 with an average value of 1.311. Again it is seen that, the plane – strain solutions overestimate the average settlements. However, the average R_{s-3} value obtained for this pile configuration is surprisingly lower than the one in "Pile Configuration – 1" which indicates that the two – dimensional analysis gives a closer average settlement to the three – dimensional analysis for centrally located piles.

  The ξ_s values obtained, are generally in accordance with the values provided in the reference study. The calculated ξ_s are only 12.5 % below the original values. A sample graph plotted for the configuration with n = 9 piles is provided below as Fig 4.13.

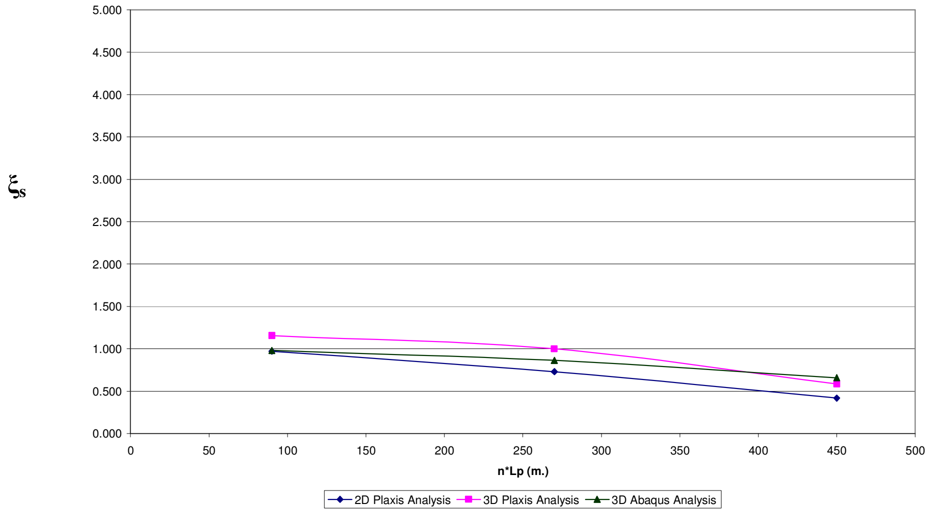


Fig 4.13 ξ_s vs. n^*L_p for Pile Configuration – 2, n=9 & V_{ult} / P_{eff} = 5

- For V_{ult} / P_{eff} = 20 : The calculated average settlements at this load level is all higher than the original values. The R_{s-1} values are between the 1.2 – 1.4 values with an average of 1.325.

The R_{s-3} values obtained for this load level is generally close to the ones in the previous one, however the average is slightly larger. The average R_{s-3} value is determined as 1.468 for this load level.

The calculated ξ_s values are very close to the ones given in the reference study only with a small deviation of 9.1 % on average. It should be emphasized that, the calculated ξ_s values show a better agreement with the original values at this lower load level compared to the case in V_{ult} / P_{eff} = 5. A sample graph plotted for the configuration with n = 9 piles is provided below as Fig 4.14.

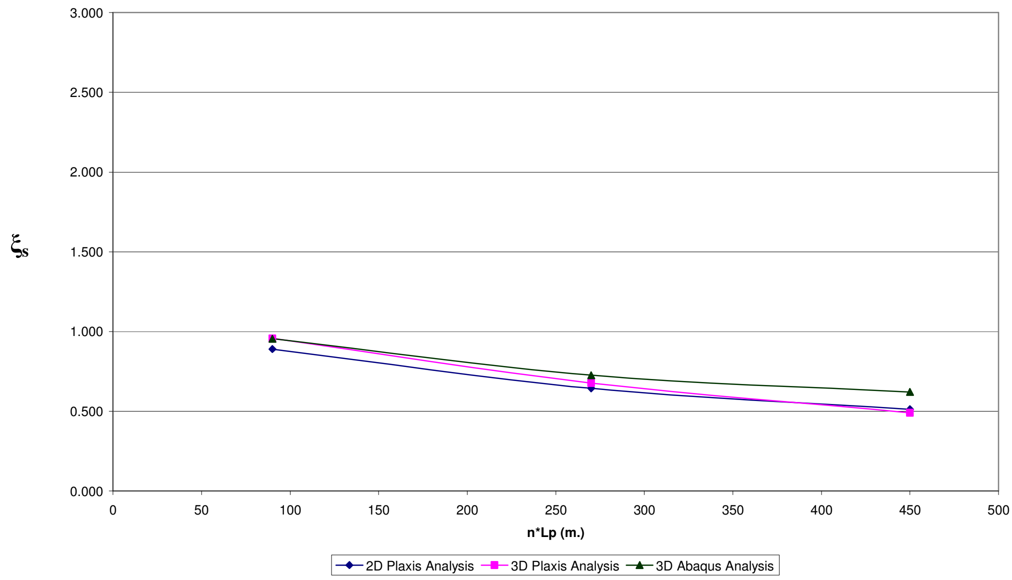


Fig 4.14 ξ_s vs. $n*L_p$ for Pile Configuration – 2, n=9 & V_{ult} / P_{eff} = 20

4.5.3.1.2 Differential Settlements

- For V_{ult} / P_{eff} = 5 : The highly scattered results were obtained for this option throughout the study. The calculated differential settlements were sometimes in different directions compared to the original study. In the analysis with the shortest pile length (10m.) the plane – strain analysis gives negative differential settlements (hogging) while the ABAQUS software in the original study yields positive values. The obtained $R_{\Delta s\text{-}1}$ values were in a large range such that marginal values like 20.384 were included. Although the average value was equal to the 3.068, it had no meaning since no correlation could be made between the performed and the original analysis. The $R_{\Delta s\text{-}1}$ values for the corresponding total pile length are plotted below as Fig 4.15.

Correspondingly, the $R_{\Delta s\text{-}3}$ values obtained for this load level are not so meaningful since they vary in a large range. Although the average $R_{\Delta s\text{-}3}$

value is obtained as nearly equal to the minus unity (-1.008), no correlation could be made between the two – dimensional and three – dimensional analysis.

Also, the rational decrease in the differential settlements of the unpiled rafts with the addition of piles ($\xi_{\Delta s}$) was also wrongly estimated by the plane – strain analysis making any correlation unavailable. The discordance of the $\xi_{\Delta s}$ values is illustrated in Fig 4.16.

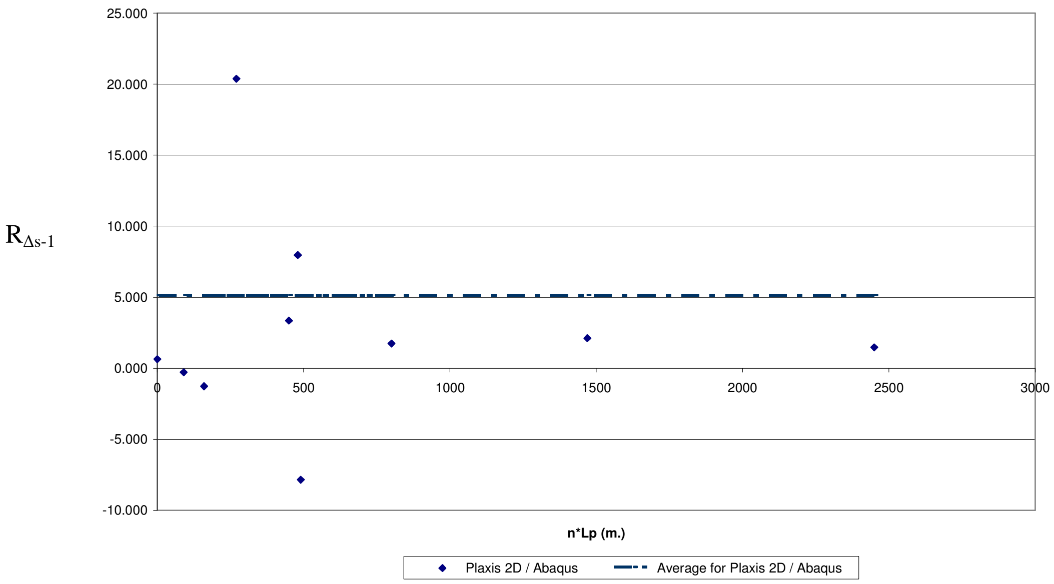


Fig 4.15 $R_{\Delta s-1}$ vs. n^*L_p for Pile Configuration – 2 & V_{ult} / P_{eff} = 5

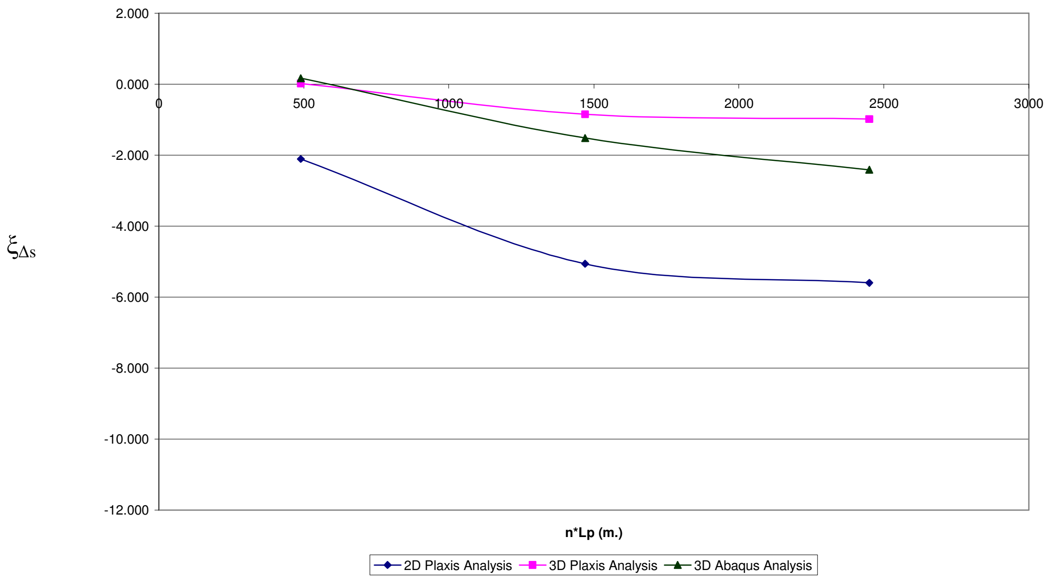


Fig 4.16 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=49 & V_{ult} / P_{eff} = 5

- For V_{ult} / P_{eff} = 20 : Opposite to the prescribed situation in the previous load level, the directions of the differential settlements obtained from the plane – strain analysis were in accordance with the values given in the reference study for all analysis. Moreover, the $R_{\Delta s\text{-}1}$ values showed a concentration in the range of 1.3 – 1.9 excluding the values obtained for the analysis performed for L_p = 10m. For L_p = 10m. differential settlements were interestingly underestimated in plane – strain analysis whereas it was overestimated in the other analysis options. An average $R_{\Delta s\text{-}1}$ value of 1.526 is obtained when the results of the analysis for L_p=10m. are excluded and the average value decreases to 1.175 when these results are included.

The calculated $R_{\Delta s\text{-}3}$ values show a similar characteristic as the $R_{\Delta s\text{-}1}$ values indicating a better correlation between the three – dimensional analyses and the reference study. Again the differential settlements for L_p=10m. are underestimated and the others are overestimated. An average $R_{\Delta s\text{-}3}$ value of

1.522 is obtained when the underestimated results are excluded and the average value becomes equal to 1.141 when these values are included.

The $\xi_{\Delta s}$ values obtained from the plane – strain analysis were generally below the ones given in the reference study. The underestimation was again larger for the analysis carried out for L_p = 10m. whereas the results were slightly overestimated for some of the analysis with n=9 piles. The $\xi_{\Delta s}$ showed a deviation of -16.3 % from the original results on average, as shown in the Fig 4.17.

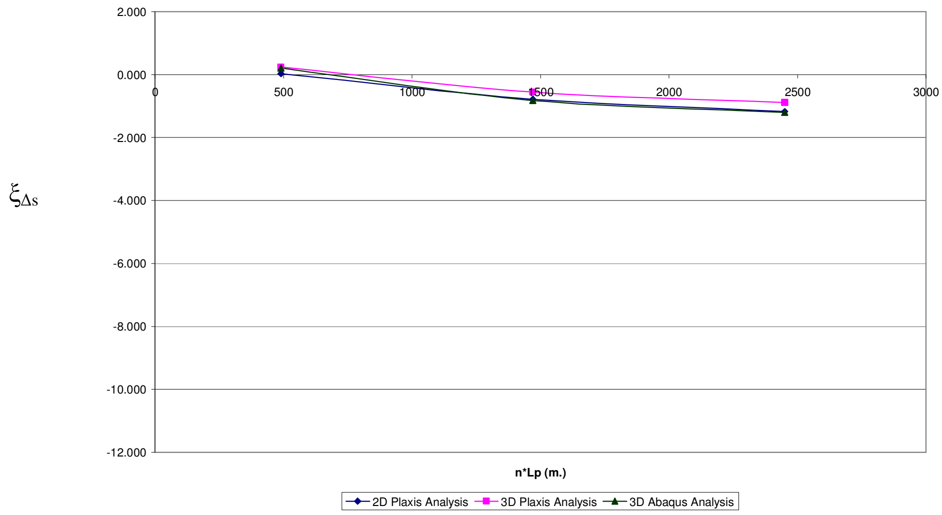


Fig 4.17 $\xi_{\Delta s}$ vs. $n*L_p$ for Pile Configuration – 2, n=49 & V_{ult} / P_{eff} = 20

4.5.3.2 The Results of the Three – Dimensional Analysis

4.5.3.2.1 Average Settlements

- For V_{ult} / P_{eff} = 5 : The average settlements calculated by "Plaxis 3D Foundation" were larger than the ones given in the original study by

Reul & Randolph. The R_{s-2} ratio calculated for this load level changed between 1.1 – 1.7 with an average of 1.474.

The values of ξ_s calculated by 3D Plaxis analyses were in very good agreement with the original values both in magnitude and trend. As a result, the ξ_s values showed only a small deviation of 8.8% from the original values on average.

- For V_{ult} / P_{eff} = 20 : The average settlements at this load level were generally slightly smaller than the original values. The calculated R_{s-2} ratio ranged between 0.8 - 1.0 (excluding a marginal value of 0.67 for n=16 and L_p=50m.) with an average of 0.915 indicating a reasonable correlation between the calculated and the original values.

  The ξ_s values obtained by the three – dimensional analyses are in accordance with the ones given in the reference study. The deviation was only at - 9.9% on average.

4.5.3.2.2 Differential Settlements

- For V_{ult} / P_{eff} = 5 : Unfortunately, the calculated differential settlements showed a large deviation from the original values at this load level. The calculated differential settlements reached up to 5 times of the original value for some of the cases. Although an average value of 1.725 was obtained for $R_{\Delta s-2}$, the result had no meaning for engineering point of view since no correlation could be developed between the calculated and the original values.
  The discordance between the calculated and the original values are also observed for the $\xi_{\Delta s}$ ratio. The calculated ratios ranged between 0.1 – 2.1 and had no correlation with the original values.

- For V_{ult} / P_{eff} = 20 : The three – dimensional solutions were better in estimating the differential settlements of the concerned pile configuration at this load level as compared to the former case. The $R_{\Delta s\text{-}2}$ value ranged between 0.777 and 1.583 with an average value of 1.128 on the average.

  Also, the $\xi_{\Delta s}$ ratios calculated for this load level were in better accordance with the original values, having a deviation of -18.6 % on the average.

4.5.4 The Results for Pile Configuration – 3

In pile configuration – 3, the piles are located at the edges as well as in the central region of the raft. This type of a smart location of the piles prevents hogging of the raft and decreases the differential settlements while providing a similar performance to pile configuration – 1 in decreasing the average settlements with smaller number of piles. The analysis were carried out for n=73, n=40 & 33 piles for s = $3d_p$ and s = $6d_p$ respectively for the prescribed conditions. The results will be presented below in two categories as "Average Settlements" and "Differential Settlements" both for two – dimensional and three – dimensional analysis:

4.5.4.1 The Results of the Plane – Strain Analysis

4.5.4.1.1 Average Settlements

- For V_{ult} / P_{eff} = 5 : The average settlements obtained for the concerned pile configuration showed similar characteristics with the pile configuration – 1. The calculated average settlements by the plane – strain analyses were all larger than the 3D results, being nearly two times of the values given in the reference study. The $R_{s\text{-}1}$ ratio ranged between 1.5 – 2.2 and was equal to 1.912 on average.

The calculated R_{s-3} values were all in the range of 1.3 – 1.7 with small deviations from the average value of 1.542 which indicates that the plane – strain solutions give approximately 1.5 times larger average settlements than the ones obtained from the three – dimensional "Plaxis" analysis.

Moreover, the calculated ξ_s values were generally underestimated as compared to the ones in the reference study, being about 13% smaller on the average. The change of the ξ_s values with total pile length $n*L_p$ is illustrated in Fig 4.18.

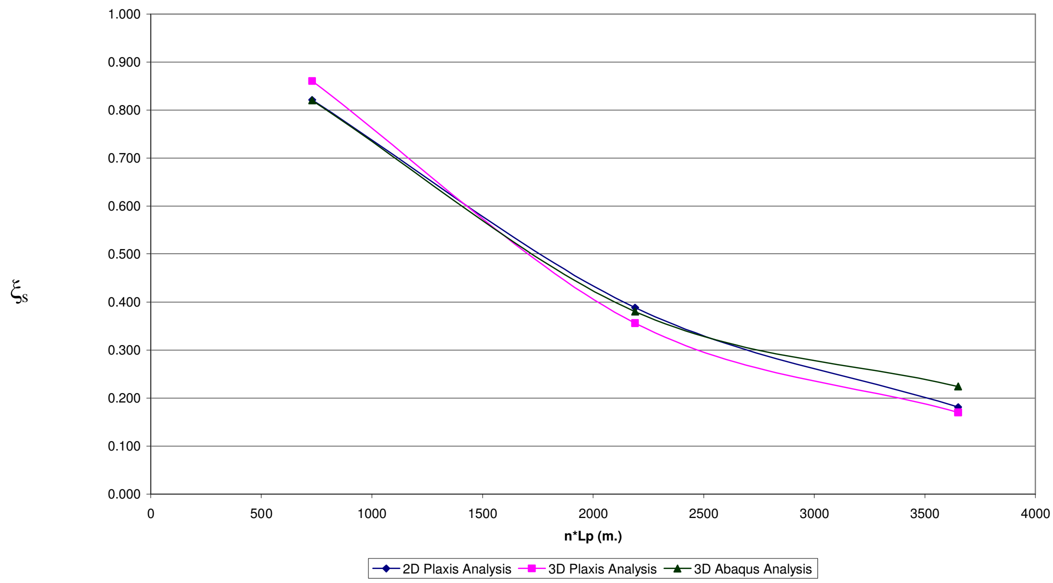


Fig 4.18 ξ_s vs. $n*L_p$ for Pile Configuration – 3, n=73 & $V_{ult} / P_{eff} = 5$

- For $V_{ult} / P_{eff} = 20$: The calculated average settlements were higher than the original values for this load level. The R_{s-1} ratio varied between 1.3 – 1.6 with an average value equal to 1.414.

Moreover, the average R_{s-3} value was determined to be equal to 1.459 and ranged between 1.367 and 1.560.

The calculated ξ_s values were very close to the original values with a deviation of 10 % in general. On average the deviation was only -% 3 indicating a high correlation between the calculated and the given ξ_s values.

4.5.4.1.2 Differential Settlements

- For V_{ult} / P_{eff} = 5 : Although the calculated differential settlements were not so close to the ones given in the reference study, they were at least in convenience in terms of the direction of the settlements. Correspondingly, although a meaningful correlation could not be made, the average $R_{\Delta s-1}$ was determined to be equal to 1.163 having a range of 0.4 – 2.1.

  The values obtained for $R_{\Delta s-3}$ showed a better picture than the ones obtained for the $R_{\Delta s-1}$ ratio. The differential settlements were generally underestimated in plane – strain analysis as compared to the ones in the three – dimensional "Plaxis" analysis, having a deviation of -12.2 % on the average. It is seen that, the deviation reaches up to a maximum value of -66 % for the analysis with L_p = 10m. indicating that making a generalization based on average values may lead to highly overconservative solutions for this option.

  The $\xi_{\Delta s}$ values obtained for this load level are significantly larger than the ones in the reference study. The average deviation from the original values is determined as 83.5 %. The situation is illustrated in Fig. 4.19 below.

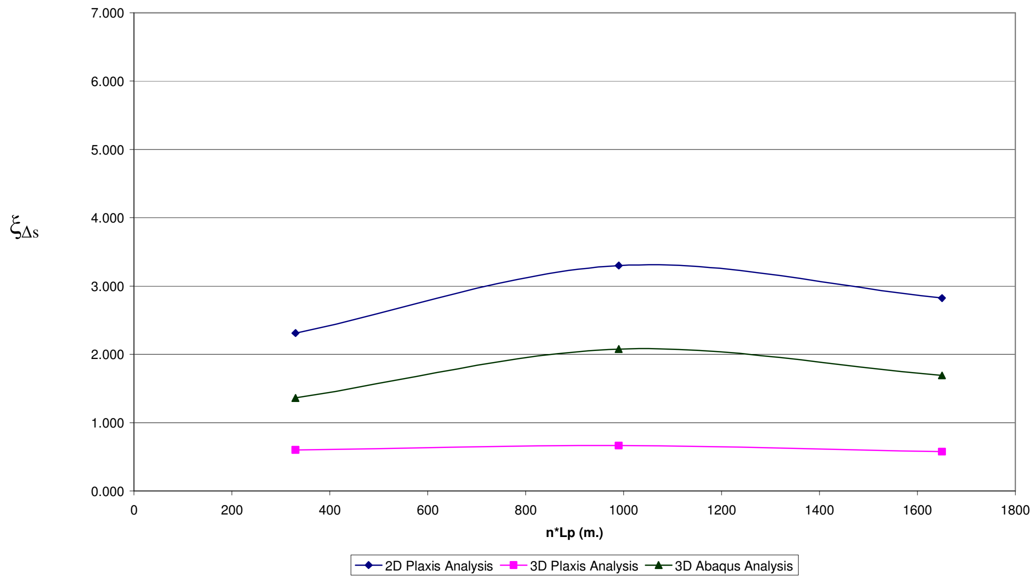


Fig 4.19 $\xi_{\Delta s}$ vs. $n{*}L_p$ for Pile Configuration – 3, n=33 & $V_{ult} / P_{eff} = 5$

- For $V_{ult} / P_{eff} = 20$: The differential settlements calculated by the two – dimensional analysis for all studied cases, are larger than the ones in the reference study. Correspondingly, the calculated $R_{\Delta s\text{-}1}$ values were all above unity having an average value of 1.438 in a range of 1.1 – 2.7.

The $R_{\Delta s\text{-}3}$ values obtained for this load level are between 1.0 – 1.4 having an average value of 1.180 which indicates that the results of the plane – strain solutions are close those of the three – dimensional "Plaxis" analysis for this pile configuration and load level.

The calculated $\xi_{\Delta s}$ values showed a much better agreement with the original values when compared with the situation in the former case. The calculated $\xi_{\Delta s}$ were generally very close to the ones in the reference study, having a small deviation of 2.4 % on average. The situation is illustrated in Fig 4.20 below.

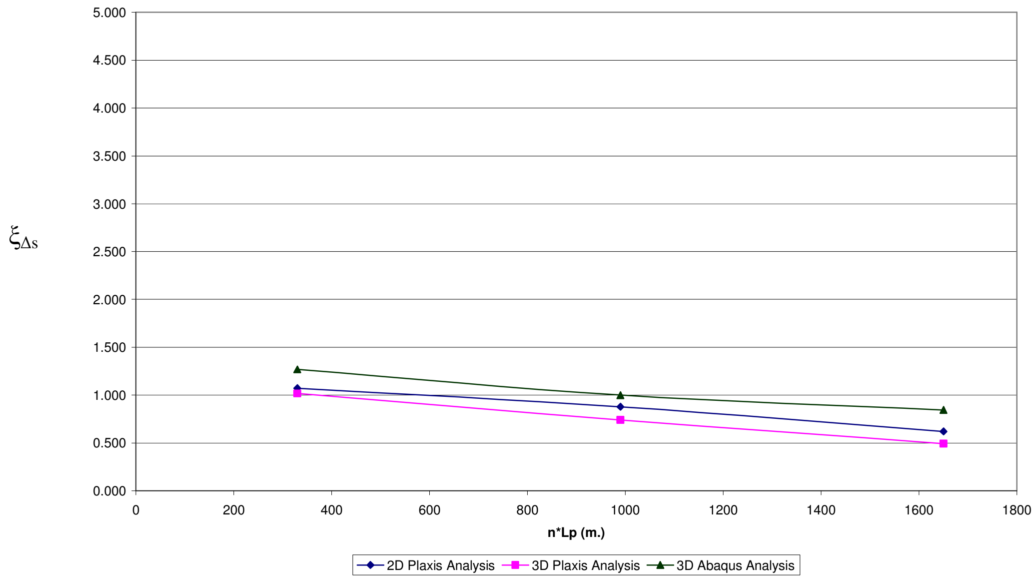


Fig 4.20 $\xi_{\Delta s}$ vs. $n{*}L_p$ for Pile Configuration – 3, n=33 & V_{ult} / P_{eff} = 20

4.5.4.2 The Results of the Three – Dimensional Analysis

4.5.4.2.1 Average Settlements

- For V_{ult} / P_{eff} = 5 : Similar to the situation in plane – strain analysis, the $R_{s\text{-}2}$ ratios obtained for this pile configuration and load level are very close to the ones obtained for pile configuration – 1, while the average settlements are slightly higher probably due to the decreased number of piles. Except for the configuration with n=33 and L_p=50m. (the average settlement was underestimated by only 4%), the calculated average settlements for all cases are overestimated by the three – dimensional "Plaxis" software. The average $R_{s\text{-}2}$ ratio was determined as 1.252 indicating an average overestimation of 25% which is exactly equal to the value that was determined for pile configuration – 1.

The ξ_s values obtained by the performed analyses show a similar characteristic with the ones given in the reference study. The deviation of the calculated ξ_s values from the original ones is only -7.6% on average.

- For V_{ult} / P_{eff} = 20 : The calculated average settlements for this load level were slightly less or more than the original values. The error in the calculations did even not reach to a level of 10%. The average R_{s-2} ratio is determined as 0.971. It should be emphasized that, also this value is very close to the one obtained for pile configuration – 1, likewise.

  The calculated ξ_s values are in accordance with the ones given in the reference study both in terms of magnitude and trend. The ξ_s values obtained for the analyses of this study shows a deviation of only -4.5% on average, from the original values.

4.5.4.2.2 Differential Settlements

- For V_{ult} / P_{eff} = 5 : Unfortunately, a logical relationship between the calculated and given differential settlements could not be developed since most of the obtained results were irrelevant with the ones given in the reference study. The $R_{\Delta s-2}$ values varied in a wide range of 0.8 – 2.6 with an average of 1.389. However this average value was not so meaningful due to this wide range.

  Like in the case of the $R_{\Delta s-2}$ ratios, the calculated $\xi_{\Delta s}$ values did not show an accordance with the original ones. The deviation reached to -47.5% on average.
- For V_{ult} / P_{eff} = 20 : As for other pile configurations, the employed software showed a better performance in estimating the differential

settlements for this load level. The calculated differential settlements were all overestimated as compared to the original ones except for the analysis with n=33 and L_p=50m. The average $R_{\Delta s\text{-}2}$ ratio was determined as 1.201 varying in a small range, which enabled a reasonable correlation between the given and the calculated differential settlements.

Although the calculated $\xi_{\Delta s}$ values are in better accordance with the original ones compared to the previous load level, a simple relationship could not have been developed between the calculated and the original values.

4.6 Engineerization of the Results

As it can also be seen from the summary of the results provided above, a general correlation was observed between the solutions of the different softwares for average settlements of the analyzed cases for all pile configurations. However, such a correlation could not be developed between the differential settlements calculated by these different softwares, especially for the higher load level. Correspondingly, since the results obtained for the differential settlements are found to be unreliable, no recommendation is made further for the estimation of the differential settlements of the piled rafts. Because, even small errors in the estimation of the differential settlements may lead to enormous changes in the structural design of the raft. However, all the data obtained about differential settlements from the analyses, are provided in the Appendix D and one can make use of these data easily for practical purposes especially for the lower load level.

As mentioned before, a meaningful correlation was determined between the average settlements of the piled rafts calculated by different softwares. Although the calculated average settlements vary depending upon the softwares, the calculated ξ_s ratios for both the results of the “Plaxis 8.2” and the “Plaxis 3D

Foundation" softwares show very similar characteristics with the ones obtained by the "ABAQUS" analysis. Also, the R_{s-1}, R_{s-2} and R_{s-3} ratios were in a logical sequence of change against the pile length (L_p) and number of piles (n). So, the results obtained for the average settlements were further engineered in order to reach meaningful conclusions. Moreover, as a result of this re-evaluation process, a new method for the estimation of the average settlements is developed and recommended. This method is described in detail in the following section of this chapter.

For comparison, the ξ_s ratios calculated for plane – strain and three – dimensional conditions together with the ones obtained in the reference study are plotted against total pile length $n*L_p$ for each pile configuration and load level separately. All of the results were found to be in very good accordance and it was concluded that the ξ_s ratios obtained from both plane – strain and three – dimensional analysis could be safely used in the estimation of average settlements of the piled rafts. All the related graphs are provided in Appendix A at the end of the thesis.

When the results of the analyses are scrutinized, it is observed that the R_s ratios are dependent on the pile length and number of piles rather than the pile configuration. As a result, the R_s ratios were plotted against number of piles (n) for each pile length regardless of the pile configuration. Moreover, a linear regression curve was fitted to the each set of data. At the end of the task, charts are obtained which can be used in estimating the "true" average settlements (as discussed in the previous chapters, the results of the Abaqus analysis are assumed to be equal to the situation in reality) of the piled rafts by using either two – dimensional or three – dimensional "Plaxis" analysis. One can easily obtain the "true" average settlement of a piled raft by calculating the settlements with the help of the either two – dimensional or three – dimensional "Plaxis" software and dividing the calculated average settlement to the R_s ratio of the corresponding condition. All the graphs for each of the R_{s-1}, R_{s-2} and R_{s-3} ratios are given in Appendix B as ”Kaltakci, V. Design Charts” at the end of the thesis.

4.7 The Recommended Method : Kaltakci, V. Method

This method is based on the accordance of the ξ_s ratios which were obtained from the solutions of different softwares. Since the calculated ξ_s ratios of different softwares are so close to each other, one can make use of these data easily as follows:

In order to use this method, first of all one should create his/her own ξ_s vs. $n*L_p$ relationship by either two – dimensional or thee – dimensional “Plaxis” for different alternatives of piled rafts if the thought alternatives are not similar to the ones provided in this study. Then the average settlement of the unpiled raft under the concerned loading should be computed. This computation may either be by the “ABAQUS” software or by the simple hand calculations. Then the average settlement of the concerned piled raft can easily be obtained by multiplying the average settlement of the unpiled raft with the ξ_s ratio of the corresponding case. An example procedure is provided below:

For pile configuration – 1 and L_p=50m. with n=169 piles, the ξ_s ratios are determined as 0.158 and 0.140 for the solutions of “Plaxis 8.2” and “Plaxis 3D Foundation” programs respectively at a load level of V_{ult} / P_{eff} = 5. When the calculated ξ_s ratios are multiplied by the average settlement of the unpiled raft (the average settlement of the unpiled raft used for this example is calculated by the “ABAQUS” software) the results are :

$$S_{avg} = S_{avg,R} * \xi_s \tag{4.10}$$

For $S_{avg,R}$ = 217.647 mm. (Result of the Abaqus Solution) ➔

S_{avg} of the investigated case by 2D Plaxis: S_{avg} = 217.647*0.158 = 34.388 mm.

S_{avg} of the investigated case by 3D Plaxis: S_{avg} = 217.647*0.140 = 30.470 mm.

The average settlement of the concerned case calculated by the "ABAQUS" software equals to 37 mm. and as it can be seen easily, the difference in the results obtained by using the "Plaxis 8.2" and "Plaxis 3D Foundation" softwares is only a few millimeters when compared with the original "ABAQUS" solution. A comparative graph for Pile configuration – 1 and n=169 piles at a load level of V_{ult} / P_{eff} = 5 is provided below as Fig 4.21. Similar graphs are plotted for all analyzed cases and given in Appendix C of the thesis.

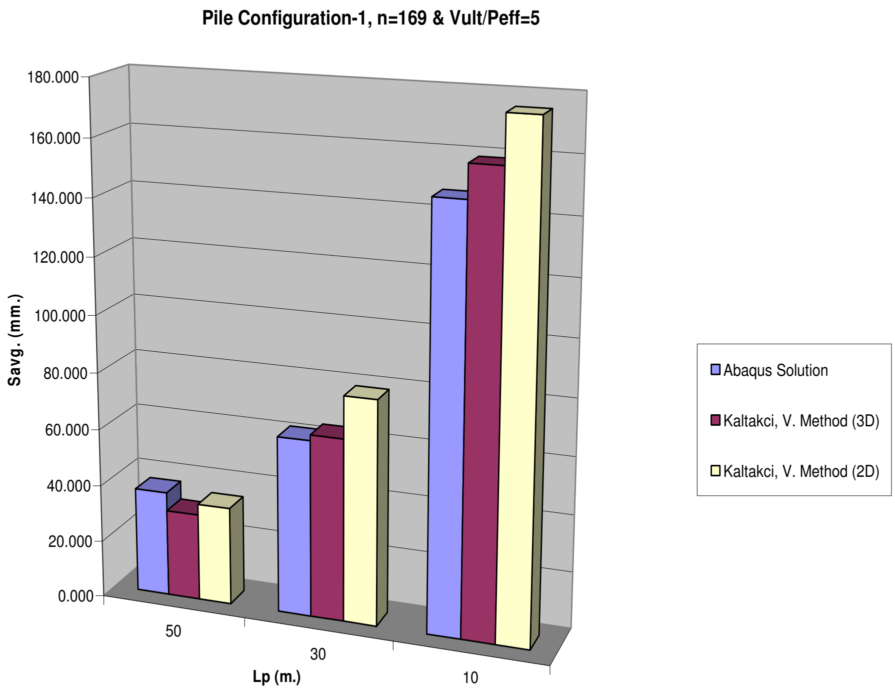


Fig 4.21 Comparison of the Solutions

4.8 Kaltakci, V. Design Charts

These charts are developed based on the fact that the calculated average settlements by 2-D Plaxis, 3-D Plaxis and ABAQUS programs can be converted to each other simply by using conversion factors named as "R_{s-1}, R_{s-2} and R_{s-3}". By the provided charts, one can calculate the average settlement of a piled raft by directly determining the average settlement of the investigated case by either two – dimensional or three – dimensional "Plaxis" and then dividing the calculated settlement to the suitable R_s ratio. An example solution is provided below:

For the same case discussed above the average settlement calculated by the "Plaxis 8.2" and "Plaxis 3D Foundation" are equal to 75.537 mm. and 41.382 mm. respectively. The recommended R_s ratios are obtained from the related charts given in Appendix B. Also a sample R_{s-1} vs. n graph is provided below as Fig 4.22.

To be used for 2D Solutions: $R_{s-1} \approx 2.1$ for n=169 & L_p=50m.

To be used for 3D Solutions: $R_{s-2} \approx 1.1$ for n=169 & L_p=50m.

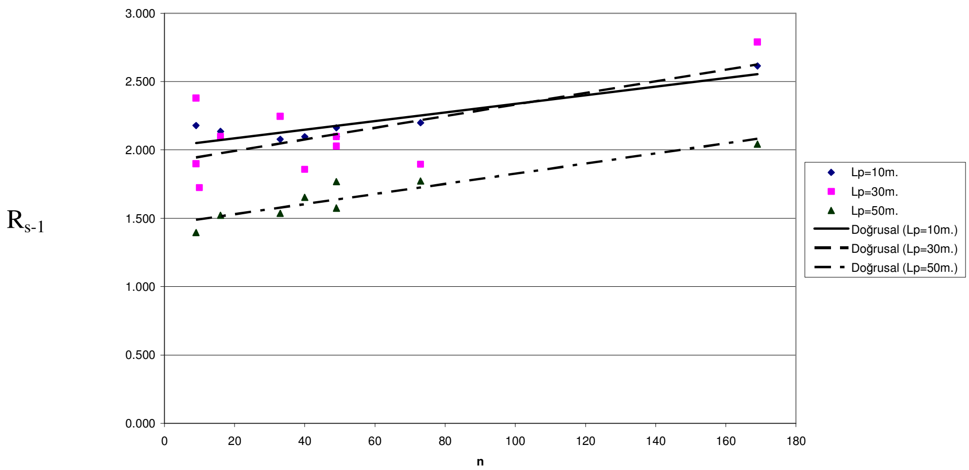


Fig 4.22 R_{s-1} vs. n for $V_{ult} / P_{eff} = 5$

Then the “true” average settlement of the investigated case can be calculated from equations 4.11 & 4.12 as follows:

$$S_{avg} = \frac{S_{avg,2D}}{R_{s-1}} \tag{4.11}$$

$$S_{avg} = \frac{S_{avg,3D(Plaxis)}}{R_{s-2}} \tag{4.12}$$

The average settlement of the investigated case by plane – strain analysis:

S_{avg} = 75.537 / 2.1 = 35.97 mm.

The average settlement of the investigated case by three – dimensional analysis:

S_{avg} = 41.382 / 1.1 = 37.62 mm.

Since the result of the “ABAQUS” solution for this case is equal to 37 mm., it is seen that, estimated value by use of “Kaltakci, V. Design Charts” is very close to the original values given in the reference study. The difference for this case is only a few millimeters.

CHAPTER 5

CONCLUSIONS

5.1 Summary

Both two – dimensional and three – dimensional finite element analyses of piled rafts were carried out throughout this study by Plaxis software, in order to compare the results of these analyses with the ones provided in a former study by Reul & Randolph (2004), the analyses of which were performed by three – dimensional finite element software, ABAQUS. Mainly, three basic pile configurations were investigated for three different pile lengths, two different pile spacings and two different load levels. The total number of piles varied from n=9 to n=169. The total load is assumed to be applied uniformly on the whole raft area in all analyses. Also, the applied load levels were arranged to be equal to either 20% or 5% of the ultimate capacity of an equivalent unpiled raft. Moreover, Mohr – Coulomb failure criterion under drained conditions was used to model the soil behavior instead of the elasto – plastic model which was employed in the analyses of the reference study.

As mentioned before, the piles were uniformly distributed on the whole raft area in "Pile Configuration – 1" while they were located only in the central region of the raft in "Pile Configuration – 2". On the other hand, the piles were located at the edges of the raft as well as in the central region in Pile Configuration – 3".

A new method for the estimation of the average settlements of the piled rafts is developed and recommended as "Kaltakci, V. Method" within the thesis. Also, the average settlements of piled rafts calculated by different softwares were suggested to be corrected by certain factors for an estimation of the "true" average settlements by making use of "Kaltakci, V. Design Charts".

5.2 Conclusions

5.2.1 General Conclusions

- Mohr – Coulomb failure criterion under drained conditions can be used instead of an advanced elasto – plastic cap model, for modeling the behavior of overconsolidated clays in finite element analysis. The average settlements estimated by the elastic model shows accordance with the results of the elasto – plastic analyses at least in terms of trend. However, the differential settlements estimated by elastic analyses do not have good correlation with those of the elasto – plastic analyses especially for the higher load level. This is due to the fact that, the elastic soil model is blind of the plastication under the raft and still behaves in elastic manner even for the loads at which the soil moves to plastic state in reality. As a result the differential settlements are overestimated in general.

- The decrease in the total settlements of the piled rafts is much more dependent on the length of piles rather than their number. To illustrate, for L_p=50m. in pile configuration – 1, the average settlement of the system with n=49 piles is only 1.9cm larger than that of the one with n=169 piles. Whereas, for the same configuration with n=169 piles, the average settlement nearly doubles when the length of piles are decreased to L_p=30m.

- Pile Configuration – 1 is determined to be the most effective way of decreasing the average settlements of the piled rafts. The minimum average settlements are obtained for this pile configuration for the same total pile length when compared with the other configurations.

- Pile Configuration – 2 is an effective way of preventing excessive differential settlements. However, even for the same number and length of piles, the average settlements obtained by this type of configuration are significantly larger when compared with pile configuration – 1. Moreover, "hogging" phenomenon starts to occur with decreasing number of piles and increasing load level.

- Among the three configurations investigated, pile configuration – 3 seemed to be the most logical configuration to be applied in daily practice since it shows a similar performance to pile configuration – 1 in decreasing the total settlements while preventing the hogging of the raft even at high load levels. Moreover, the total number of piles used in this configuration is significantly smaller than number of the piles used for pile configuration – 1 for the same settlement level.

5.2.2 Conclusions for Two – Dimensional Analysis

- The two – dimensional "Plaxis 8.2" software can be safely used in estimating the average settlements of the piled rafts provided that the recommended method of this thesis is employed throughout the analyses.

- The average settlements obtained by two – dimensional analyses were higher than the values obtained by both the "ABAQUS" and "Plaxis 3D Foundation" analyses.

- The ratio between the calculated average settlements of the "Plaxis 8.2 – ABAQUS" and "Plaxis 8.2 – Plaxis 3D Foundation" softwares (R_{s-1} and R_{s-3} respectively) were highly dependent on the applied load level and the length of the piles. The average R_{s-1} and R_{s-3} values for the two load levels and three different pile lengths are given in Table 5.1 below. The values given in Table 5.1 are to be considered as approximate factors. However, these values provide a guidance for a first estimation of the average settlements. The "Kaltakci, V. Design Charts" given in Appendix B of this thesis is recommended for a better estimation of the R_s ratios. Also, it should be noted that the R_{s-3} ratio compares the results of "Plaxis 8.2" and "Plaxis 3D Foundation" analyses and the use of R_{s-3} ratio should be accompanied by the use of the R_{s-2} ratio in order to reach the "ABAQUS" results which are assumed to be the "true" solution.

- The differential settlements calculated with plane – strain analyses did not show a similar characteristics with the ones calculated in the reference study especially for the higher load level. Moreover, opposite to the situation in average settlements, the decrease in the differential settlements with the addition of piles ($\xi_{\Delta s}$), were not in accordance with the cited study. So, no reasonable correlation could be made for the estimation of the differential settlements.

5.2.3 Conclusions for Three – Dimensional Analysis

- The three – dimensional "Plaxis 3D Foundation" software can be safely used in estimating the average settlements of the piled rafts provided that the recommended method presented in this study is employed throughout the analyses.

- The average settlements obtained by three – dimensional Plaxis analyses were generally higher than the values obtained by the “ABAQUS” analyses, nevertheless they were much closer to the “ABAQUS” results, as compared to the plane – strain analyses.

- The R_{s-2} ratios can be directly used in the estimation of the average settlements of the piled rafts. The average R_{s-2} ratios for two different load levels and three different pile lengths are provided below in Table 5.1. These average values are provided just to give a first idea about the magnitude of the settlements of a piled raft system and not recommended for design purposes. The R_{s-2} ratios are recommended to be determined from the provided “Kaltakci, V. Design Charts”.

- The differential settlements calculated by the three – dimensional “Plaxis” analysis were in much better agreement with those of the reference study, as compared to the two – dimensional approach. However, significant deviations between the differential settlements calculated by these different softwares made any meaningful correlation unavailable for the estimation of the differential settlements.

Table 5.1 Average R_s Ratios

	V_{ult}/P_{eff} =5			V_{ult}/P_{eff} =20		
L_p (m.)	R_{s-1}	R_{s-2}	R_{s-3}	R_{s-1}	R_{s-2}	R_{s-3}
10	2.20	1.52	1.45	1.44	1.03	1.39
30	2.10	1.41	1.52	1.44	0.97	1.48
50	1.66	1.07	1.56	1.31	0.85	1.56

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APPENDIX A

ξ_s vs. $n*L_p$ Graphs

Table A.1

Pile Configuration	s/d_p	n	V_{ult}/P_{eff}	Page
Pile Configuration - 1	3	169&49	5	127
Pile Configuration - 1	3	169&49	20	128
Pile Configuration - 2	3	49	5	129
Pile Configuration - 2	3	49	20	130
Pile Configuration - 2	6	16	5	131
Pile Configuration - 2	6	16	20	132
Pile Configuration - 2	6	9	5	133
Pile Configuration - 2	6	9	20	134
Pile Configuration - 3	3	73	5	135
Pile Configuration - 3	3	73	20	136
Pile Configuration - 3	6	40	5	137
Pile Configuration - 3	6	40	20	138
Pile Configuration - 3	6	33	5	139
Pile Configuration - 3	6	33	20	140

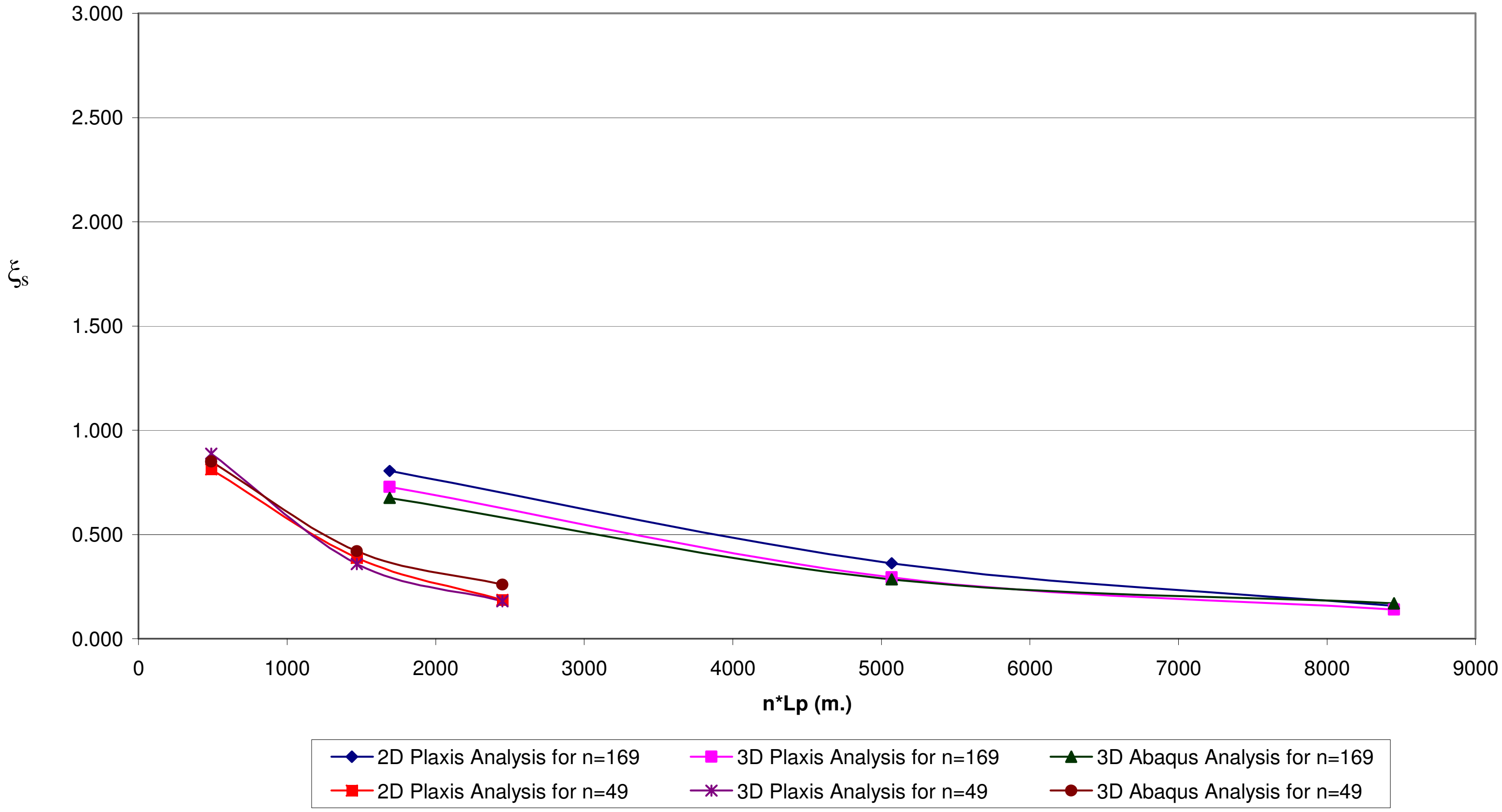

Fig A.1 ξ_s vs. n^*L_p for Pile Configuration – 1 & V_{ult} / P_{eff} = 5

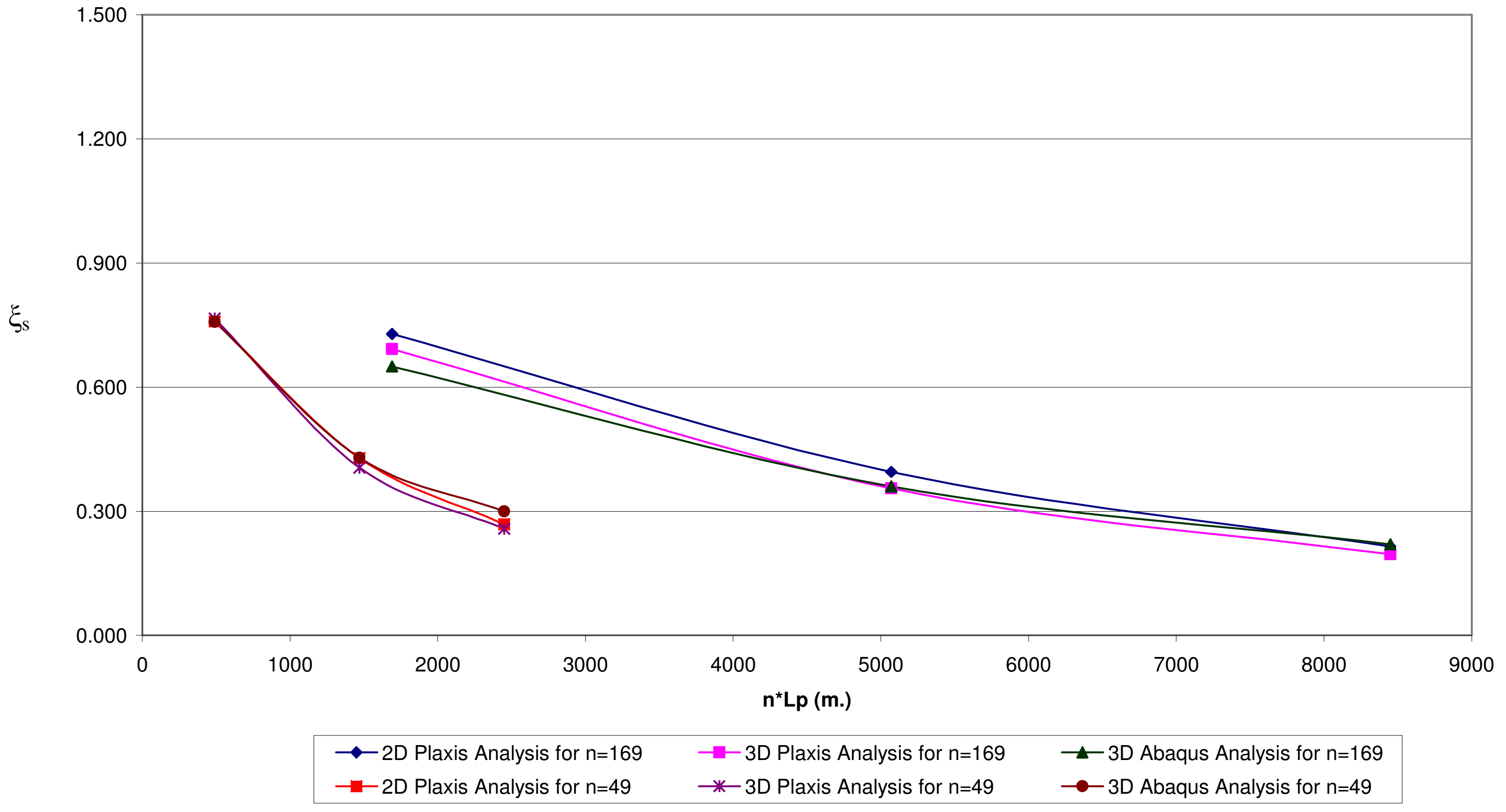

Fig A.2 ξ_s vs. n^*L_p for Pile Configuration – 1 & V_{ult} / P_{eff} = 20

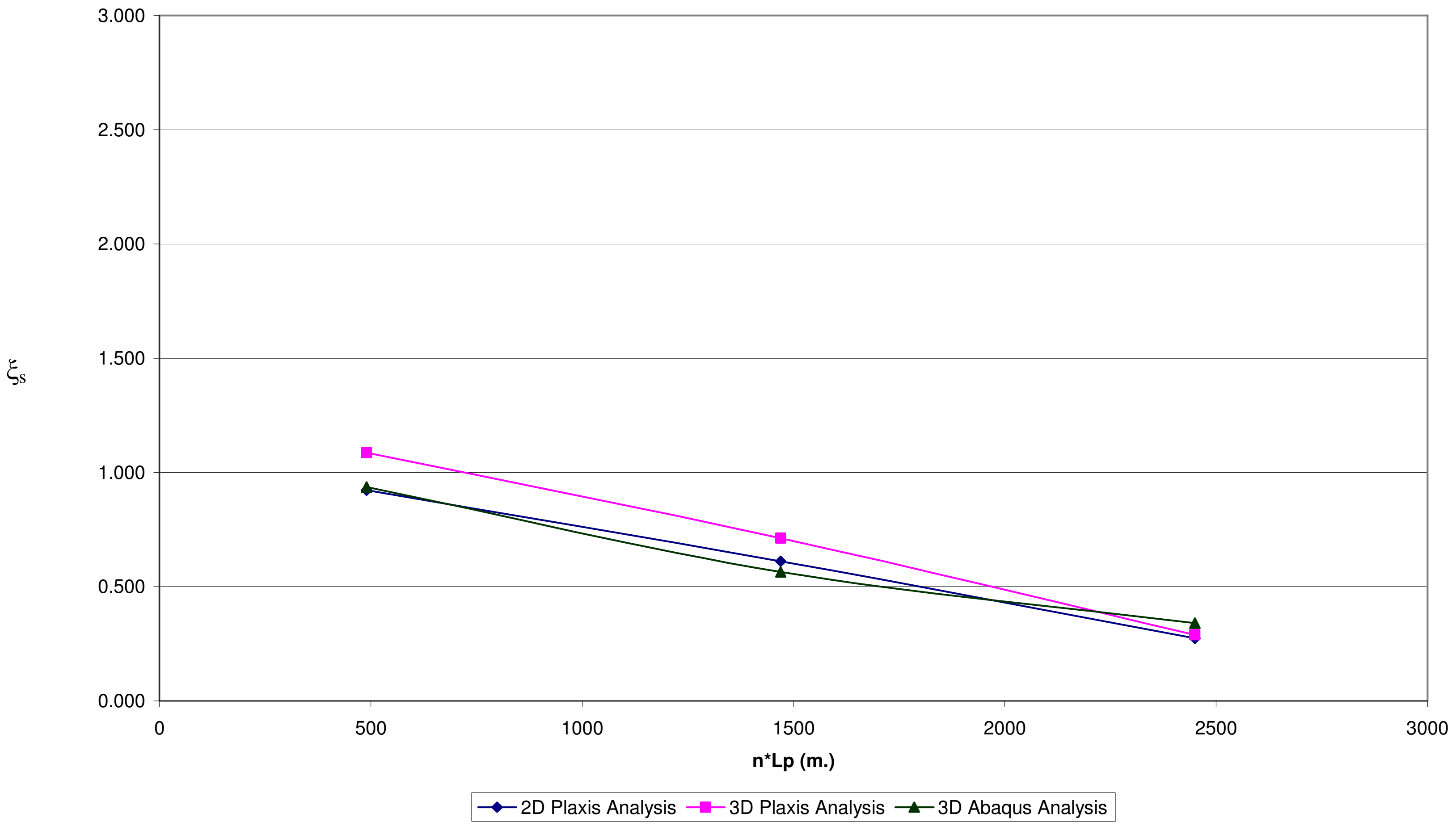

Fig A.3 ξ_s vs. $n*L_p$ for Pile Configuration – 2, n=49 & V_{ult} / P_{eff} = 5

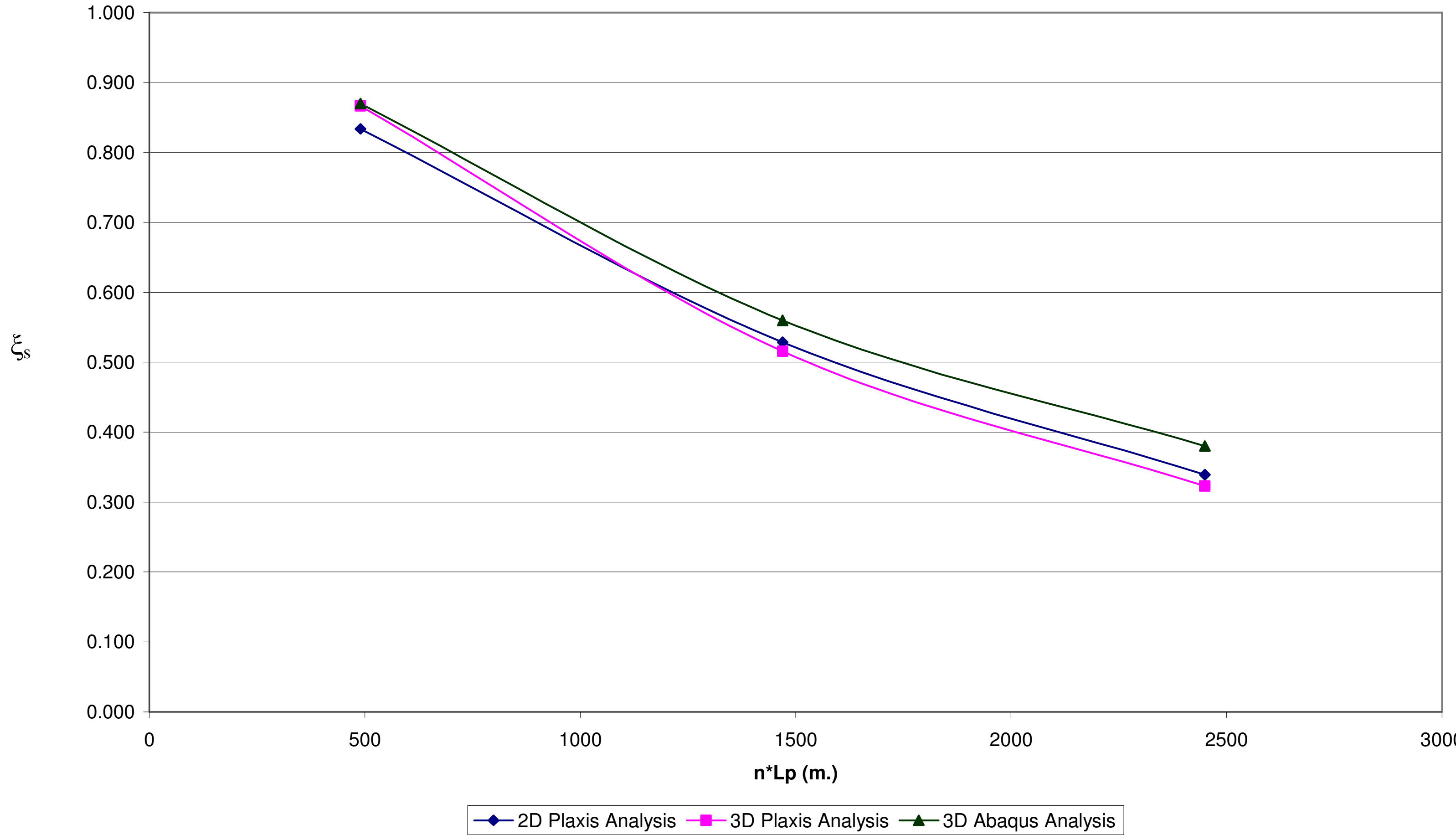

Fig A.4 ξ_s vs. n^*L_p for Pile Configuration – 2, n=49 & V_{ult} / P_{eff} = 20

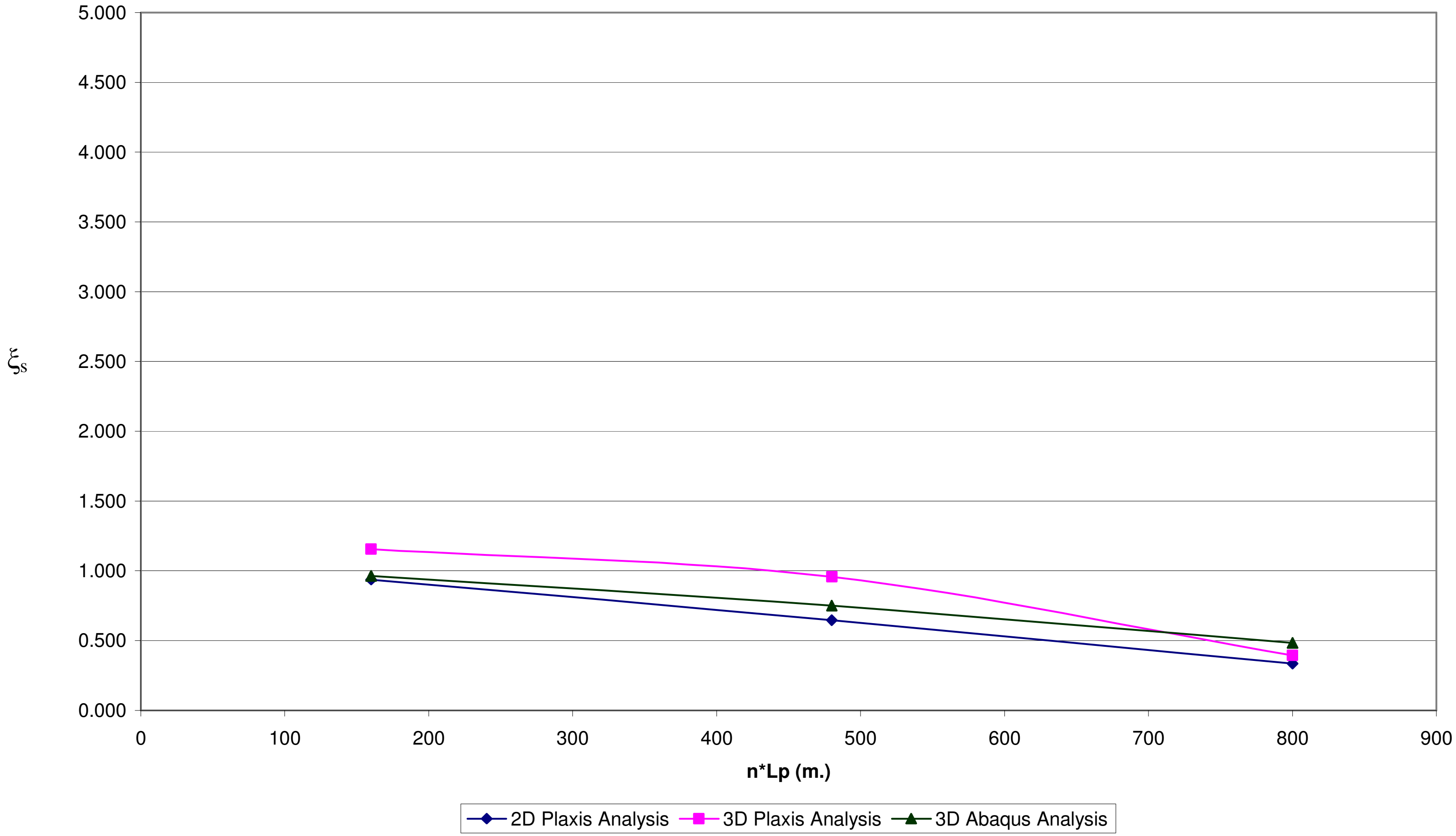

Fig A.5 ξ_s vs. n^*L_p for Pile Configuration – 2, n=16 & V_{ult} / P_{eff} = 5

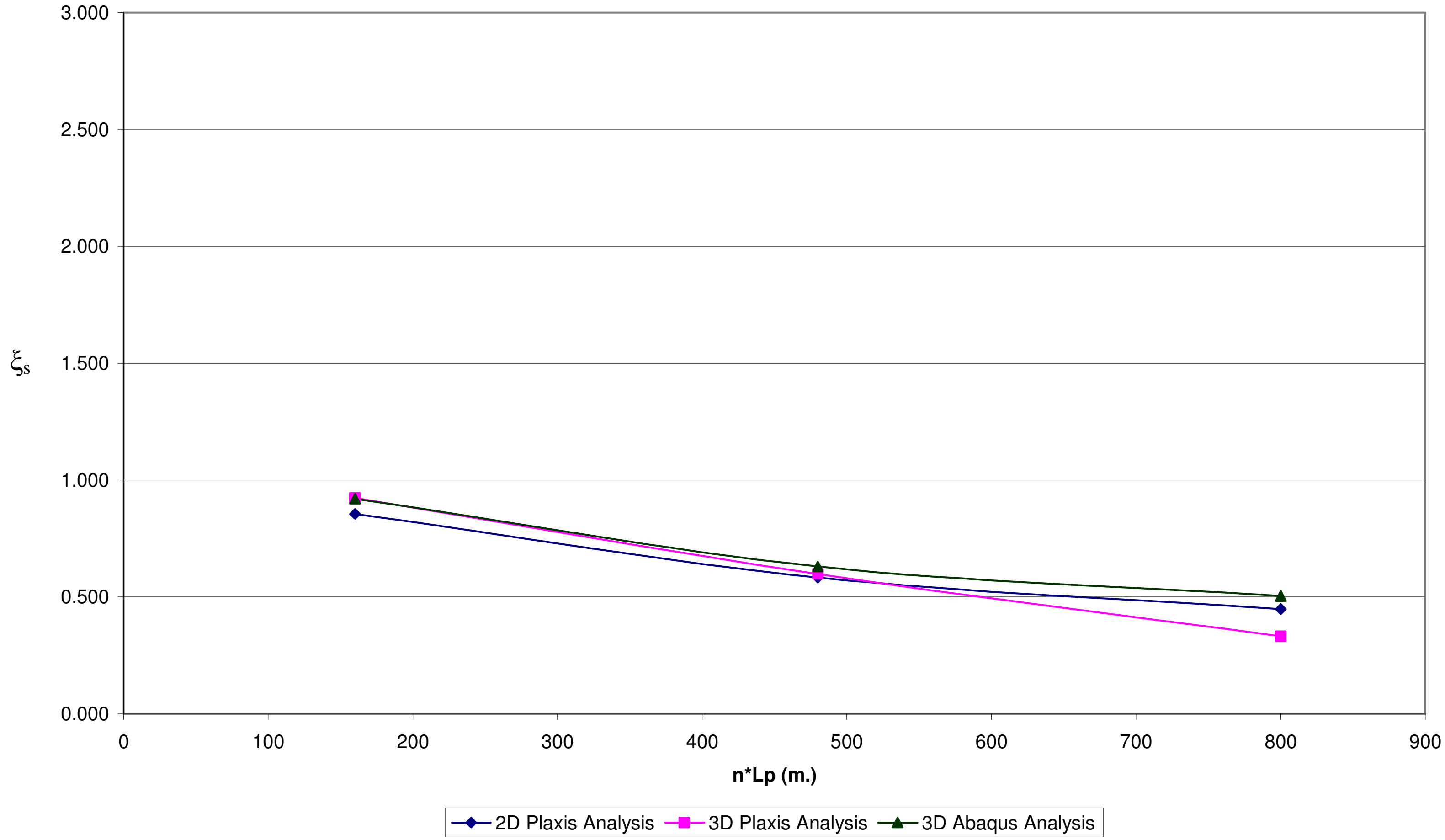

Fig A.6 ξ_s vs. n*L_p for Pile Configuration – 2, n=16 & V_{ult} / P_{eff} = 20

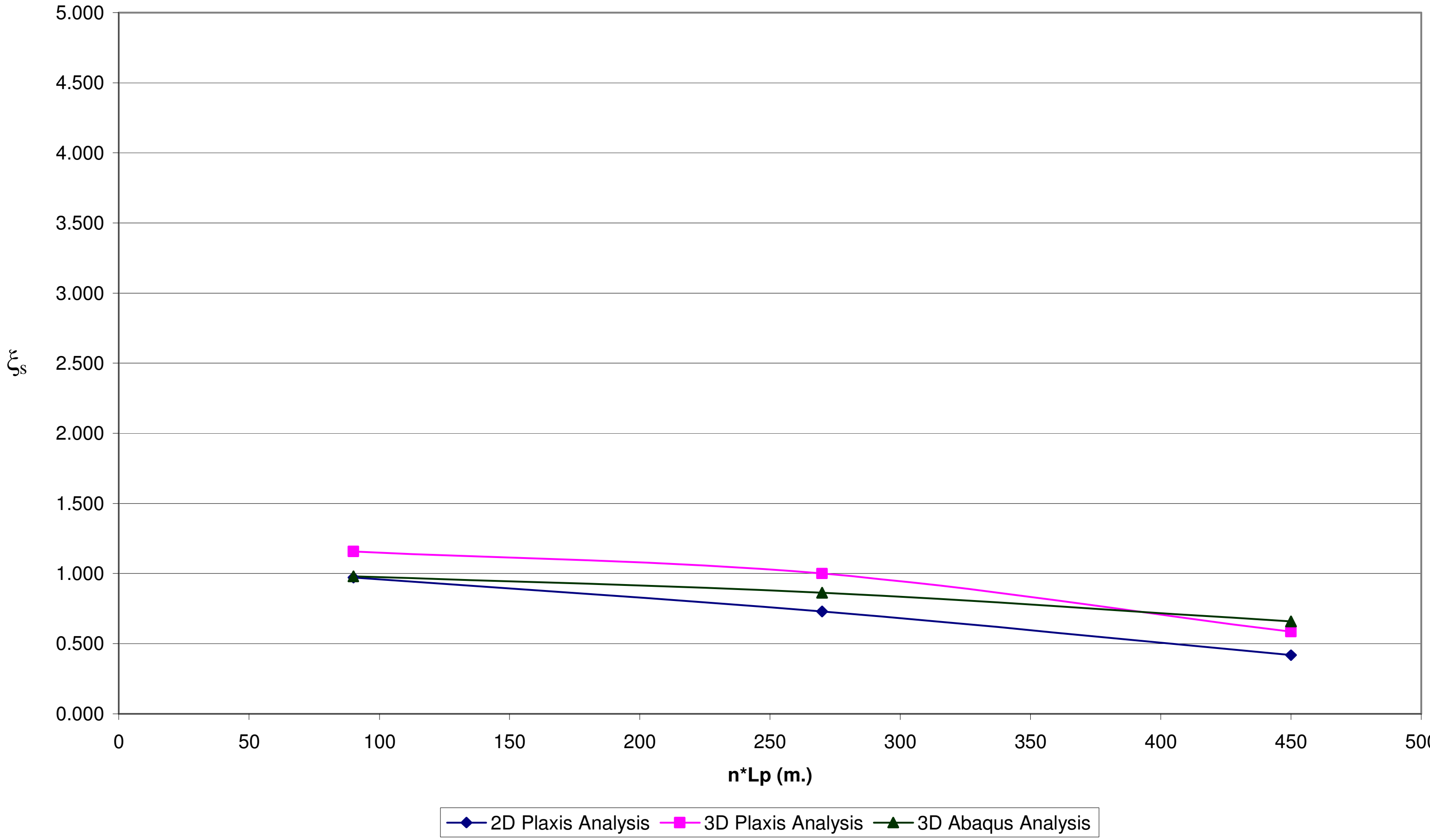

Fig A.7 ξ_s vs. n^*L_p for Pile Configuration – 2, n=9 & V_{ult} / P_{eff} = 5

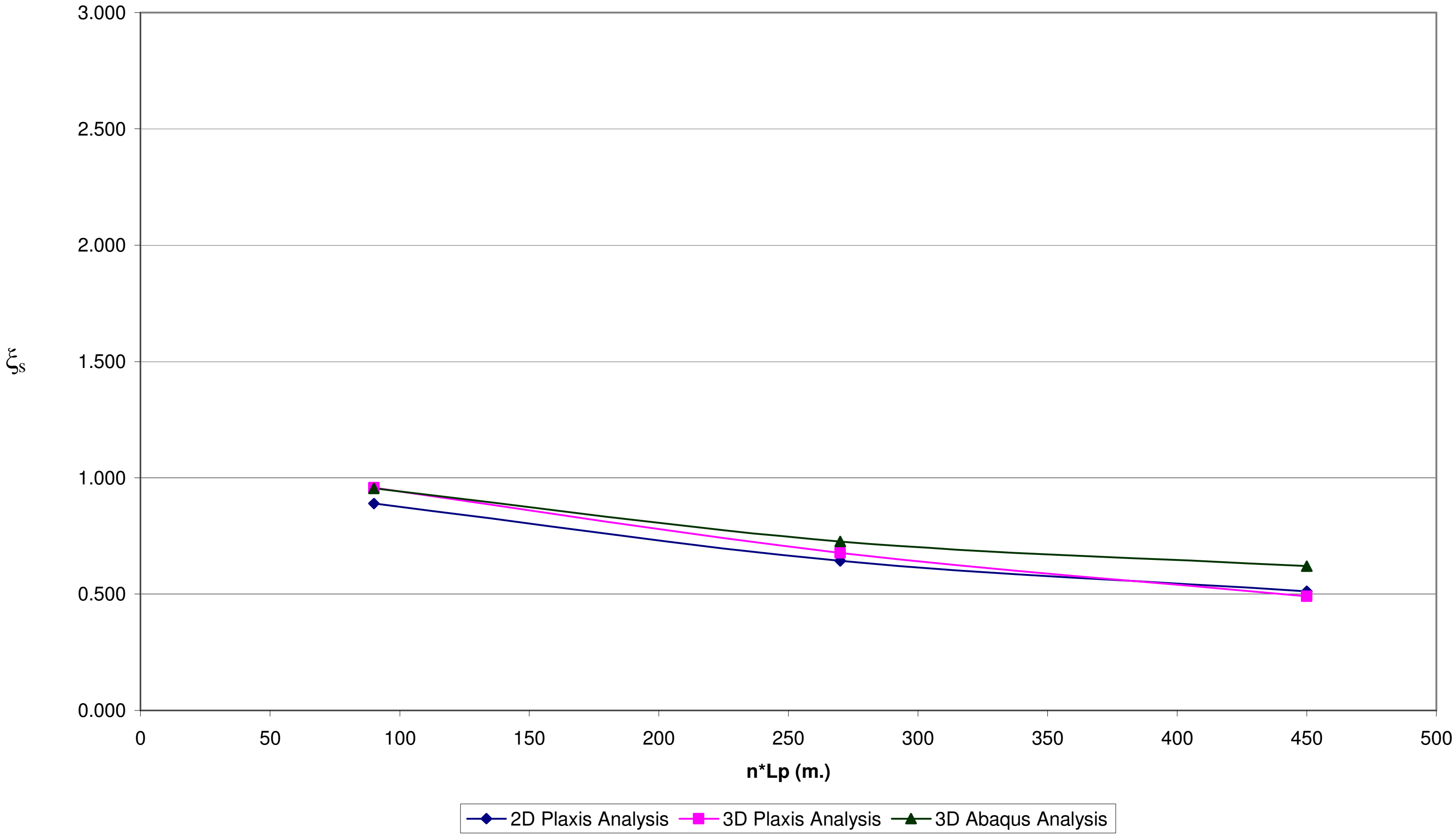

Fig A.8 ξ_s vs. n*L_p for Pile Configuration – 2, n=9 & V_{ult} / P_{eff} = 20

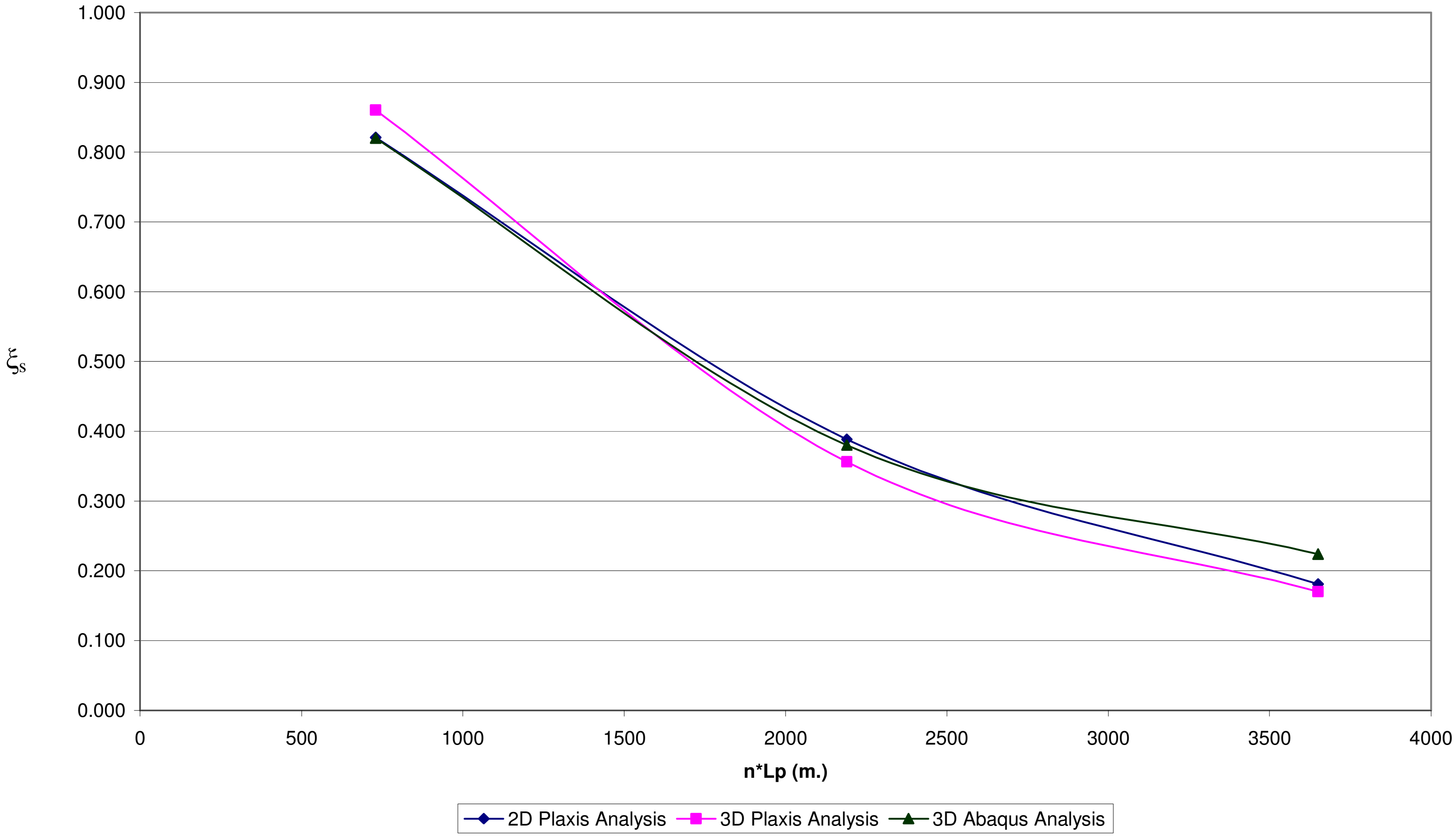

Fig A.9 ξ_s vs. $n*L_p$ for Pile Configuration – 3, n=73 & V_{ult} / P_{eff} = 5

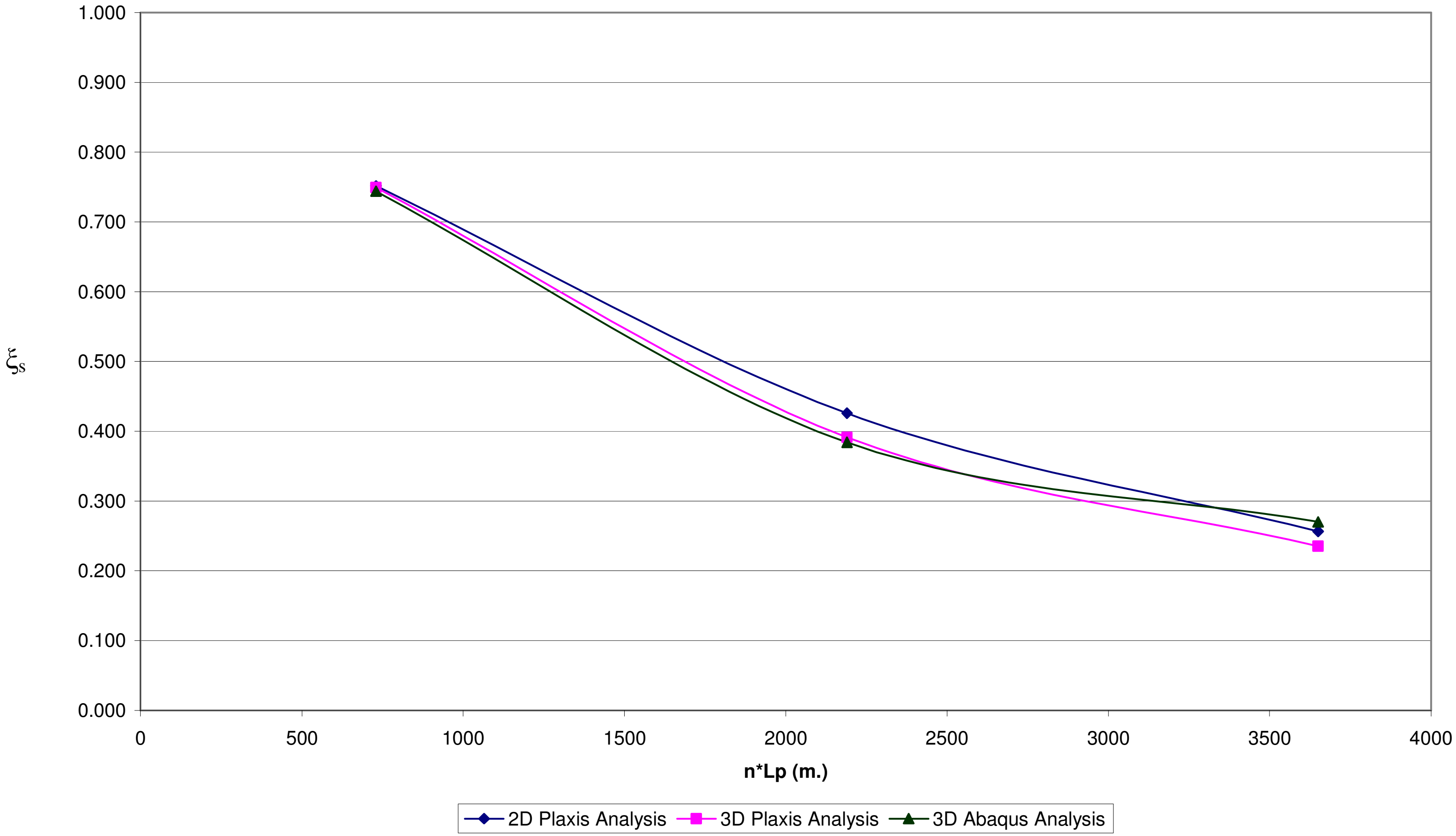

Fig A.10 ξ_s vs. n*L_p for Pile Configuration – 3, n=73 & V_{ult} / P_{eff} = 20

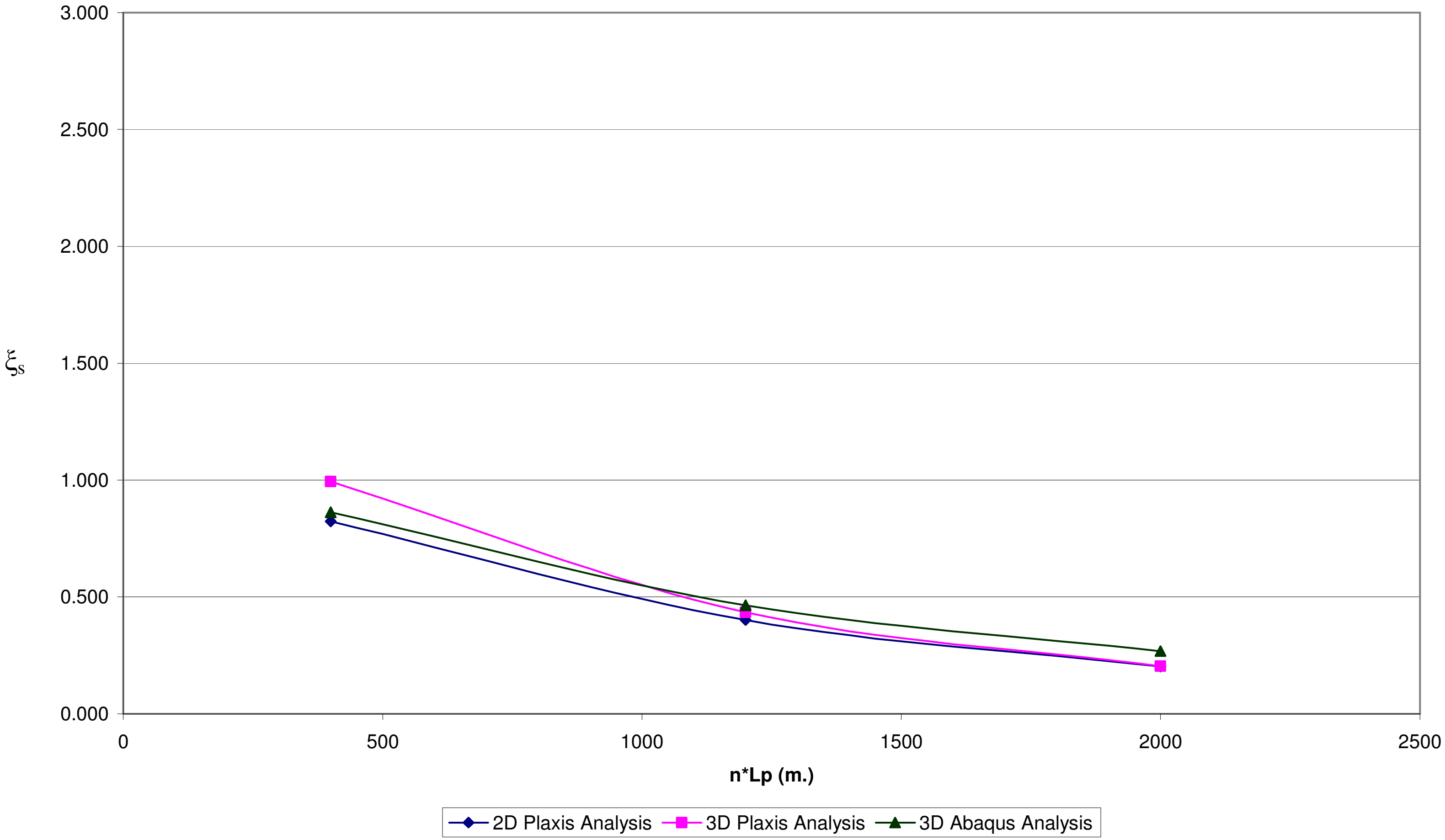

Fig A.11 ξ_s vs. n*L_p for Pile Configuration – 3, n=40 & V_{ult} / P_{eff} = 5

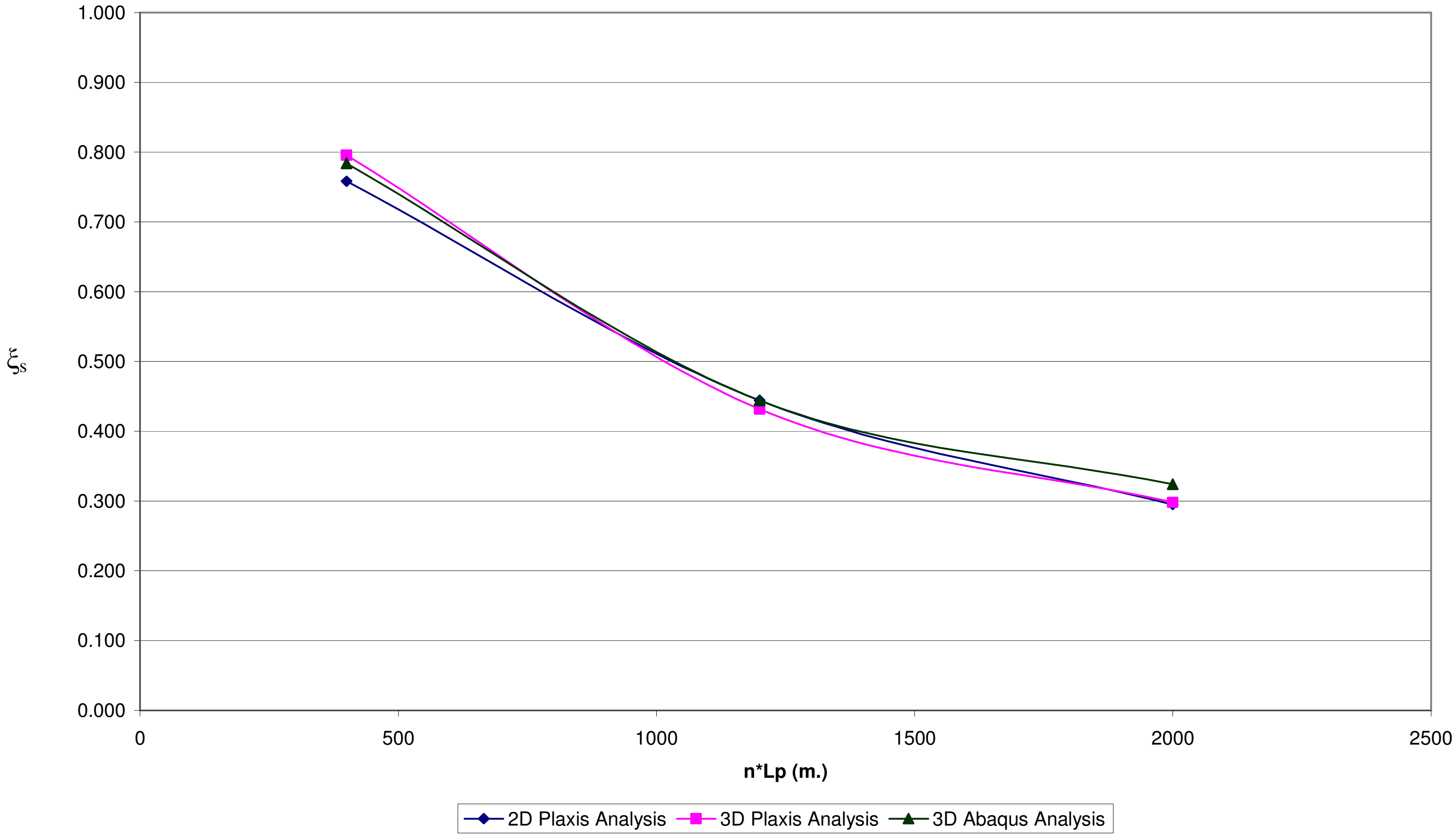

Fig A.12 ξ_s vs. n*L_p for Pile Configuration – 3, n=40 & V_{ult} / P_{eff} = 20

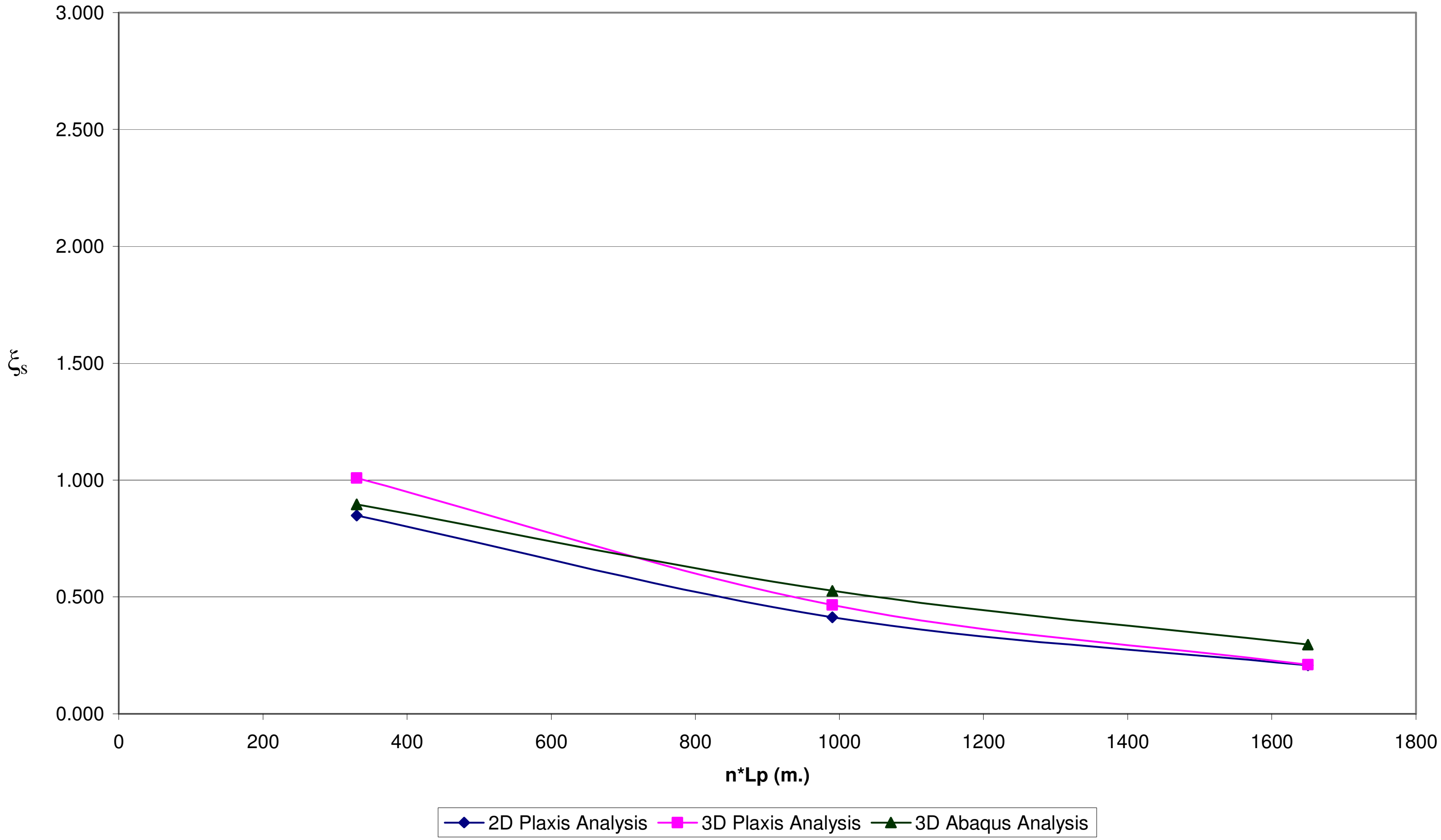

Fig A.13 ξ_s vs. n*L_p for Pile Configuration – 3, n=33 & V_{ult} / P_{eff} = 5

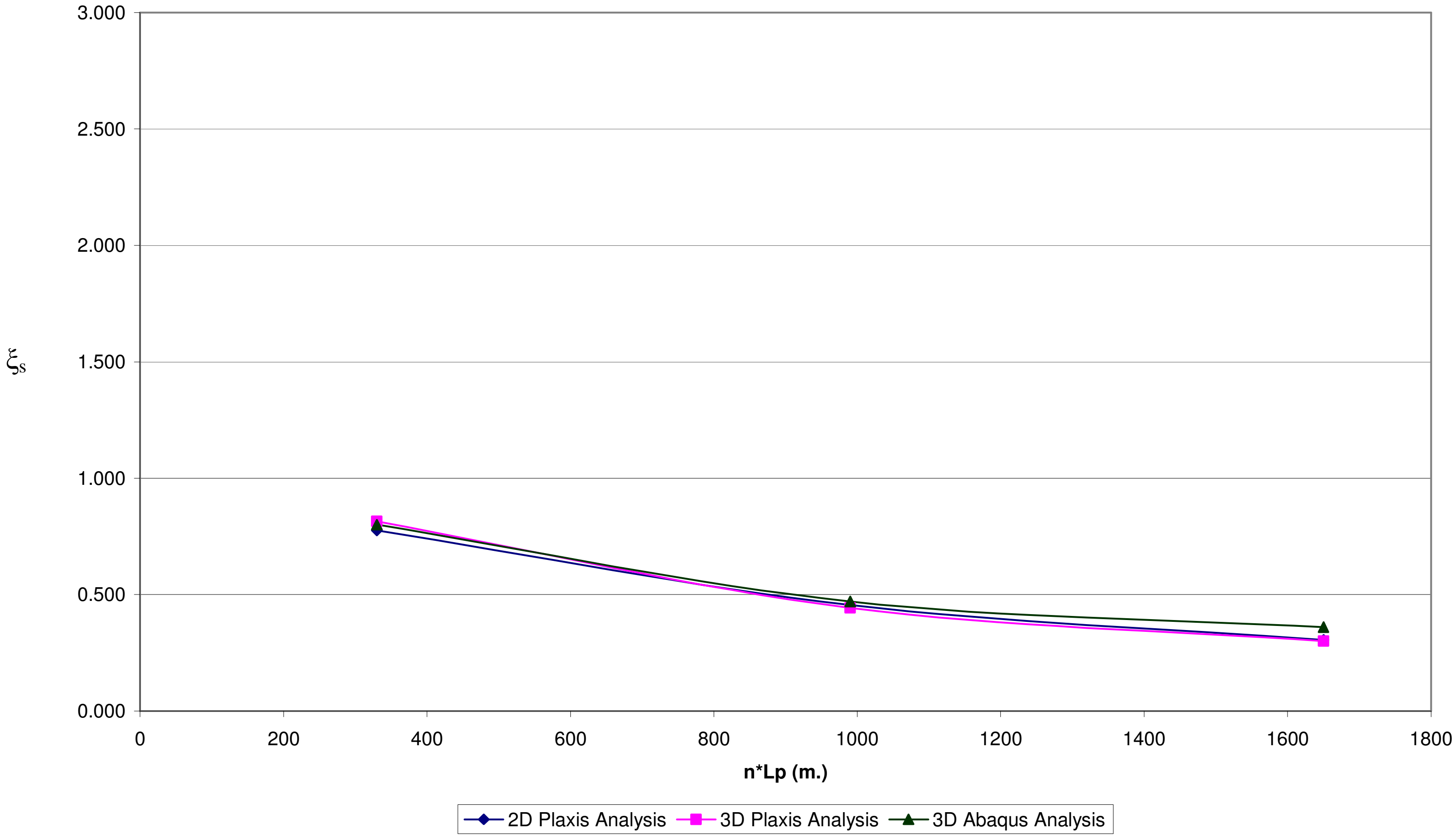

Fig A.14 ξ_s vs. $n*L_p$ for Pile Configuration – 3, n=33 & V_{ult} / P_{eff} = 20

APPENDIX B

"Kaltakci, V. Design Charts"

Table B.1

Ratio	V_{ult}/P_{eff}	Page
R_{s-1}	5	142
R_{s-1}	20	143
R_{s-2}	5	144
R_{s-2}	20	145
R_{s-3}	5	146
R_{s-3}	20	147

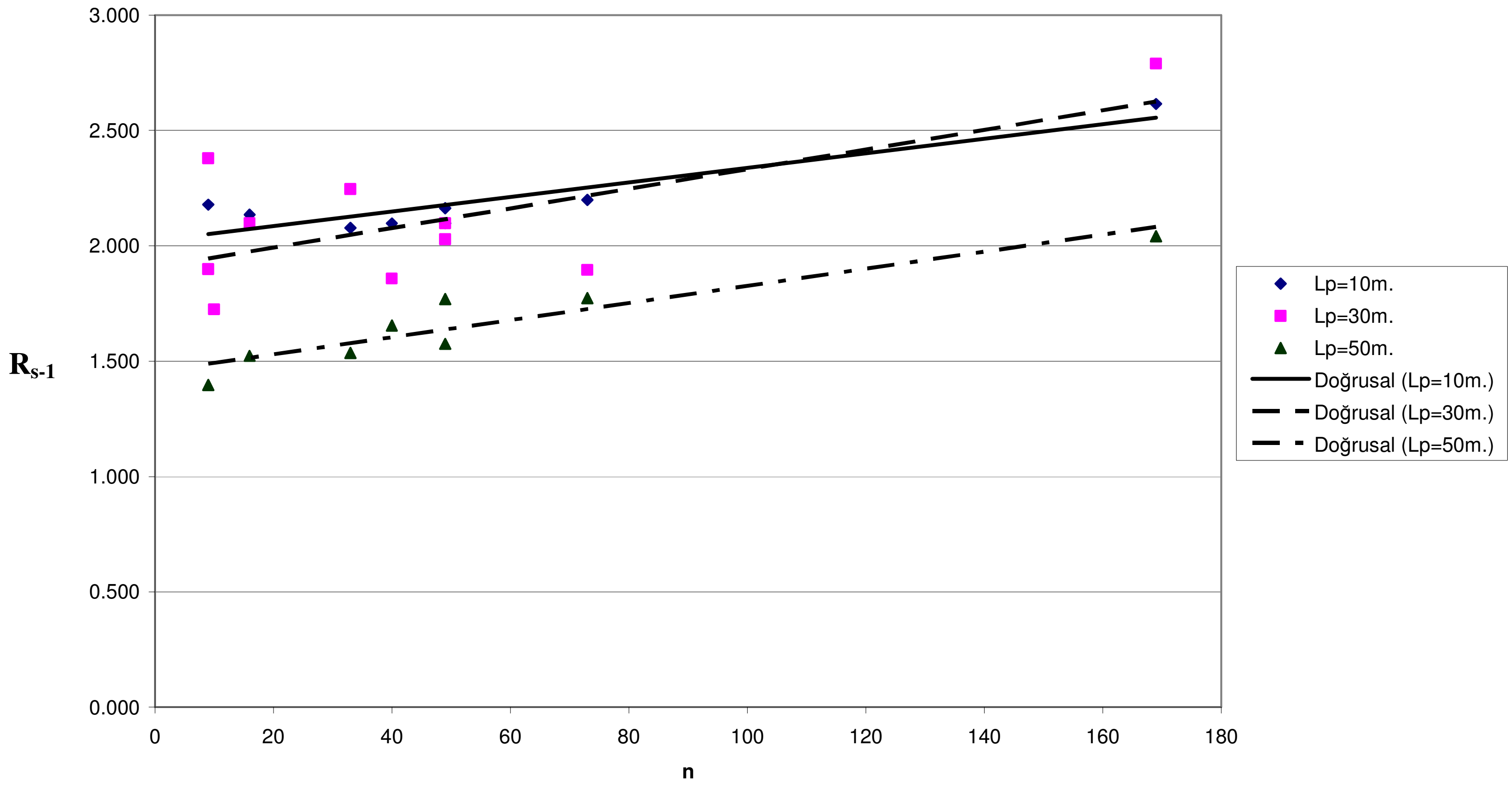

Fig B.1 R_{s-1} for $V_{ult}/P_{eff} = 5$

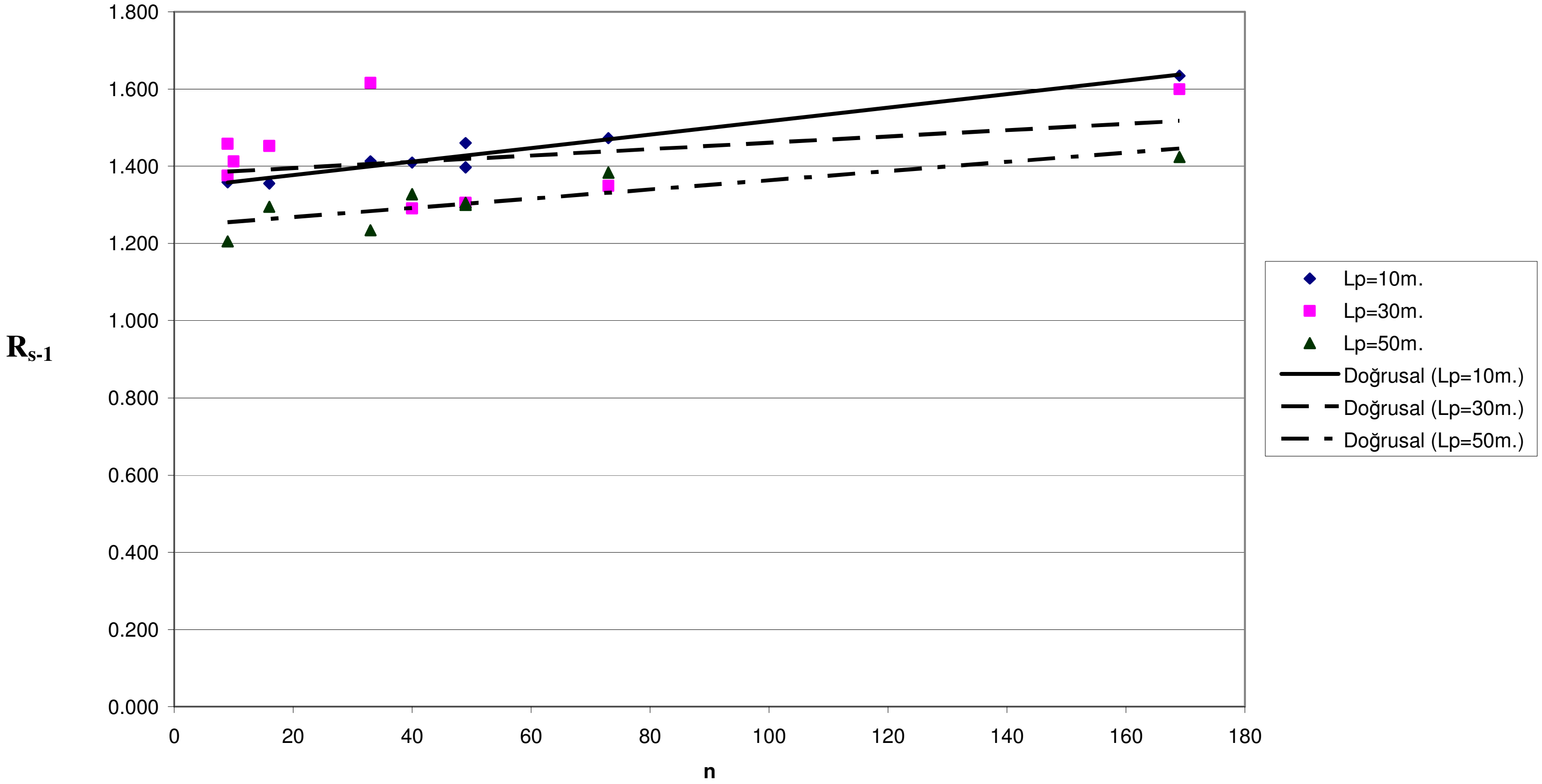

Fig B.2 R_{s-1} for V_{ult}/P_{eff} = 20

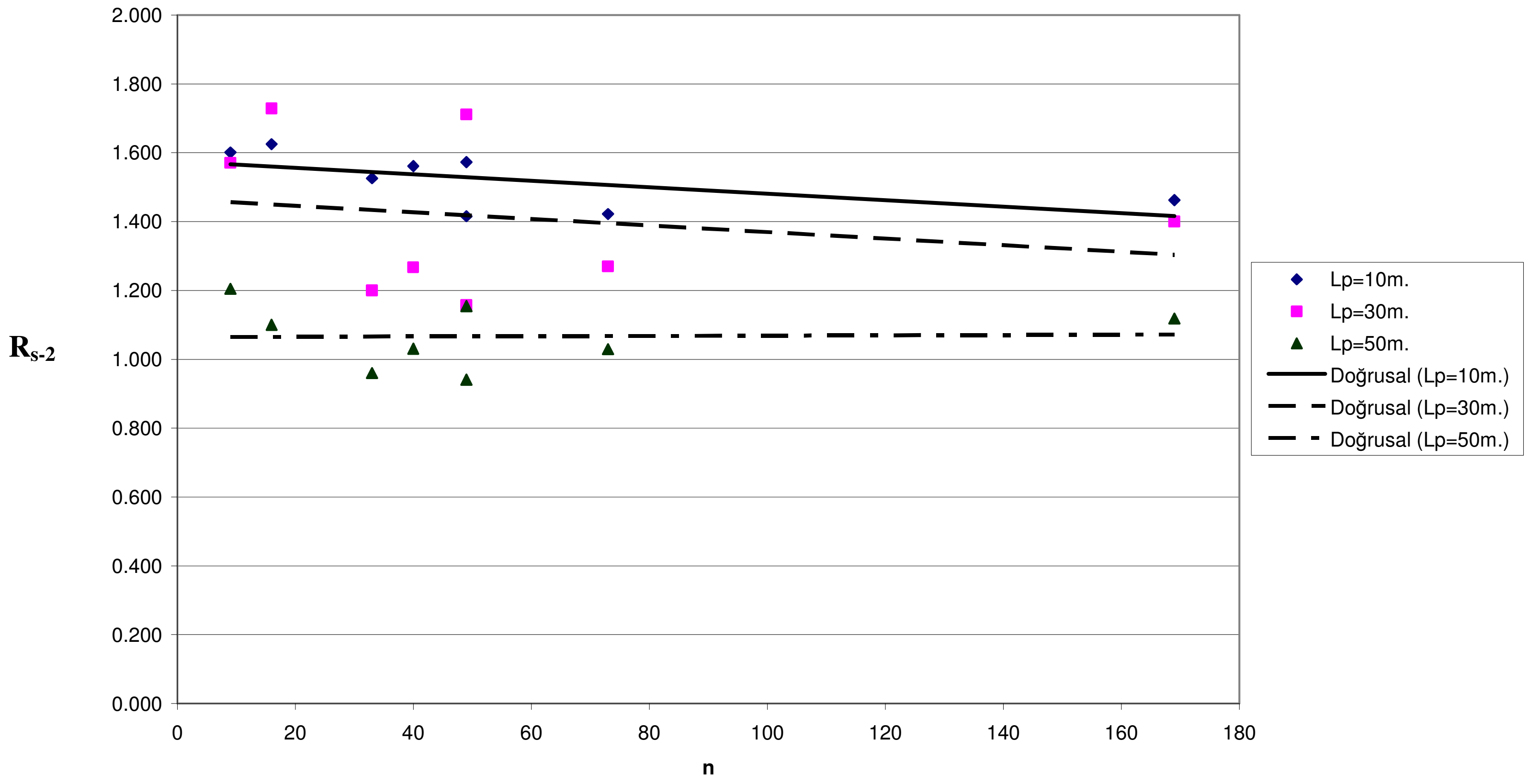

Fig B.3 R_{s-2} for V_{ult}/P_{eff} = 5

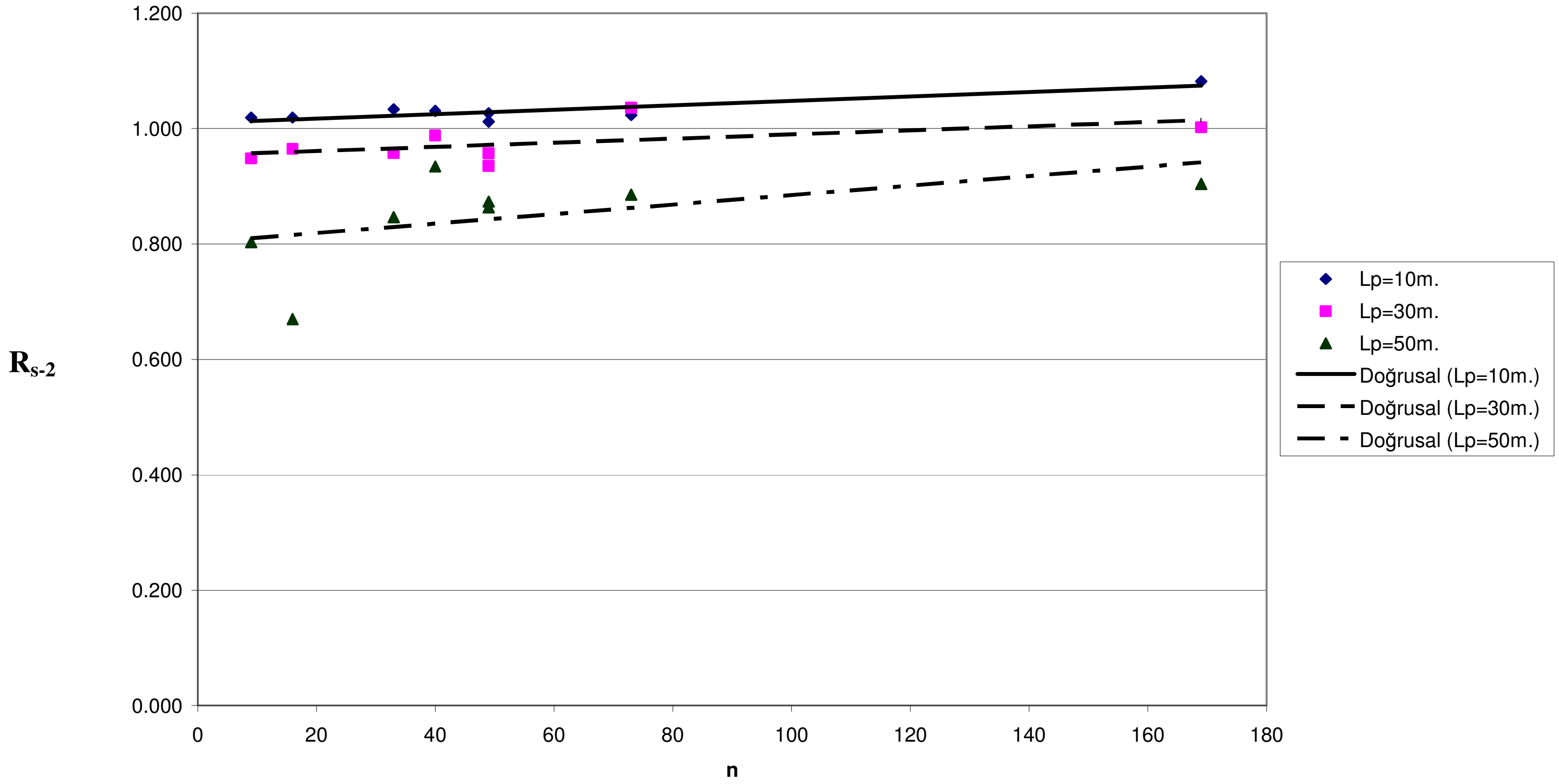

Fig B.4 R_{s-2} for V_{ult}/P_{eff} = 20

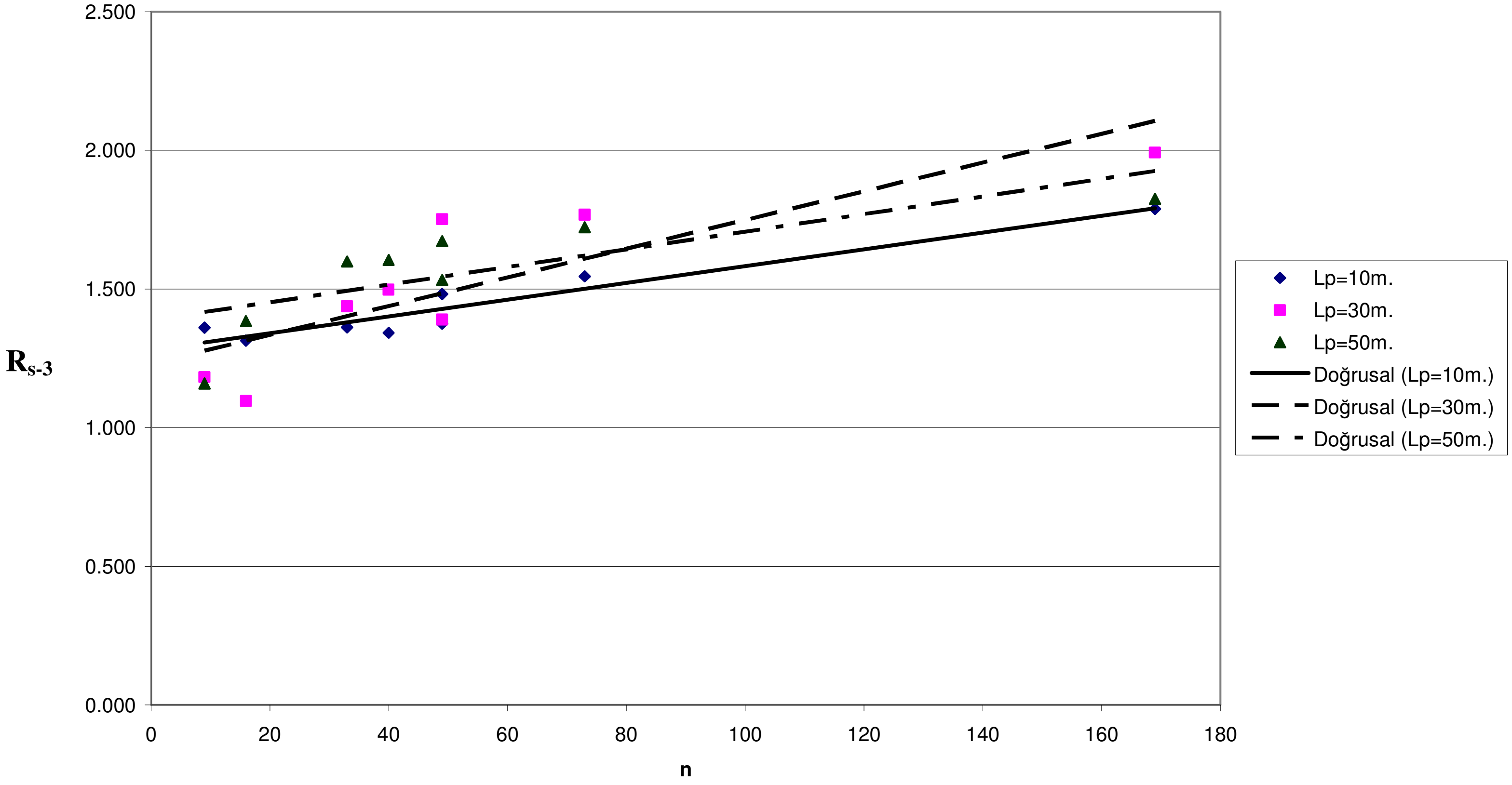

Fig B..5 R_{s-3} for V_{ult}/P_{eff} = 5

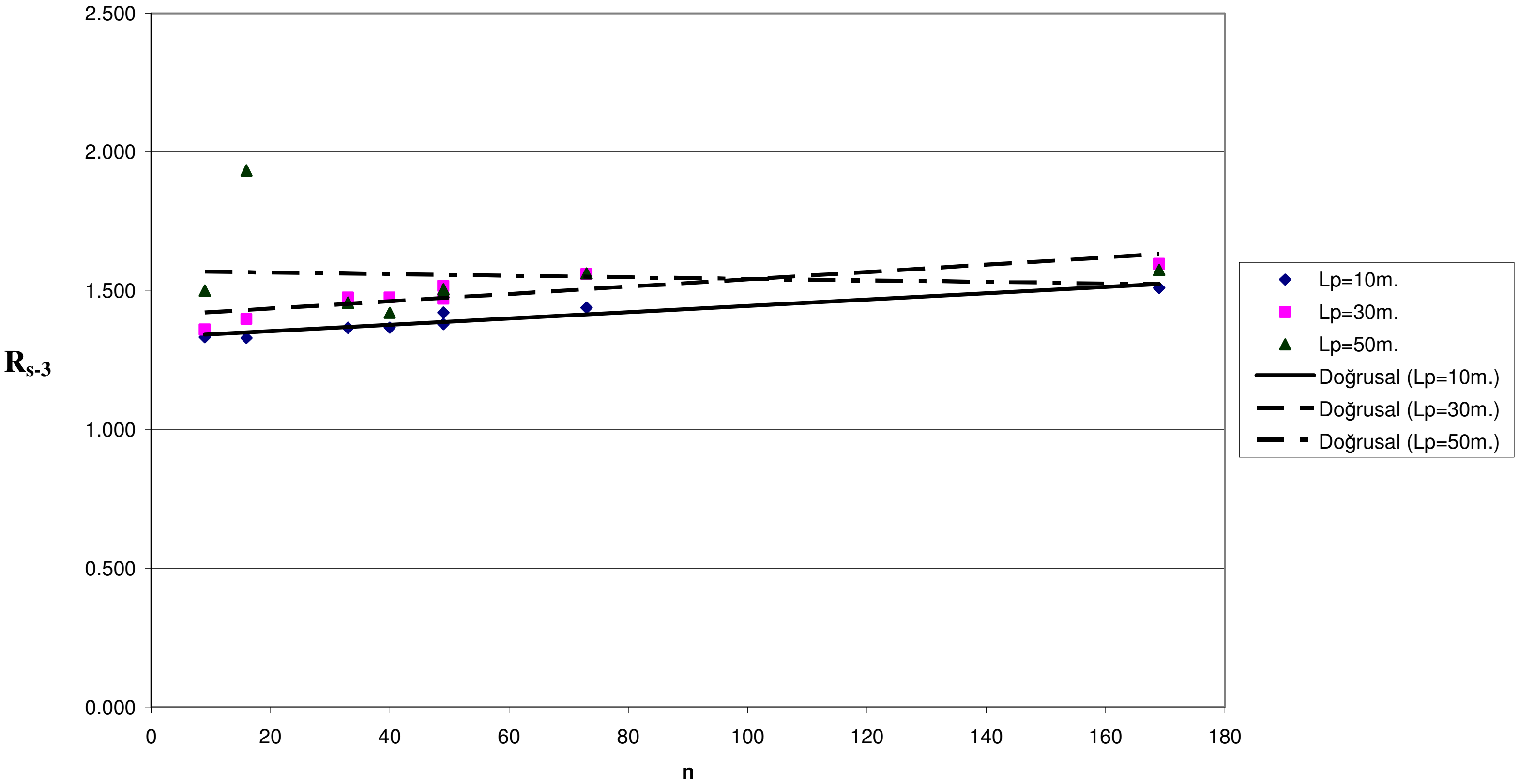

Fig B.6 R_{s-3} for V_{ult}/P_{eff} = 20

APPENDIX C

Comparison Between

"Kaltakci, V. Method"

&

ABAQUS Analysis

Table C.1

Pile Configuration	s/d_p	n	V_{ult}/P_{eff}	Page
Pile Configuration - 1	3	169	5	149
Pile Configuration - 1	3	169	20	150
Pile Configuration - 1	6	49	5	151
Pile Configuration - 1	6	49	20	152
Pile Configuration - 2	3	49	5	153
Pile Configuration - 2	3	49	20	154
Pile Configuration - 2	6	16	5	155
Pile Configuration - 2	6	16	20	156
Pile Configuration - 2	6	9	5	157
Pile Configuration - 2	6	9	20	158
Pile Configuration - 3	3	73	5	159
Pile Configuration - 3	3	73	20	160
Pile Configuration - 3	6	40	5	161
Pile Configuration - 3	6	40	20	162
Pile Configuration - 3	6	33	5	163
Pile Configuration - 3	6	33	20	164

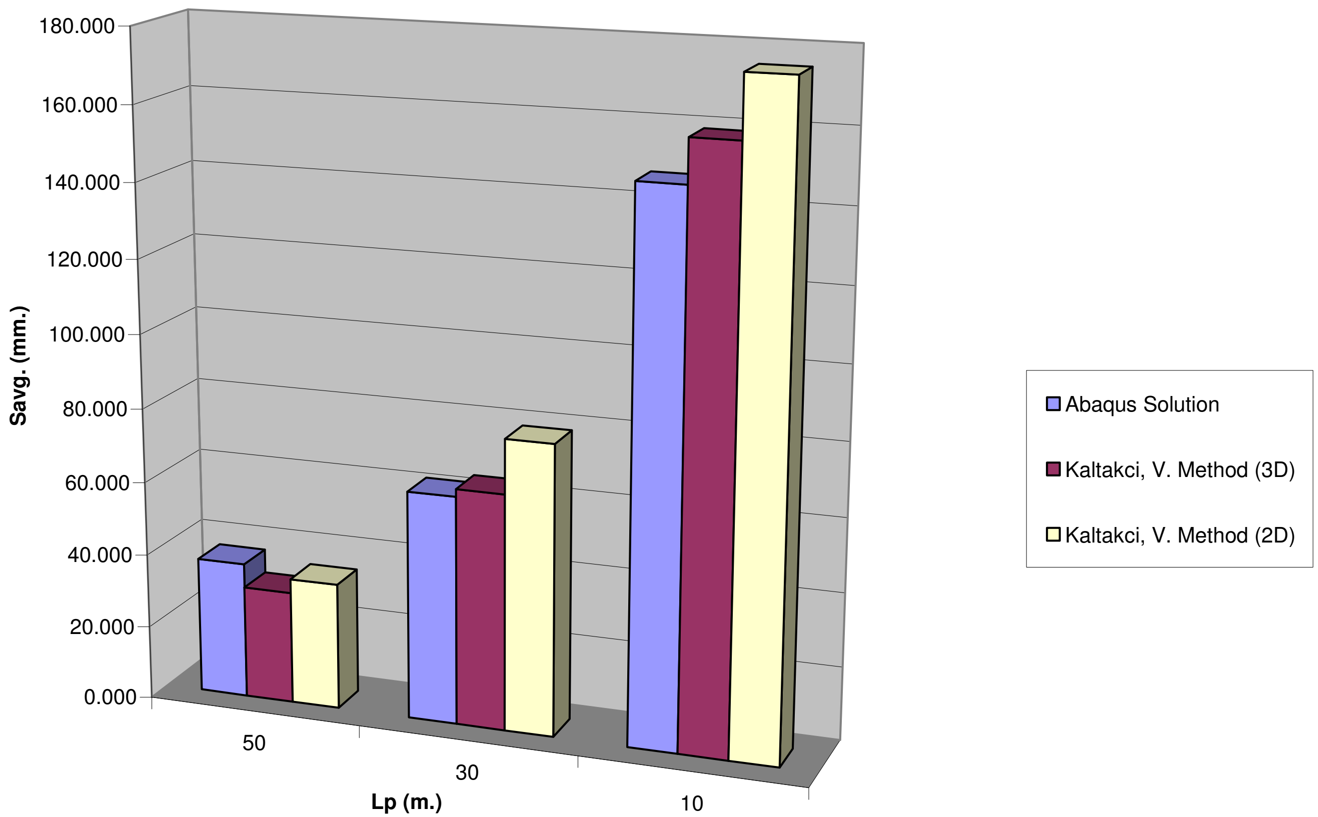

Fig C.1 $S_{avg.}$ vs. L_p for Pile Configuration -1 , n=169 & V_{ult} / P_{eff} = 5

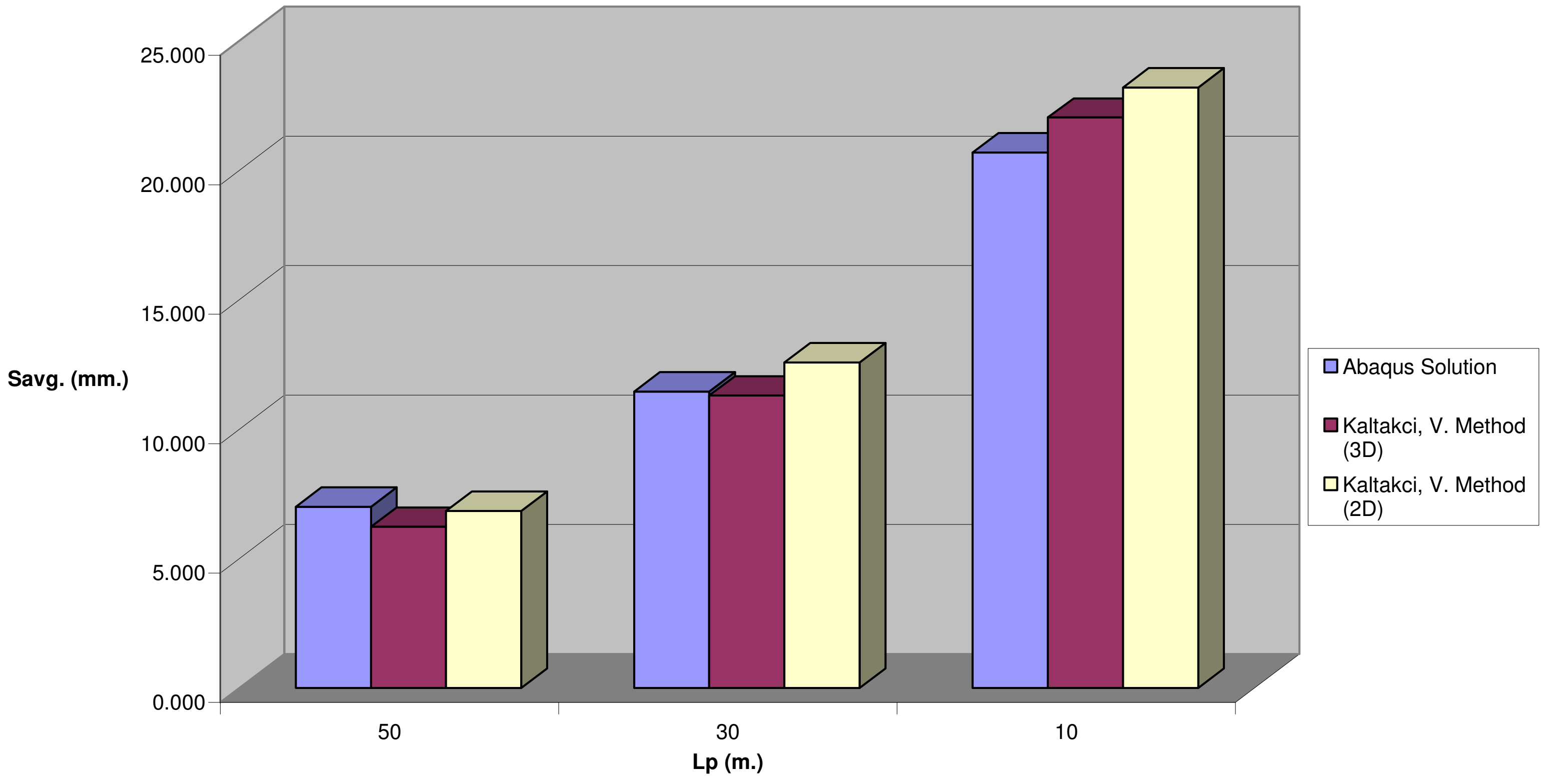

Fig C.2 $S_{avg.}$ vs. L_p for Pile Configuration -1 , n=169 & V_{ult} / P_{eff} = 20

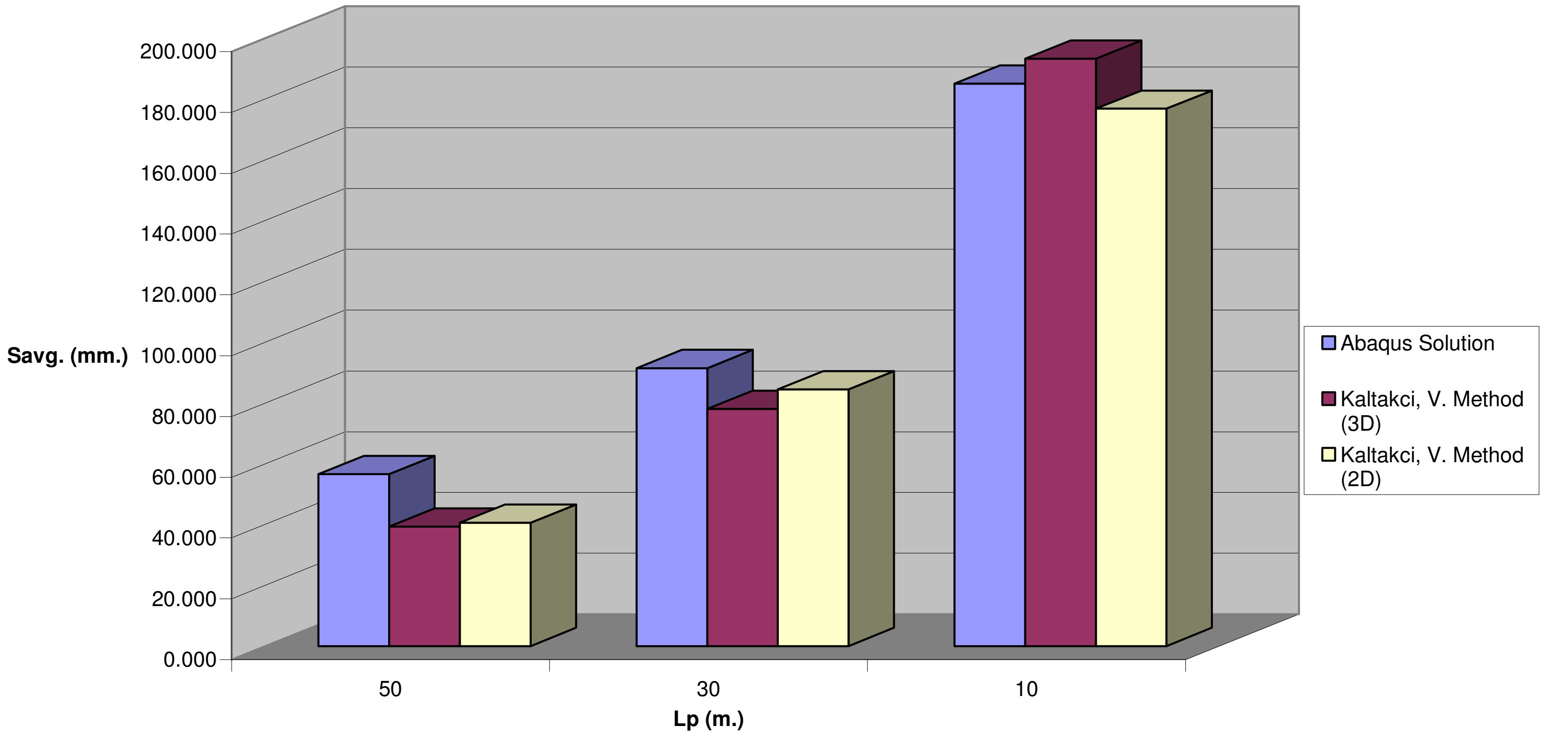

Fig C.3 $S_{avg.}$ vs. L_p for Pile Configuration -1 , n=49 & V_{ult} / P_{eff} = 5

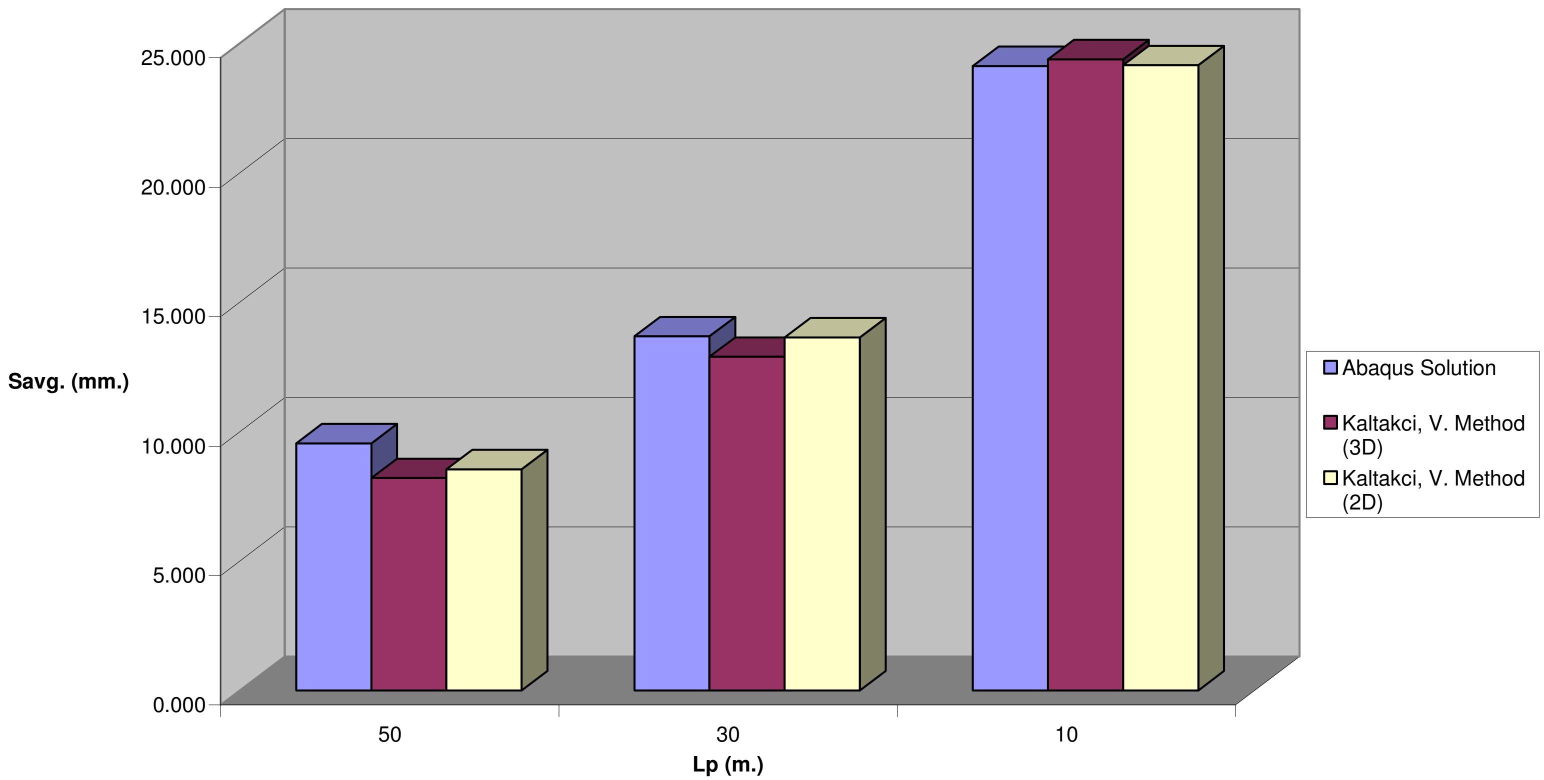

Fig C.4 $S_{avg.}$ vs. L_p for Pile Configuration -1 , n=49 & V_{ult} / P_{eff} = 20

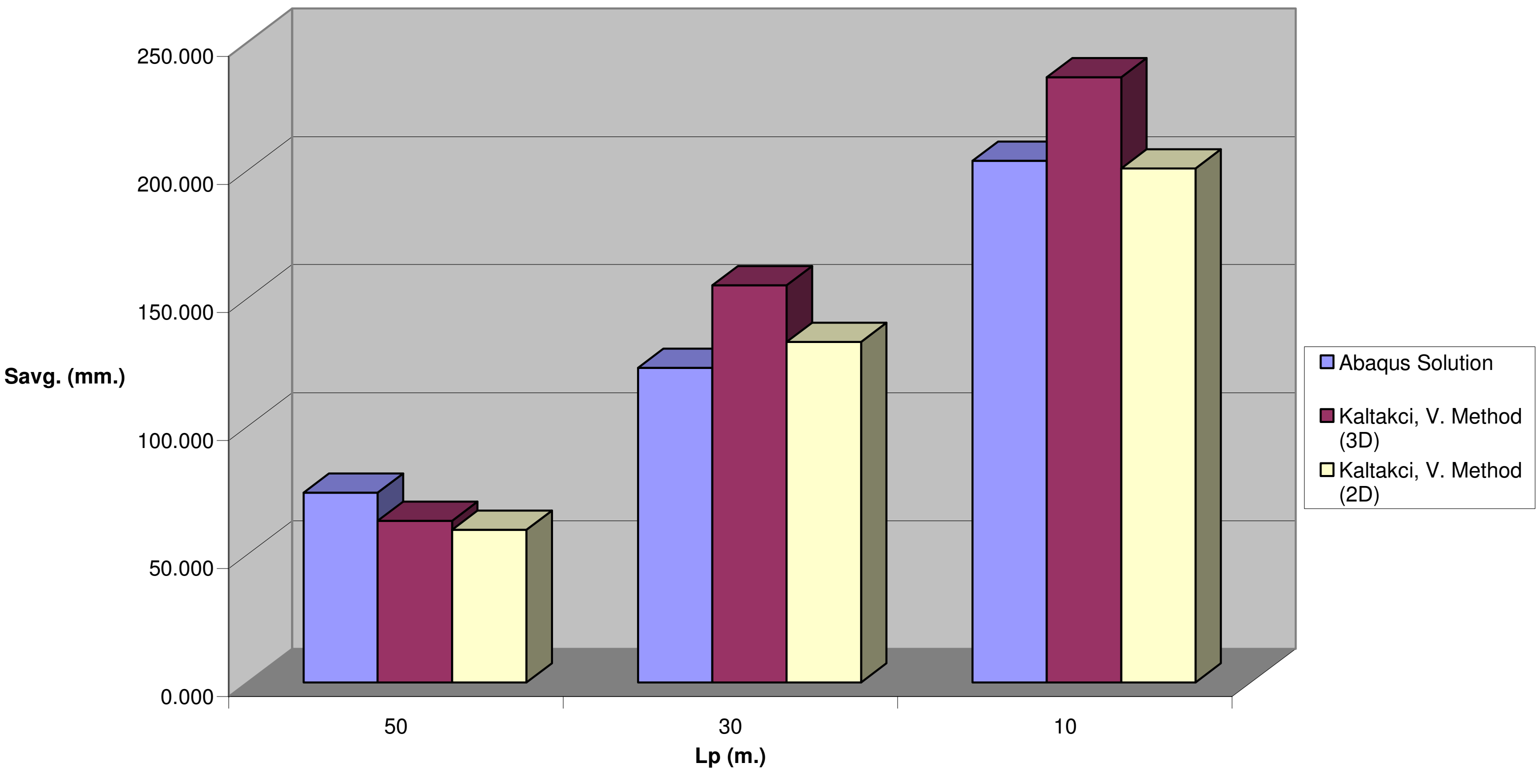

Fig C.5 $S_{avg.}$ vs. L_p for Pile Configuration -2 , n=49 & V_{ult} / P_{eff} = 5

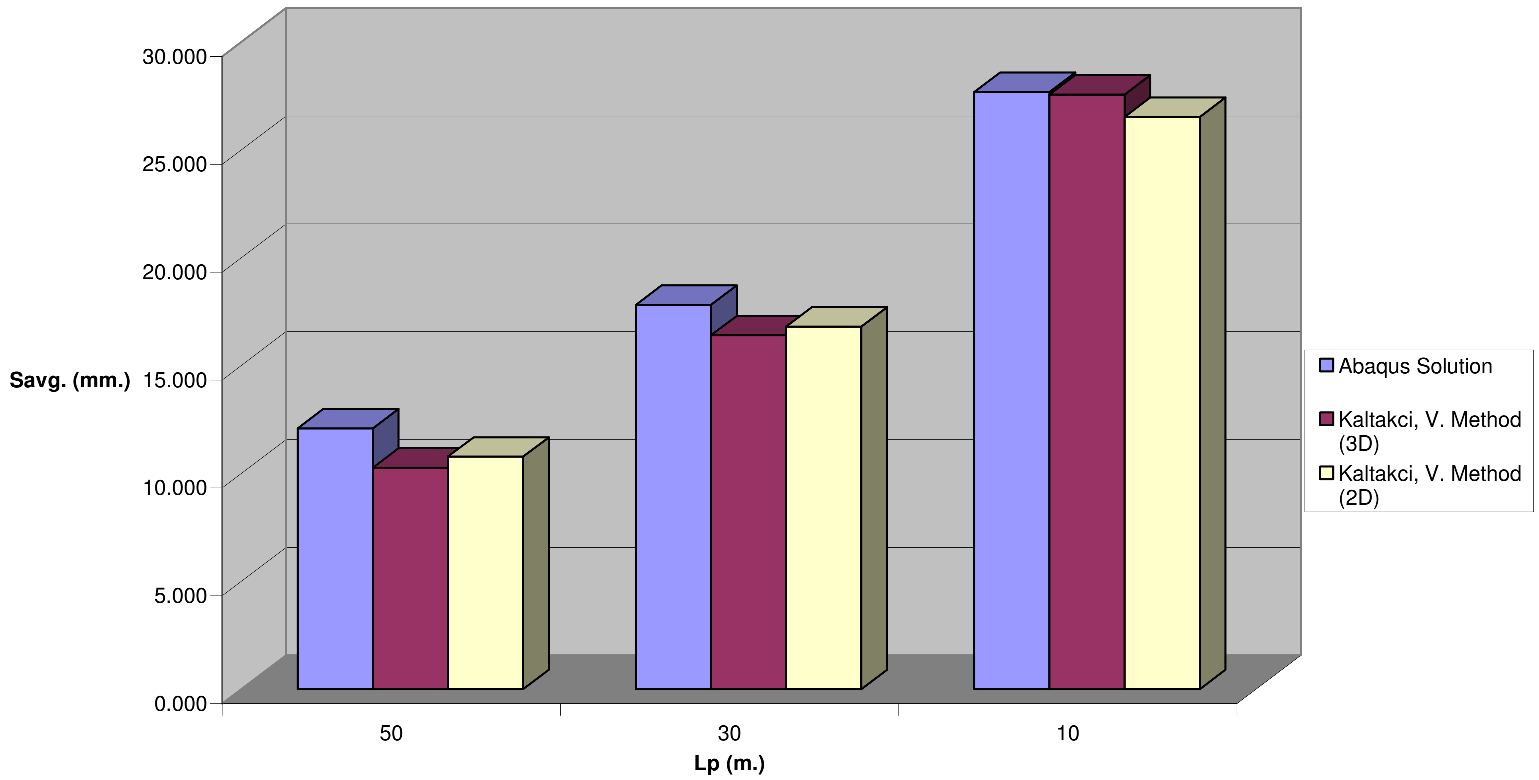

Fig C.6 $S_{avg.}$ vs. L_p for Pile Configuration -2 , n=49 & V_{ult} / P_{eff} = 20

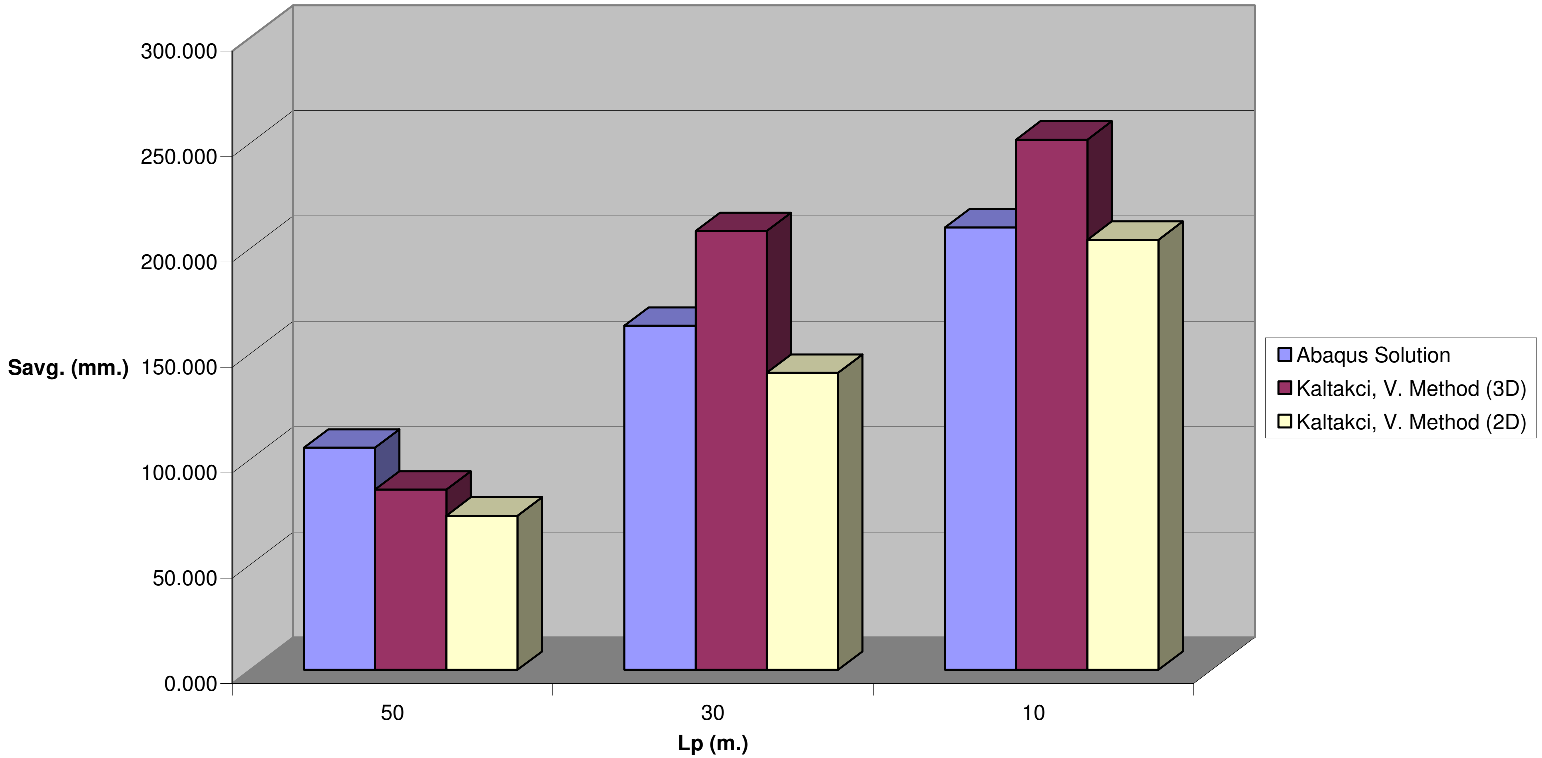

Fig C.7 $S_{avg.}$ vs. L_p for Pile Configuration -2 , n=16 & V_{ult} / P_{eff} = 5

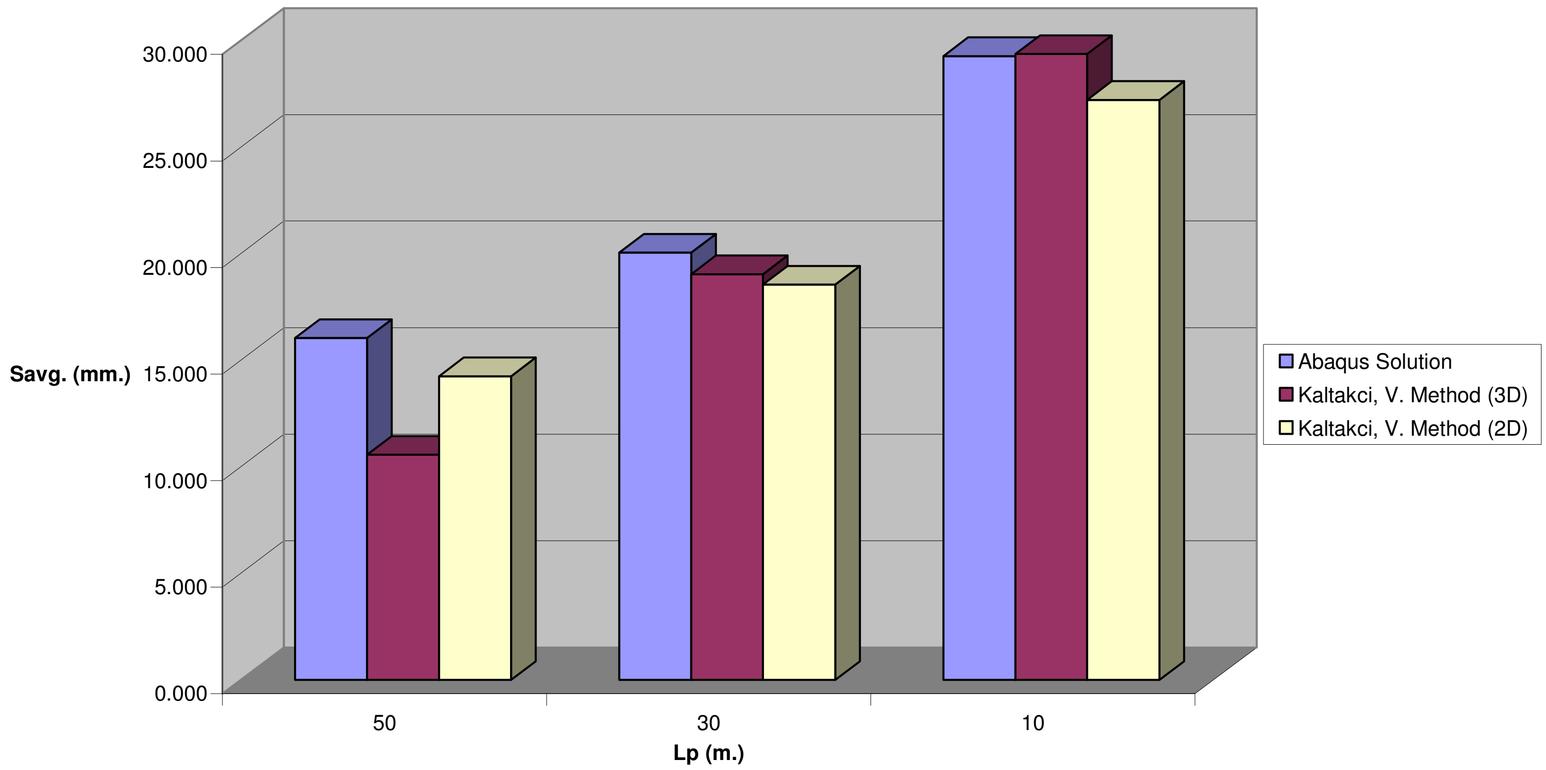

Fig C.8 $S_{avg.}$ vs. L_p for Pile Configuration -2 , n=16 & V_{ult} / P_{eff} = 20

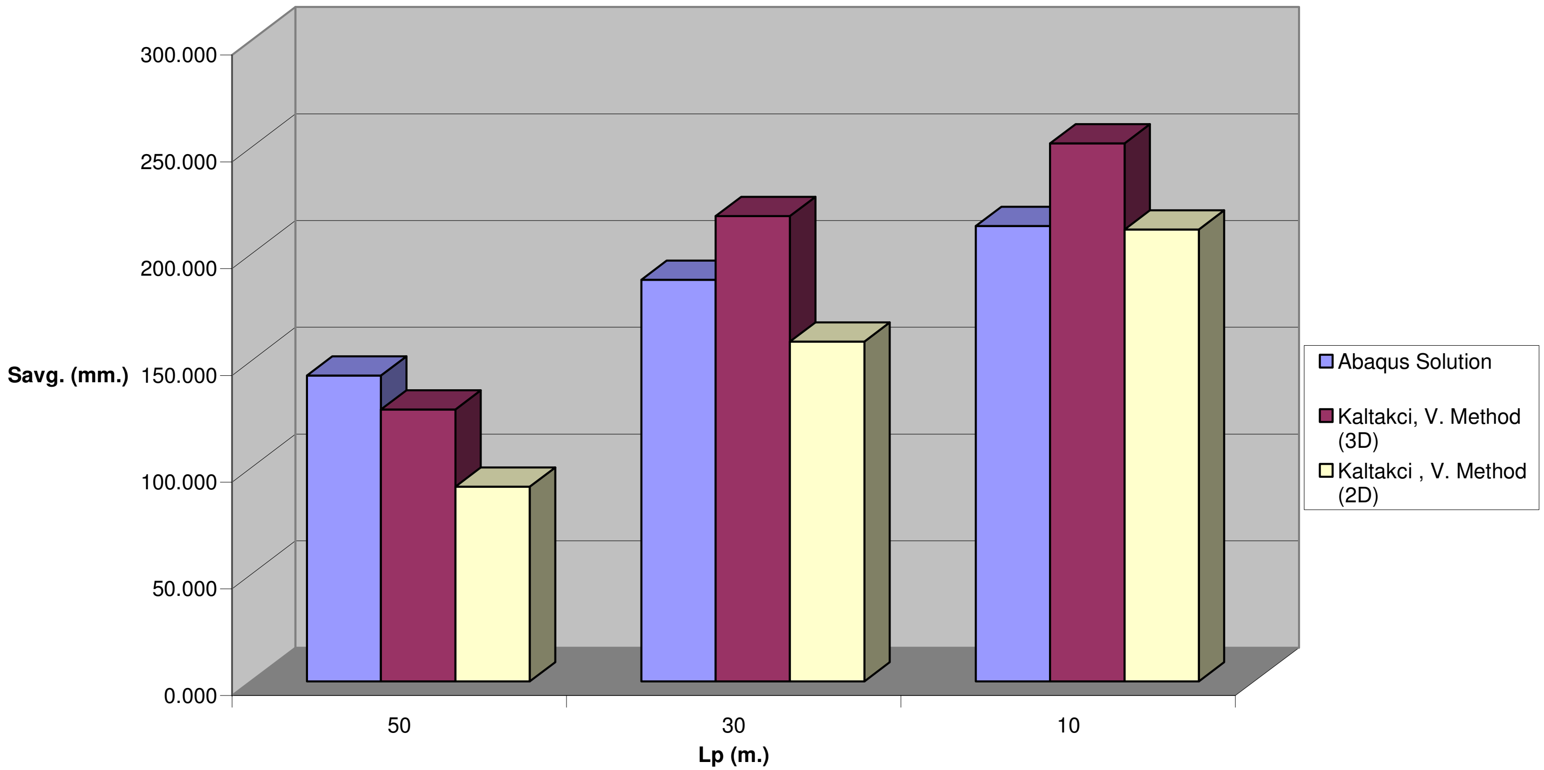

Fig C.9 $S_{avg.}$ vs. L_p for Pile Configuration -2 , n=9 & V_{ult} / P_{eff} = 5

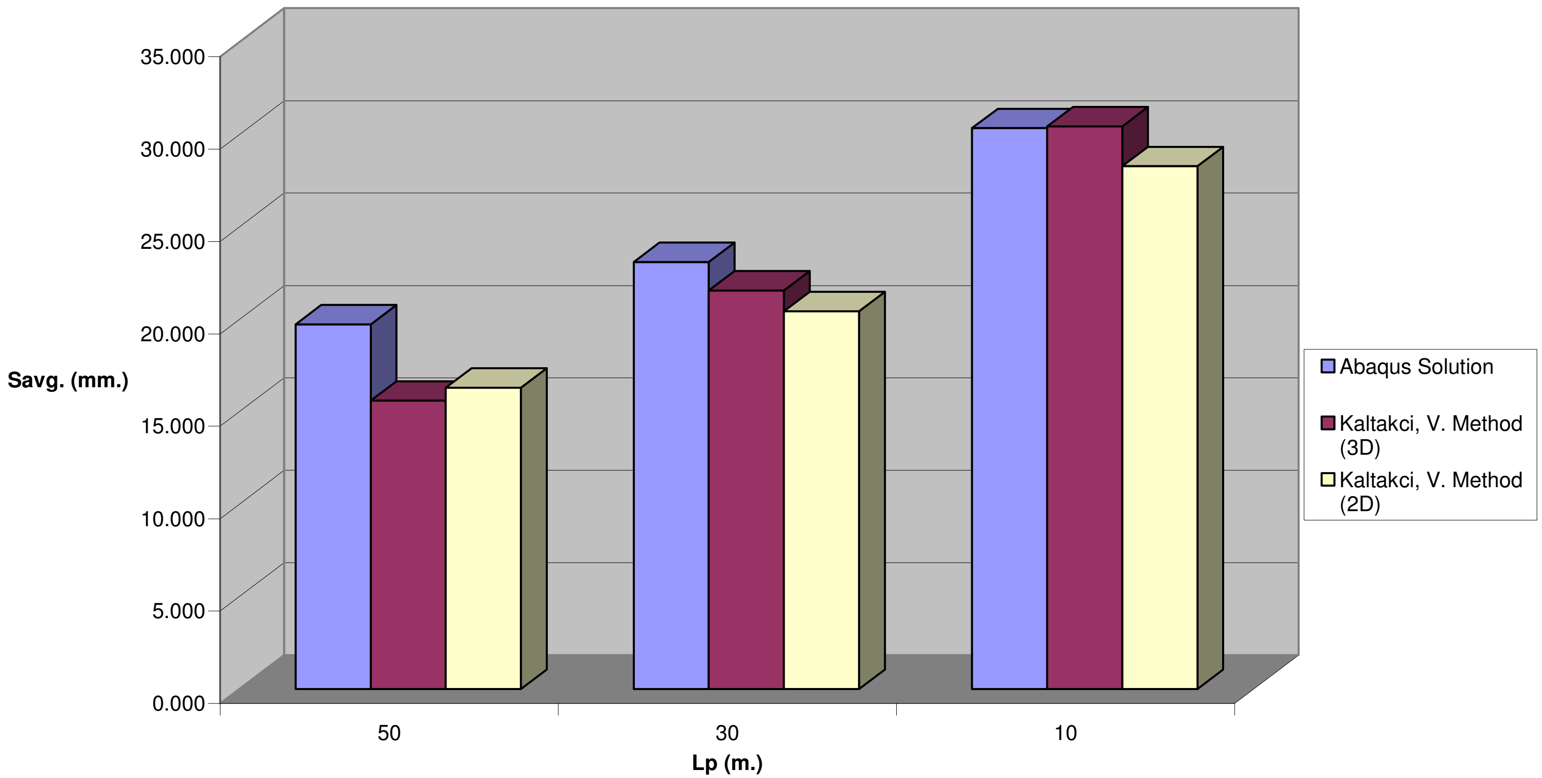

Fig C.10 $S_{avg.}$ vs. L_p for Pile Configuration -2 , n=9 & V_{ult} / P_{eff} = 20

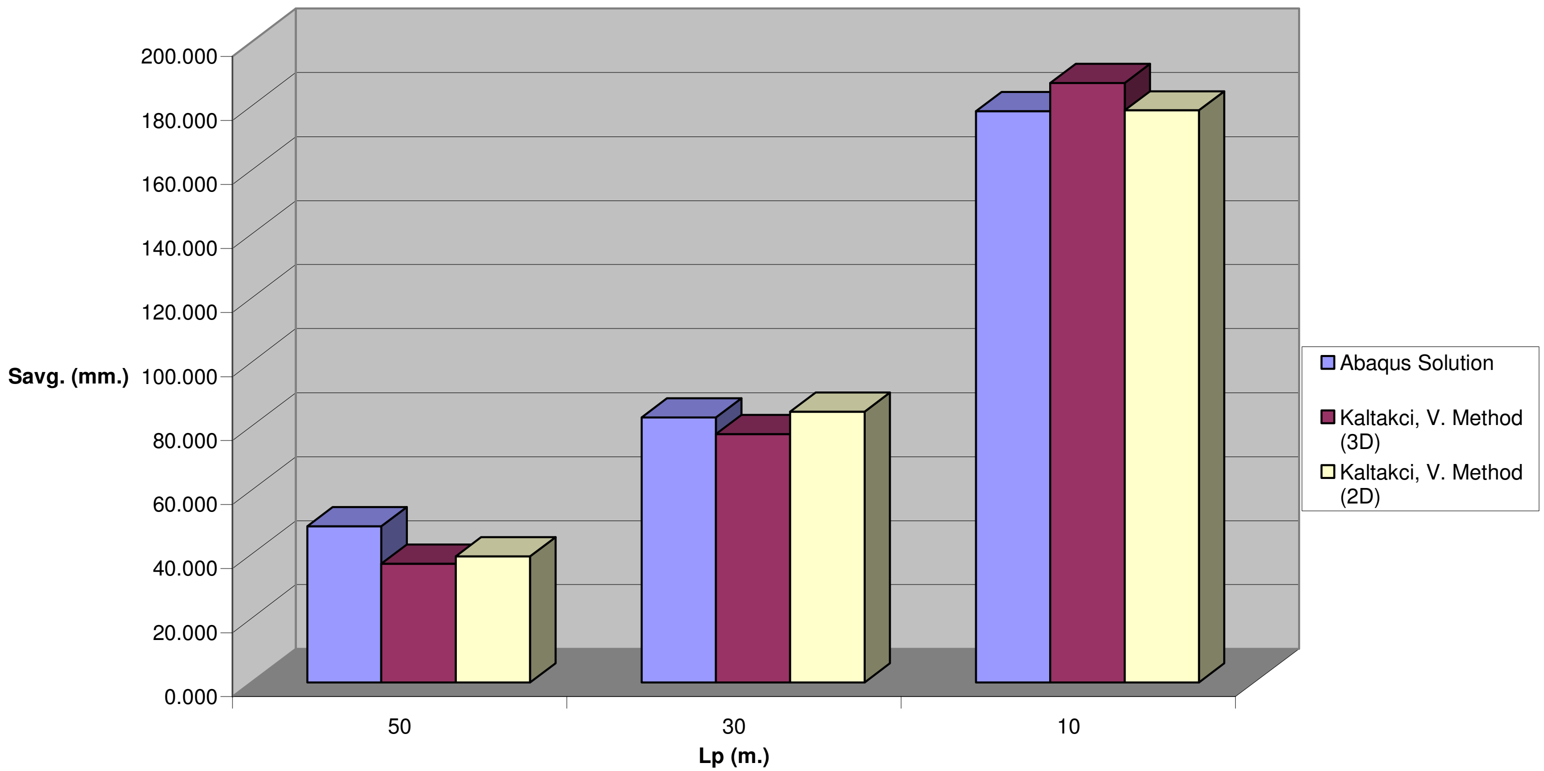

Fig C.11 $S_{avg.}$ vs. L_p for Pile Configuration -3 , n=73 & V_{ult} / P_{eff} = 5

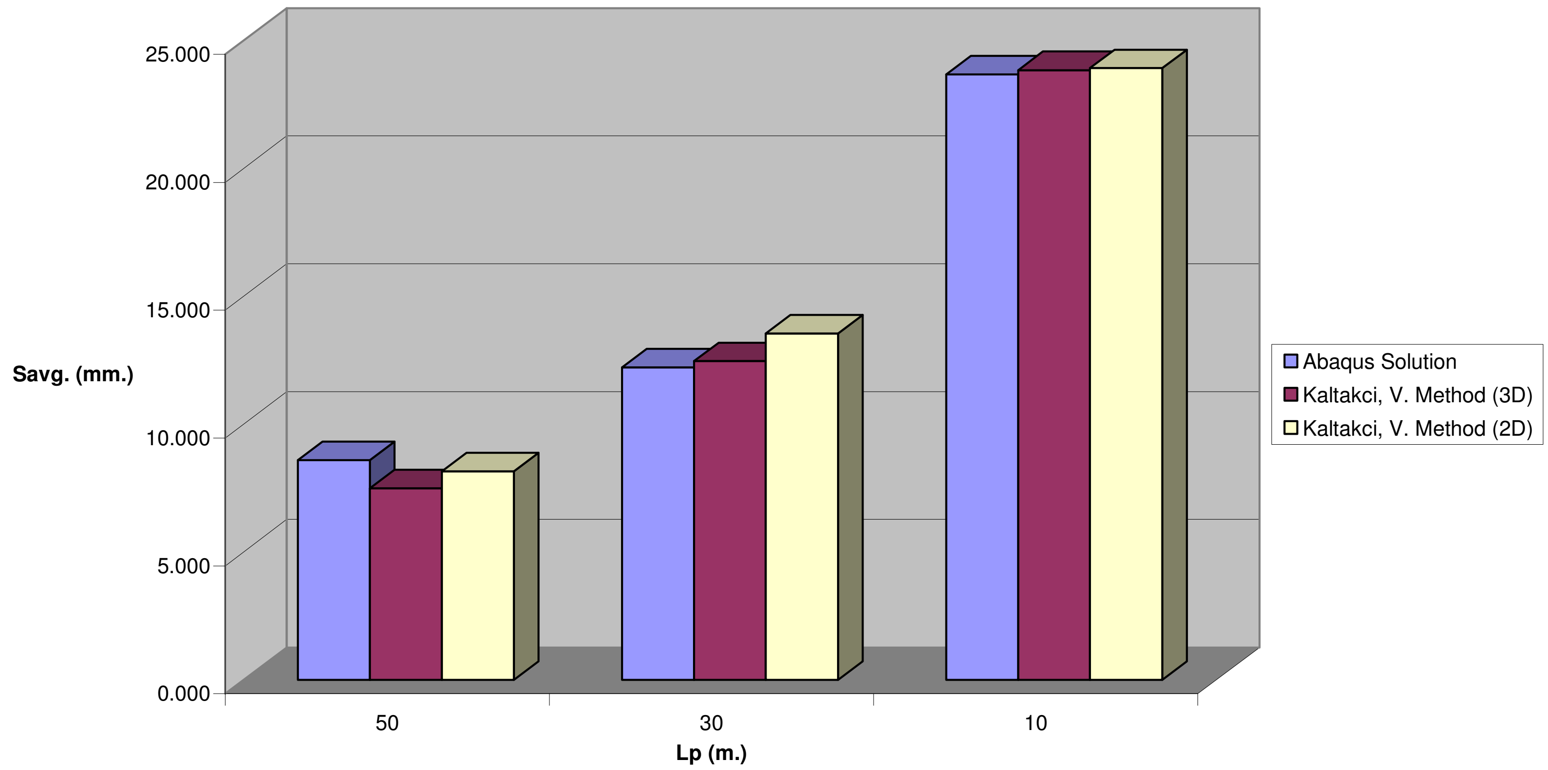

Fig C.12 $S_{avg.}$ vs. L_p for Pile Configuration -3 , n=73 & V_{ult} / P_{eff} = 20

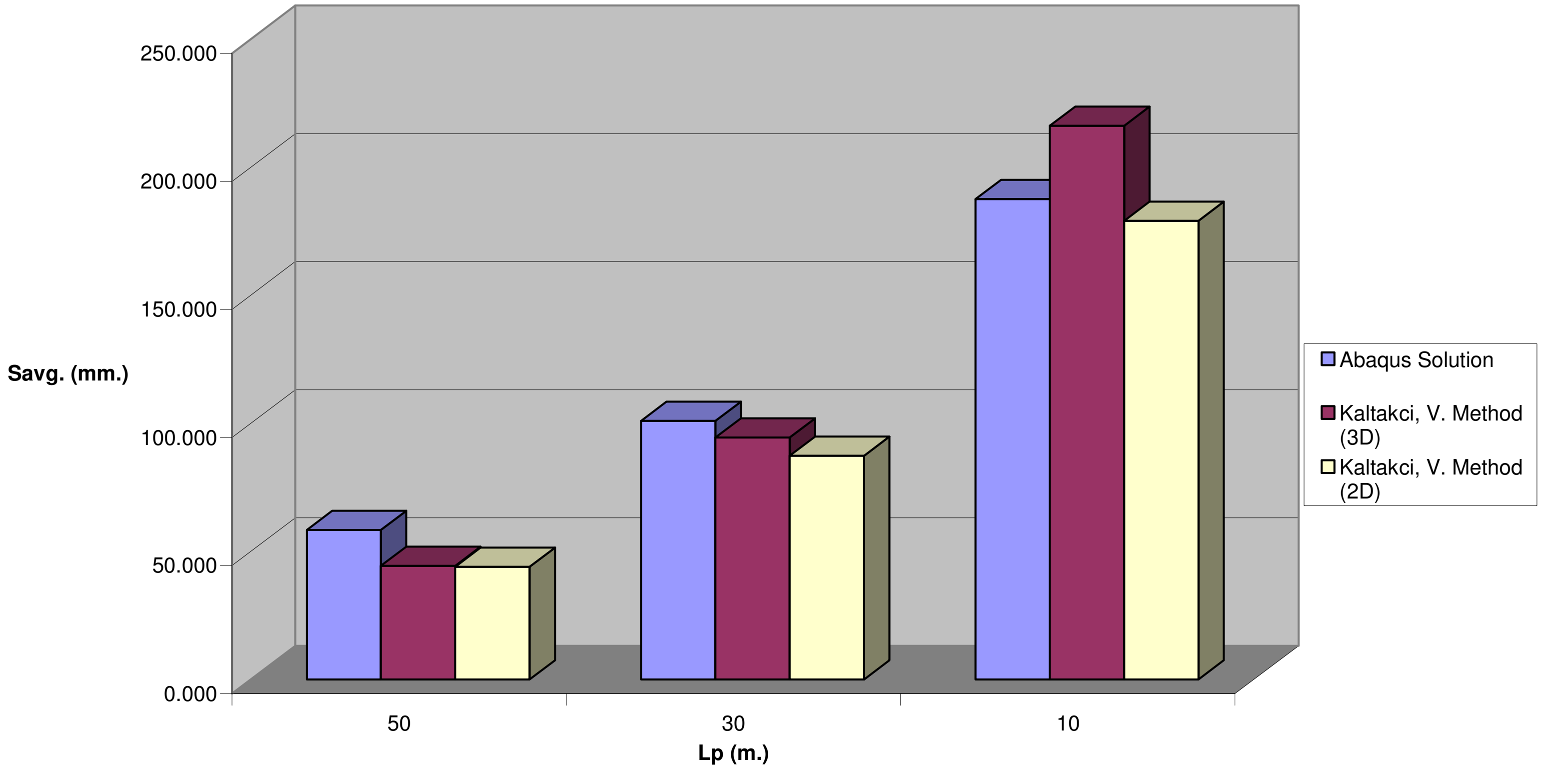

Fig C.13 $S_{avg.}$ vs. L_p for Pile Configuration -3 , n=40 & V_{ult} / P_{eff} = 5

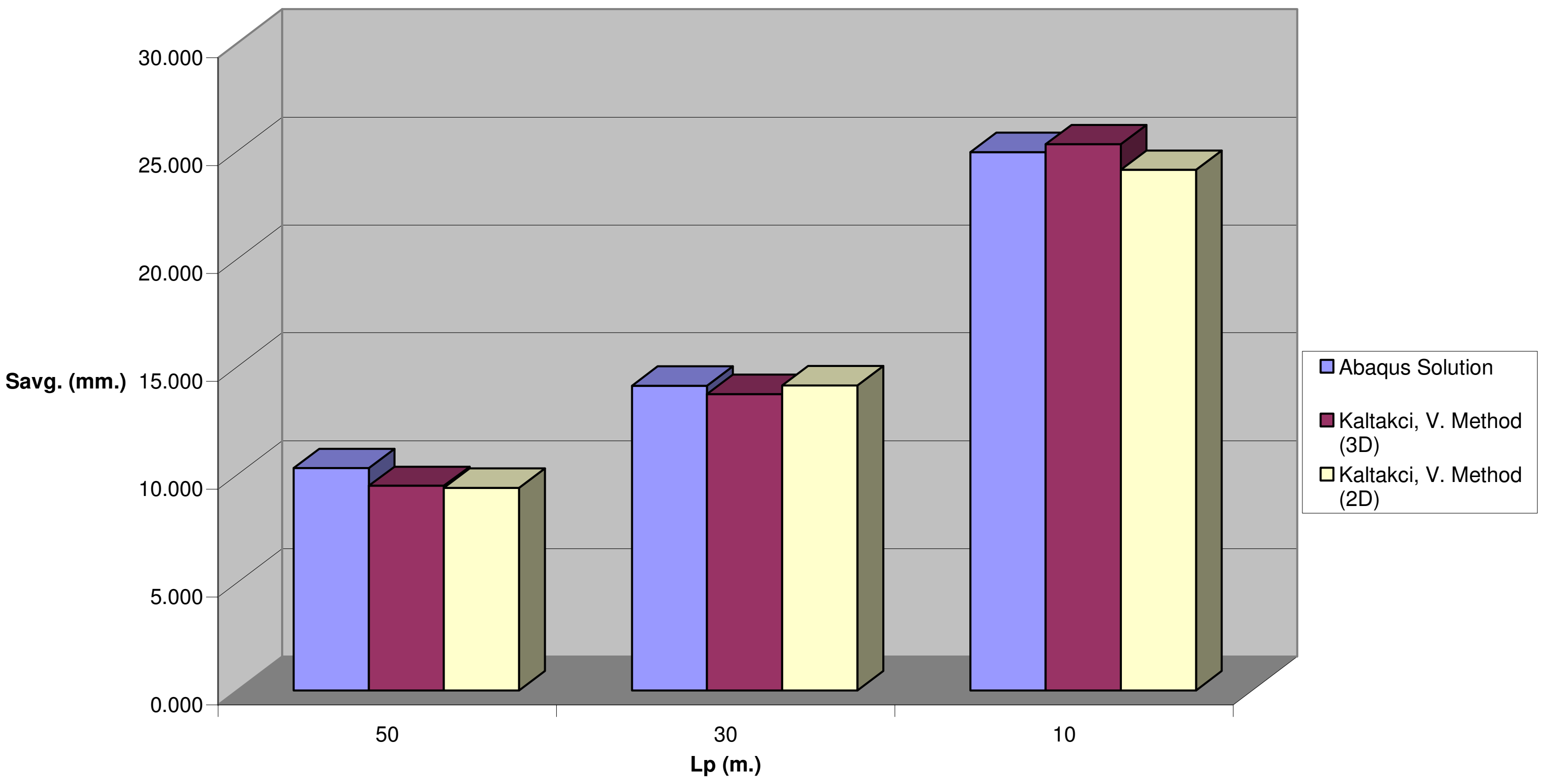

Fig C.14 $S_{avg.}$ vs. L_p for Pile Configuration -3 , n=40 & V_{ult} / P_{eff} = 20

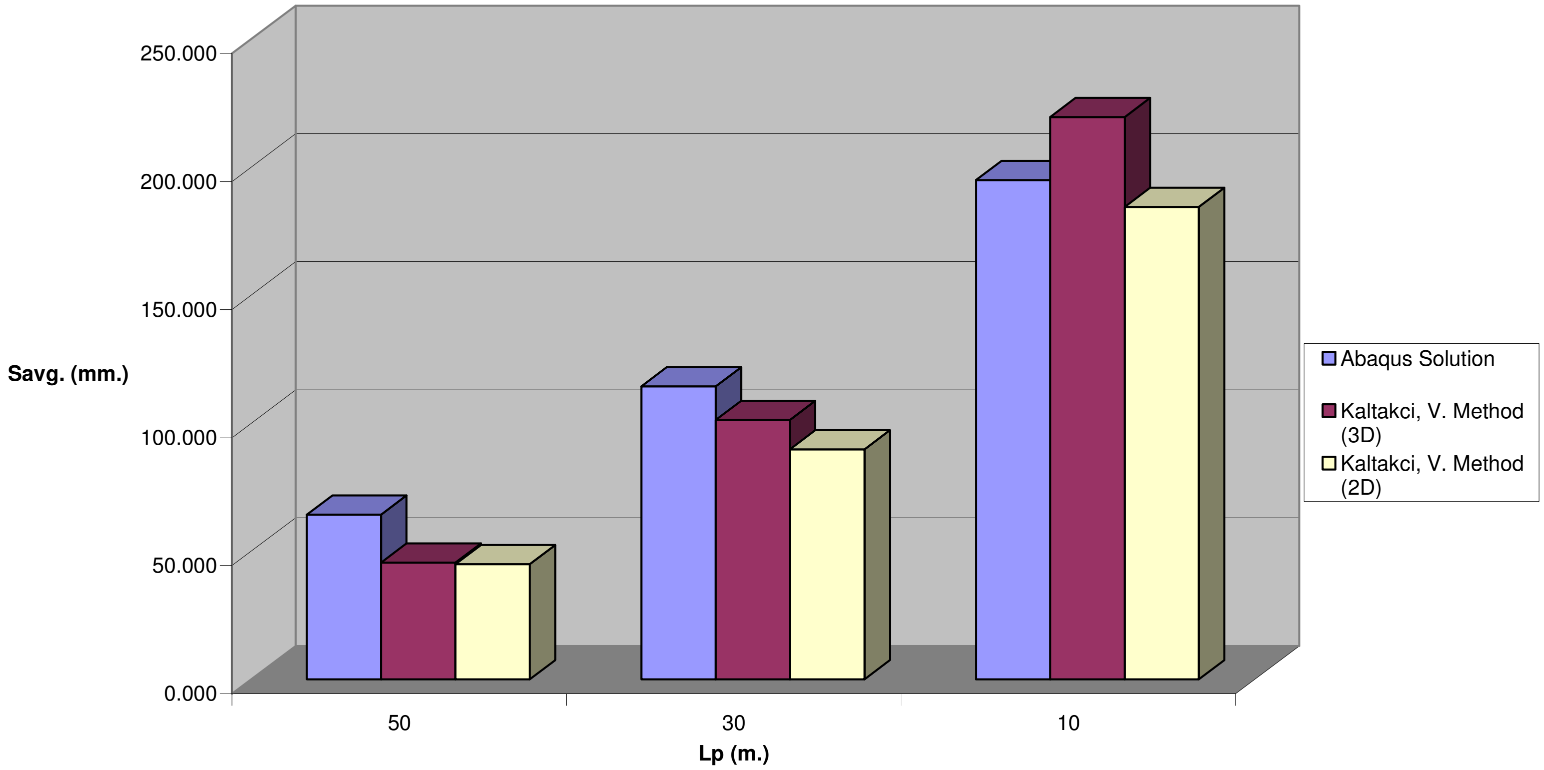

Fig C.15 $S_{avg.}$ vs. L_p for Pile Configuration -3 , n=33 & V_{ult} / P_{eff} = 5

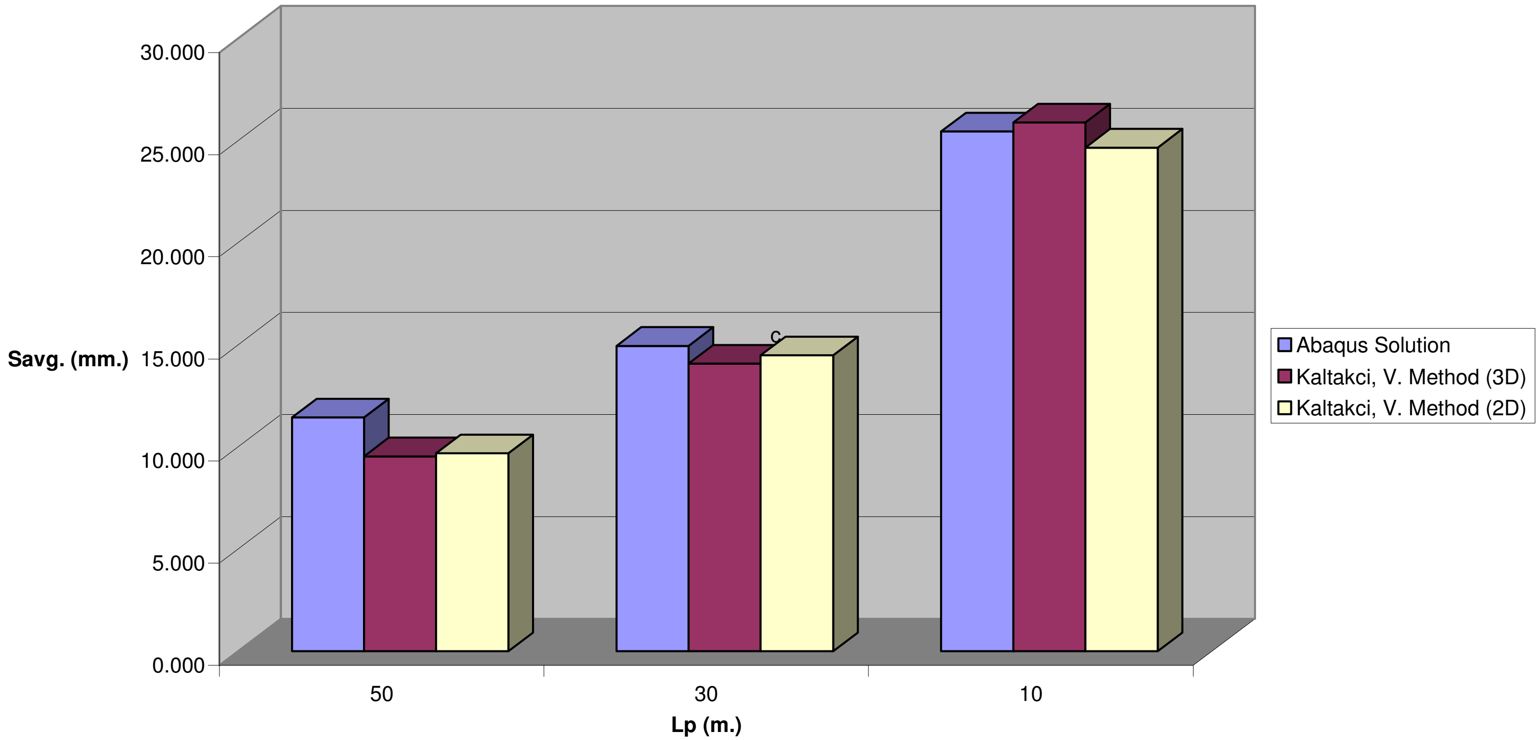

Fig C.16 $S_{avg.}$ vs. L_p for Pile Configuration -3 , n=33 & V_{ult} / P_{eff} = 20

APPENDIX D

$\xi_{\Delta s}$ vs. $n*L_p$ Graphs

Table D.1

Pile Configuration	s/d_p	n	V_{ult}/P_{eff}	Page
Pile Configuration - 1	3	169	5	166
Pile Configuration - 1	3	169	20	167
Pile Configuration - 1	6	49	5	168
Pile Configuration - 1	6	49	20	169
Pile Configuration - 2	3	49	5	170
Pile Configuration - 2	3	49	20	171
Pile Configuration - 2	6	16	5	172
Pile Configuration - 2	6	16	20	173
Pile Configuration - 2	6	9	5	174
Pile Configuration - 2	6	9	20	175
Pile Configuration - 3	3	73	5	176
Pile Configuration - 3	3	73	20	177
Pile Configuration - 3	6	40	5	178
Pile Configuration - 3	6	40	20	179
Pile Configuration - 3	6	33	5	180
Pile Configuration - 3	6	33	20	181

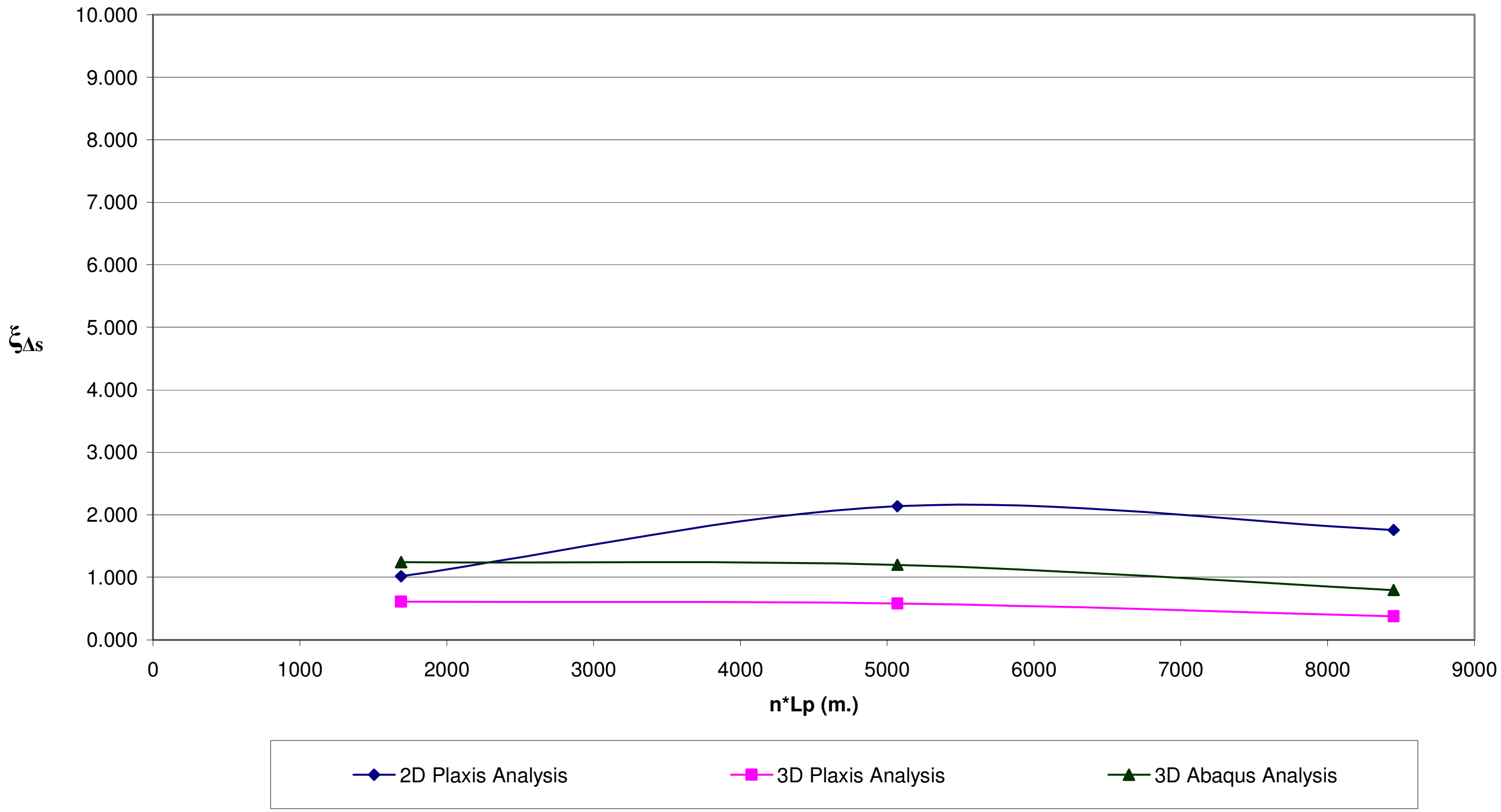

Fig D.1 $\xi_{\Delta s}$ vs. $n*L_p$ for Pile Configuration – 1, n=169 & $V_{ult}/P_{eff} = 5$

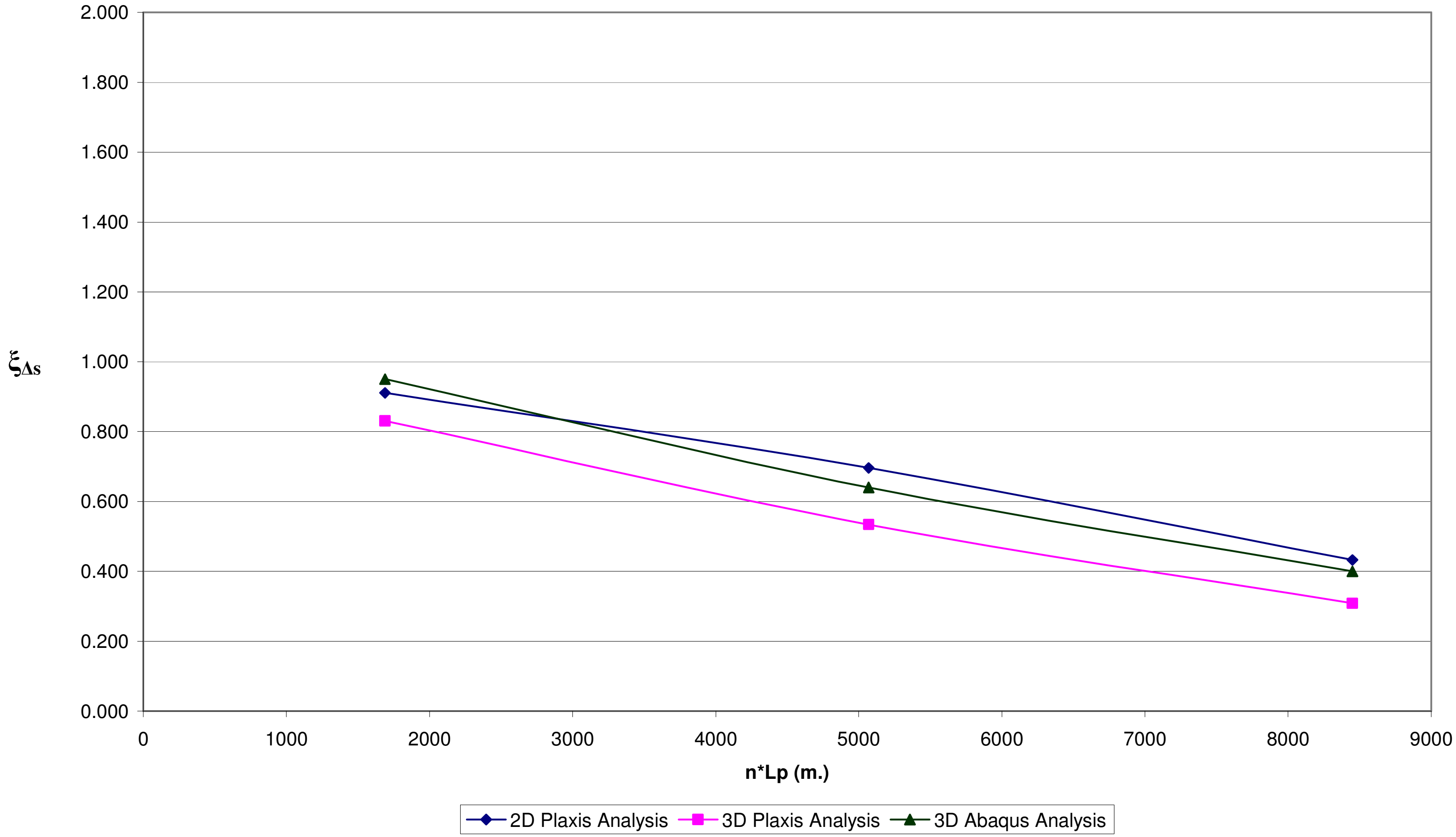

Fig D.2 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 1, n=169 & V_{ult}/P_{eff} = 20

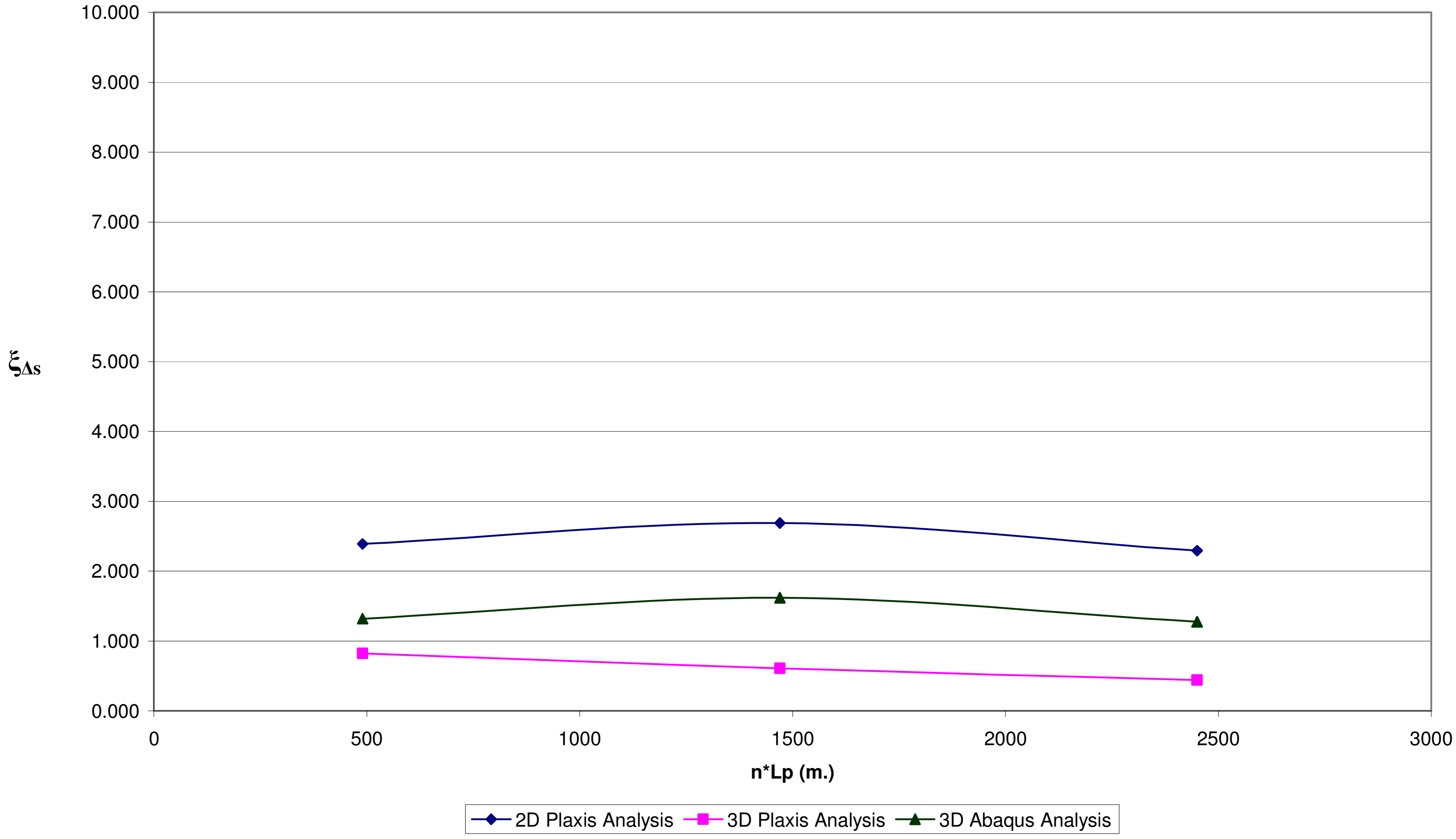

Fig D.3 $\xi_{\Delta s}$ vs. n^*L_p for Pile Configuration – 1, n=49 & V_{ult} / P_{eff} = 5

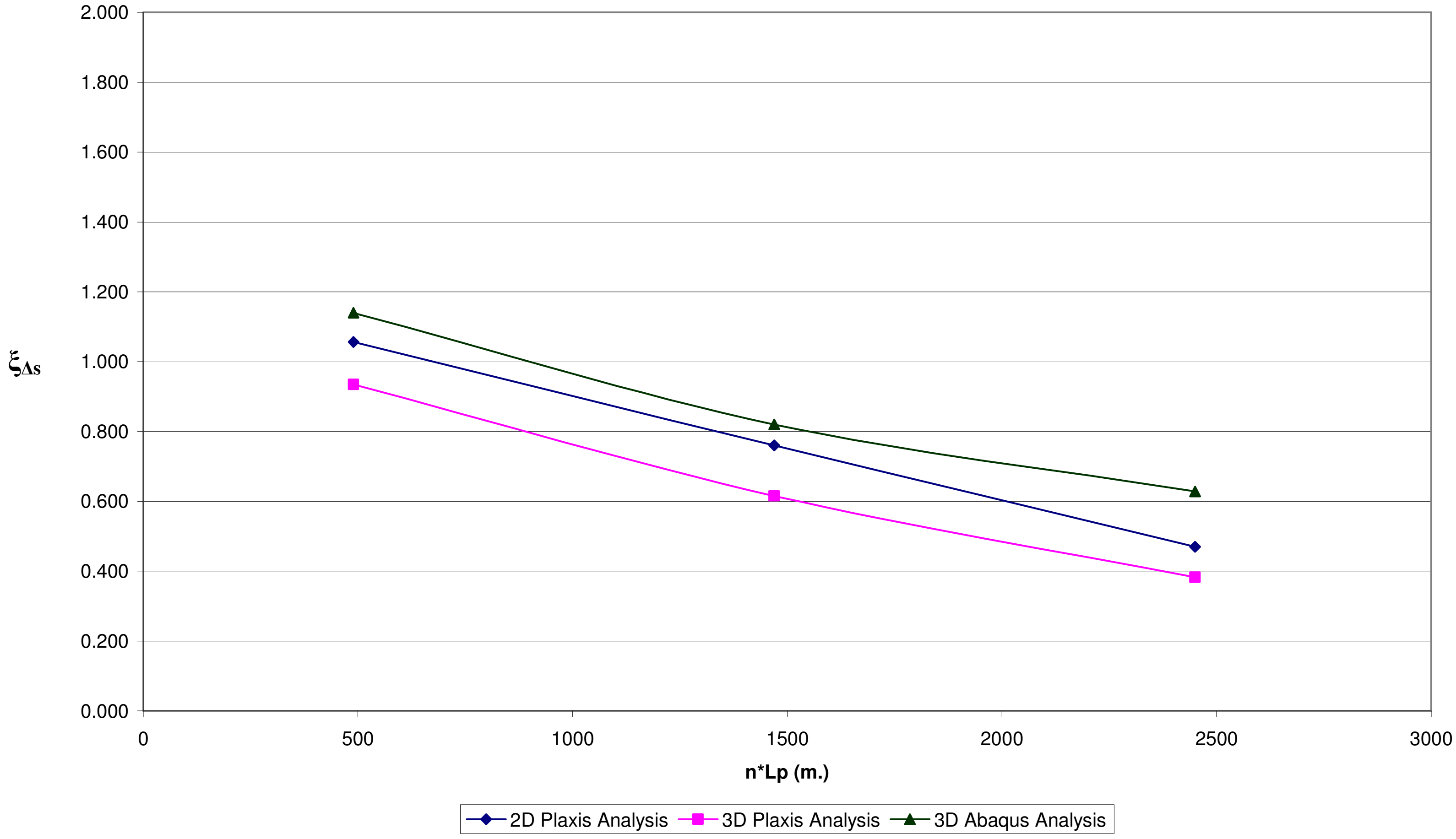

Fig D.4 $\xi_{\Delta s}$ vs. n^*L_p for Pile Configuration – 1, n=49 & V_{ult} / P_{eff} = 20

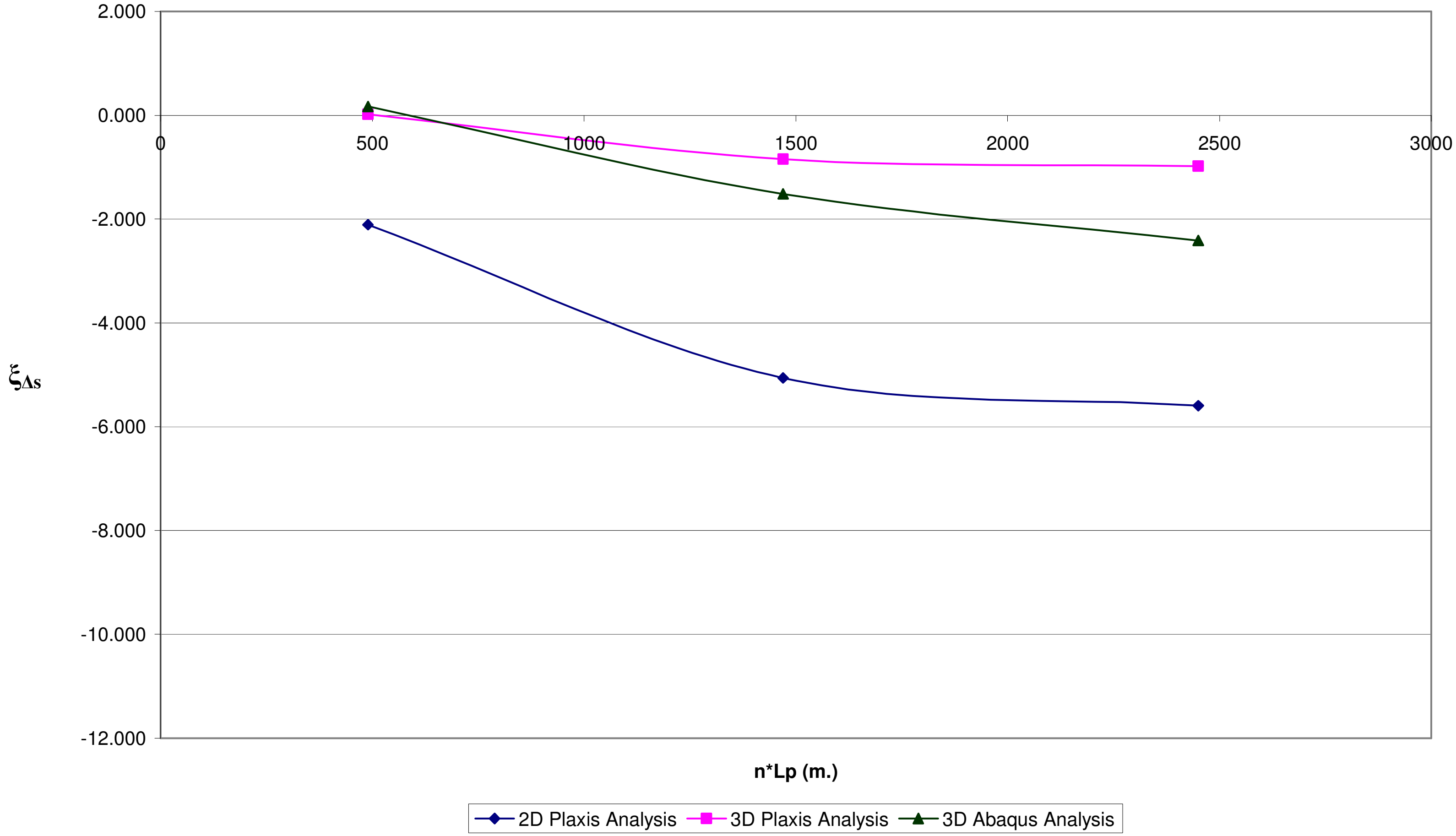

Fig D.5 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=49 & V_{ult} / P_{eff} = 5

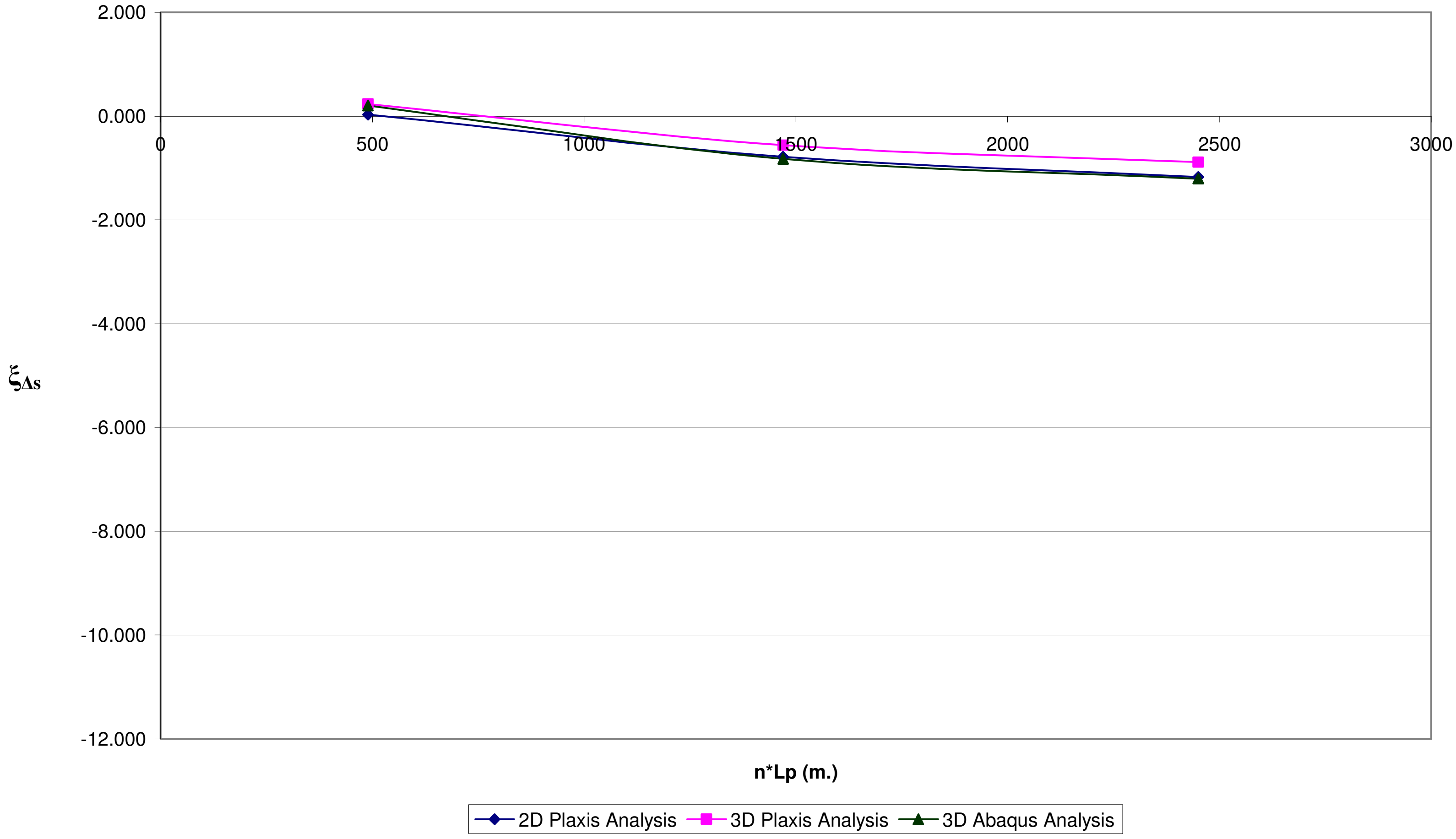

Fig D.6 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=49 & V_{ult} / P_{eff} = 20

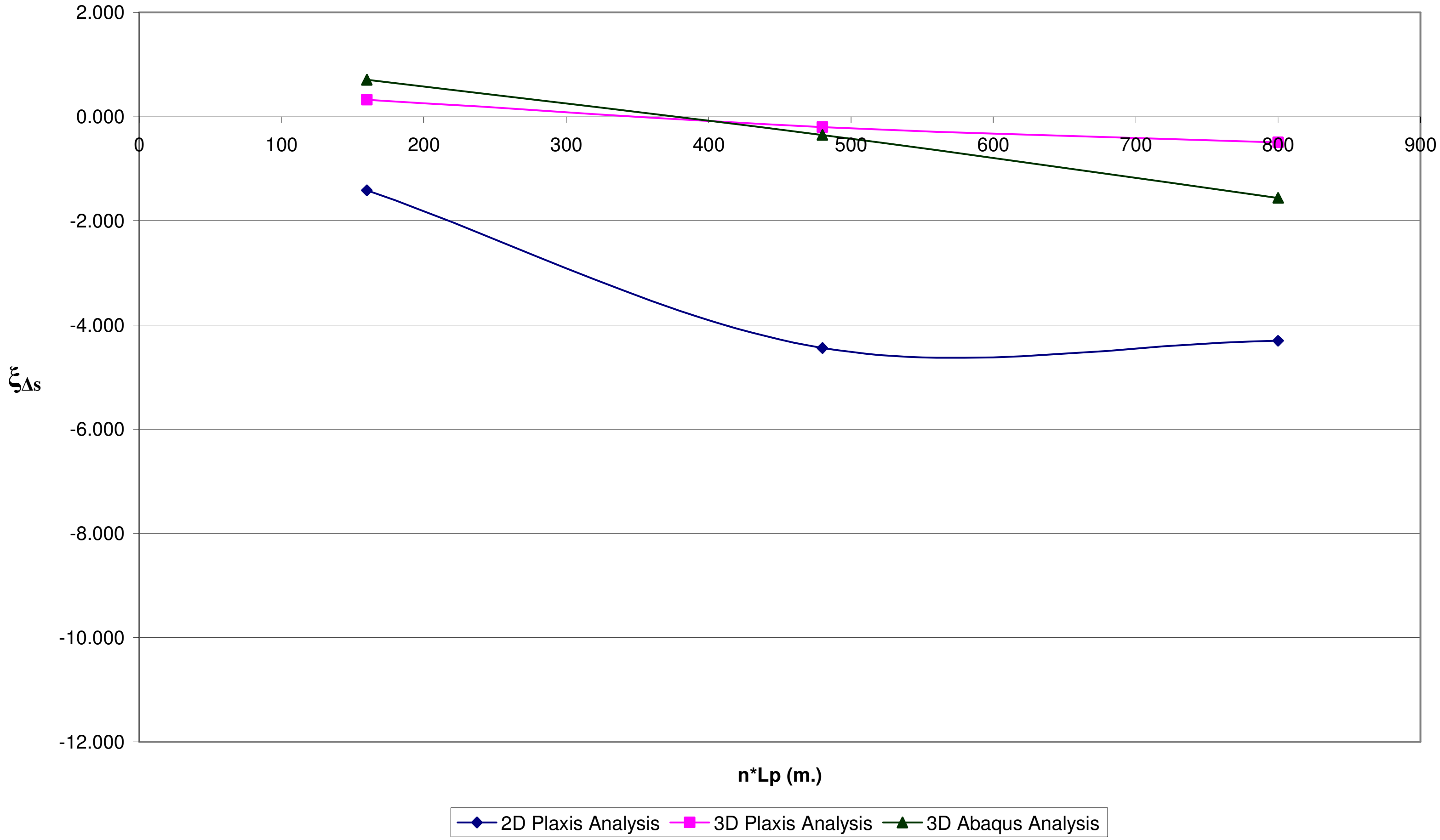

Fig D.7 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=16 & V_{ult} / P_{eff} = 5

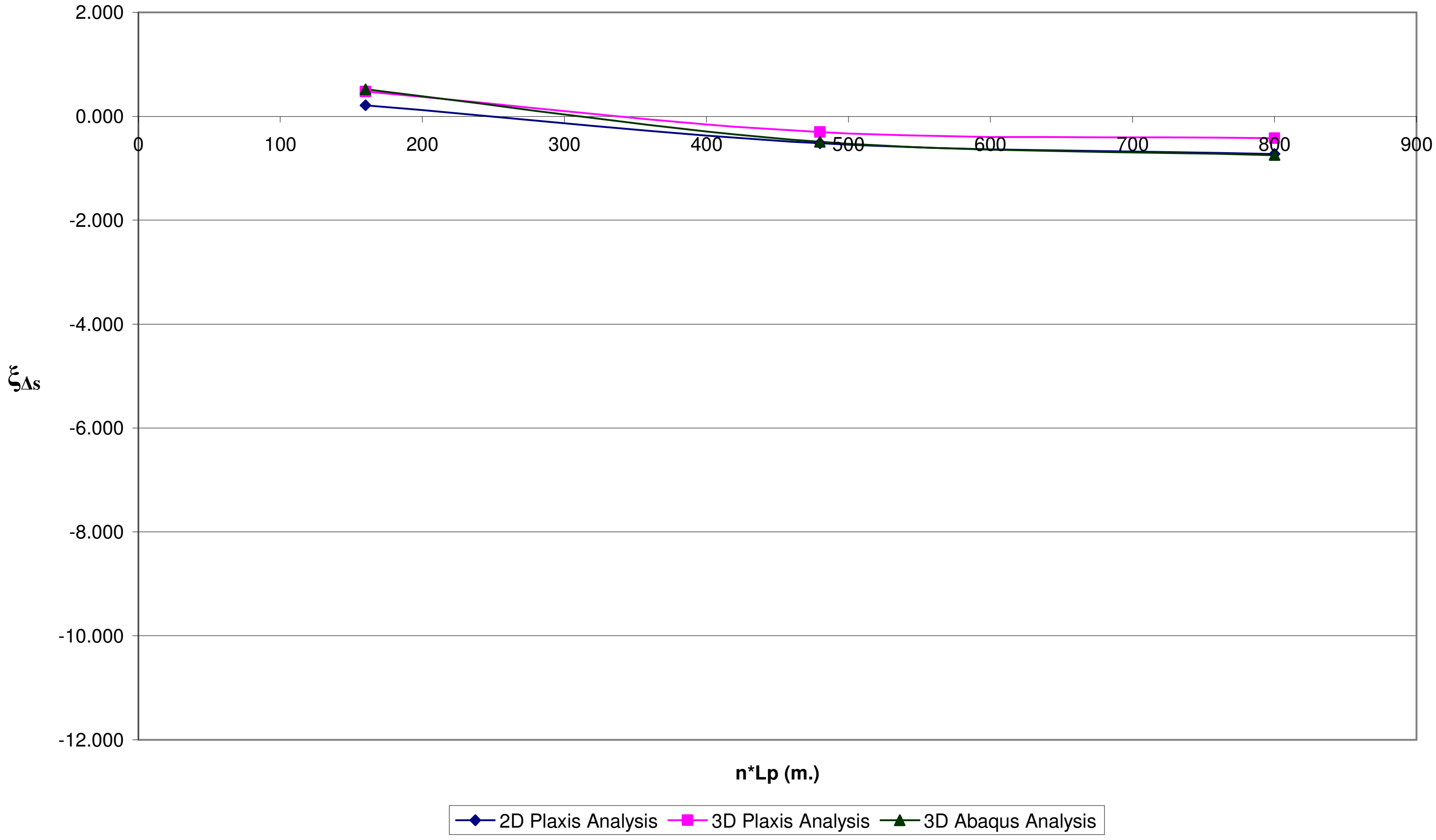

Fig D.8 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=16 & V_{ult} / P_{eff} = 20

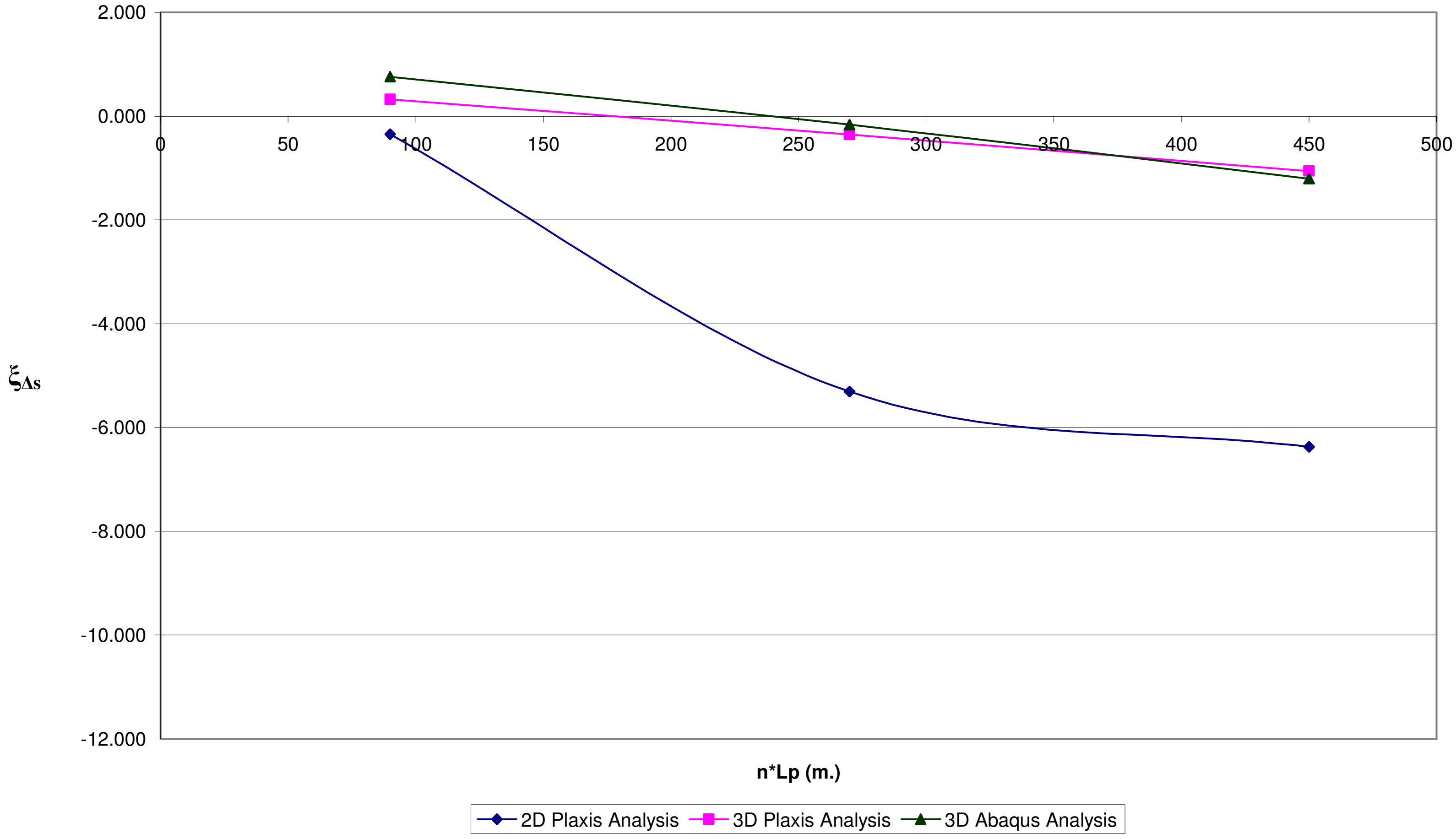

Fig D.9 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=9 & V_{ult} / P_{eff} = 5

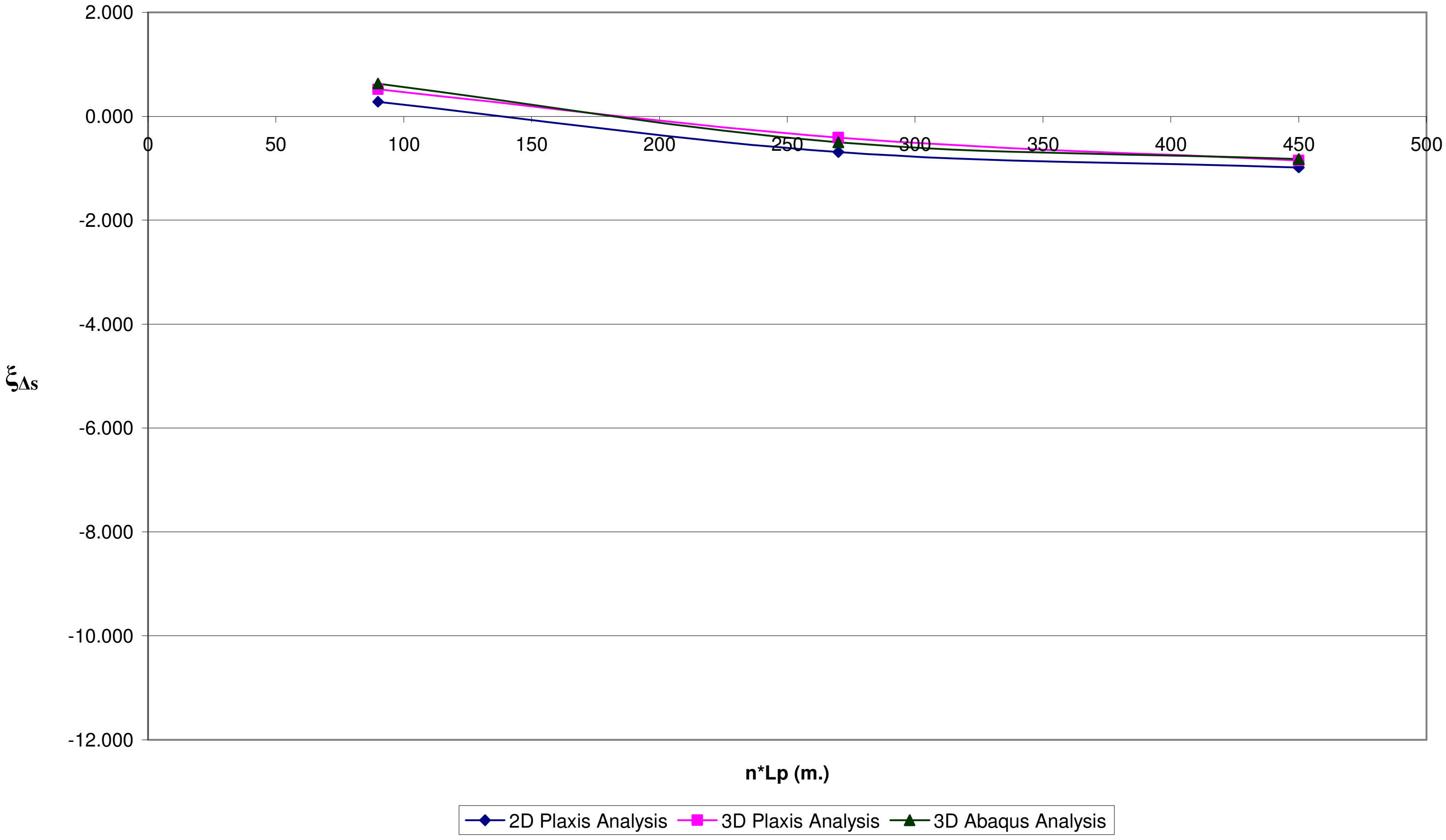

Fig D.10 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 2, n=9 & V_{ult} / P_{eff} = 20

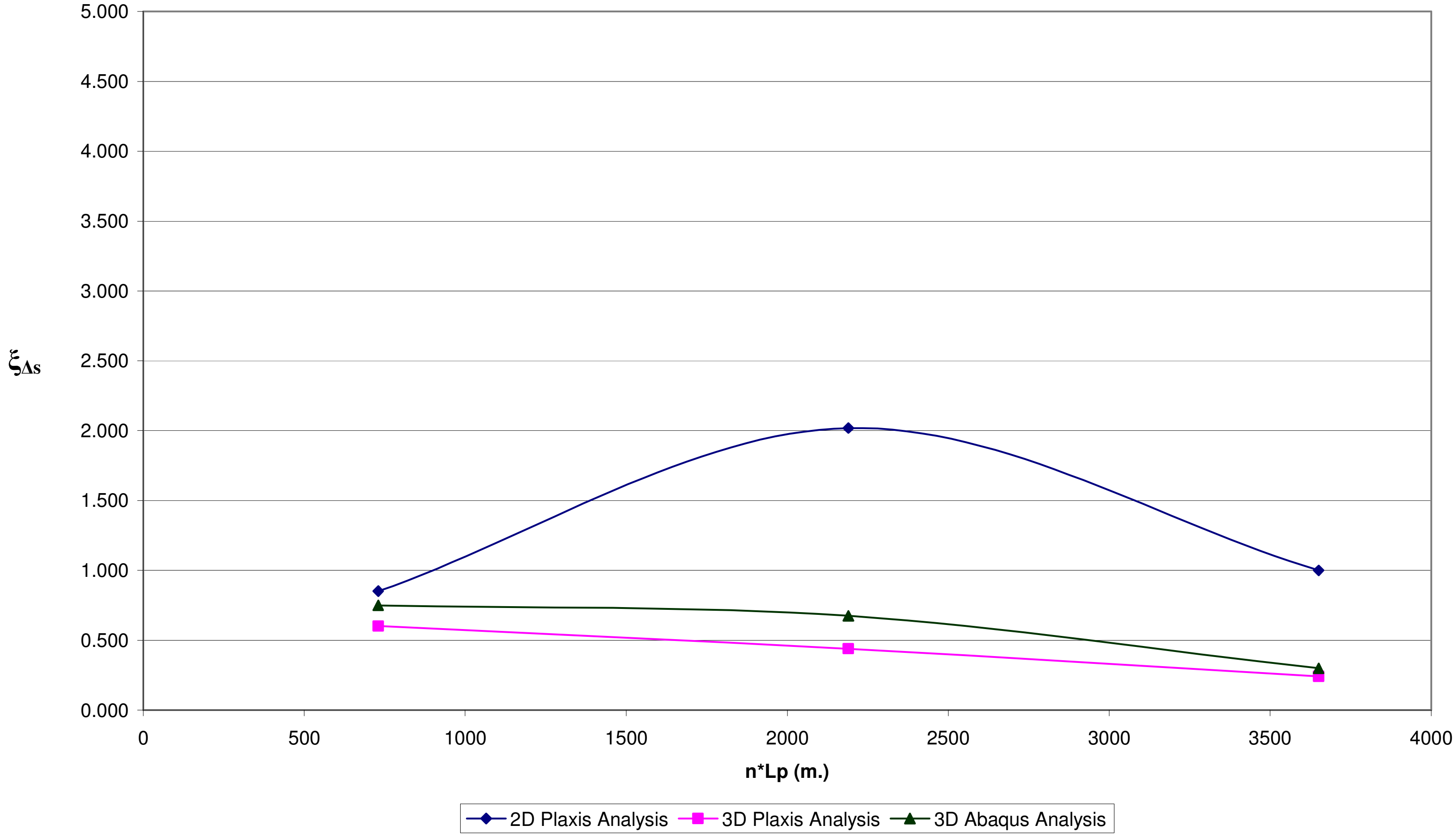

Fig D.11 $\xi_{\Delta s}$ vs. n^*L_p for Pile Configuration – 3, n=73 & V_{ult} / P_{eff} = 5

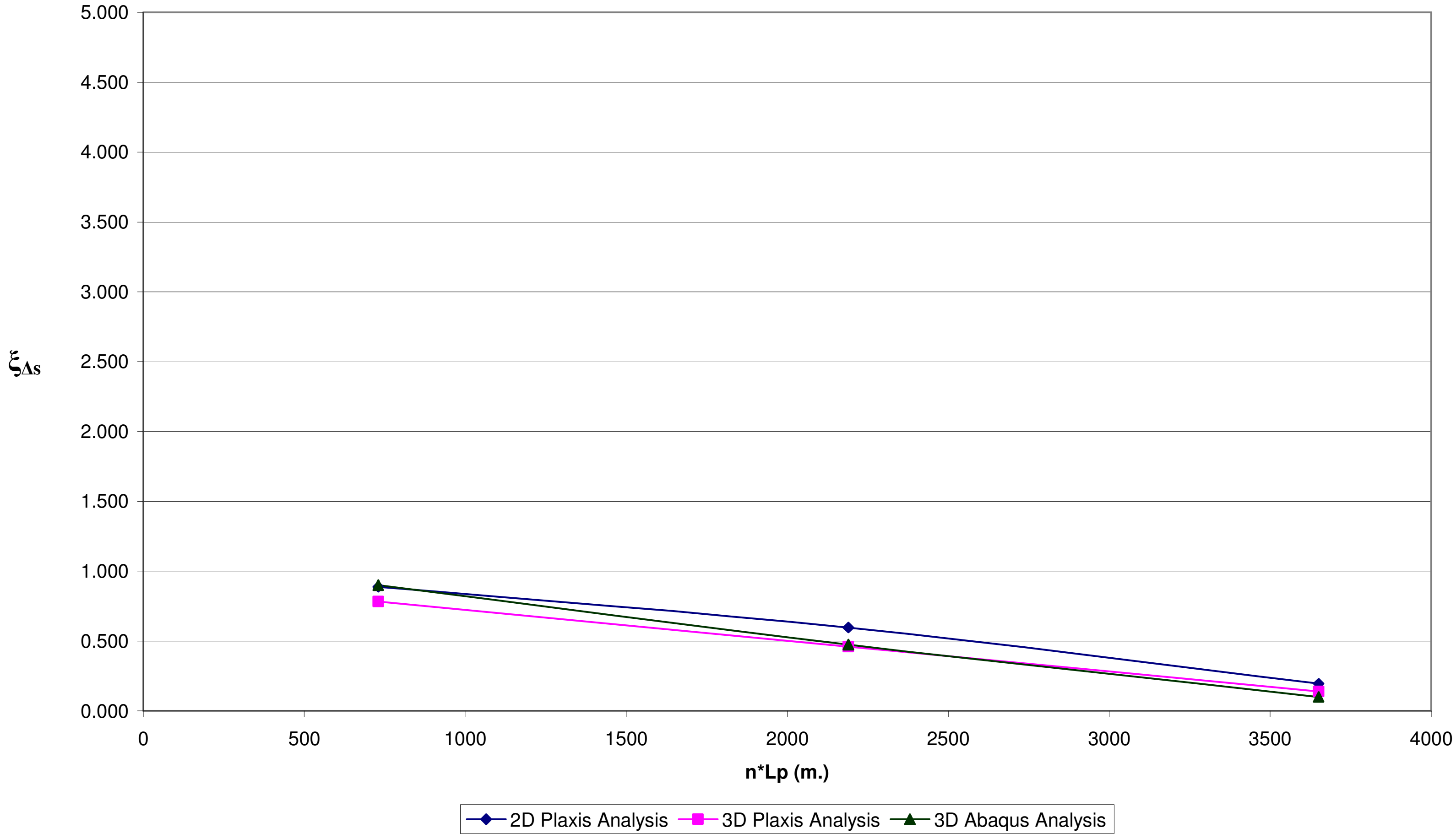

Fig D.12 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 3, n=73 & V_{ult} / P_{eff} = 20

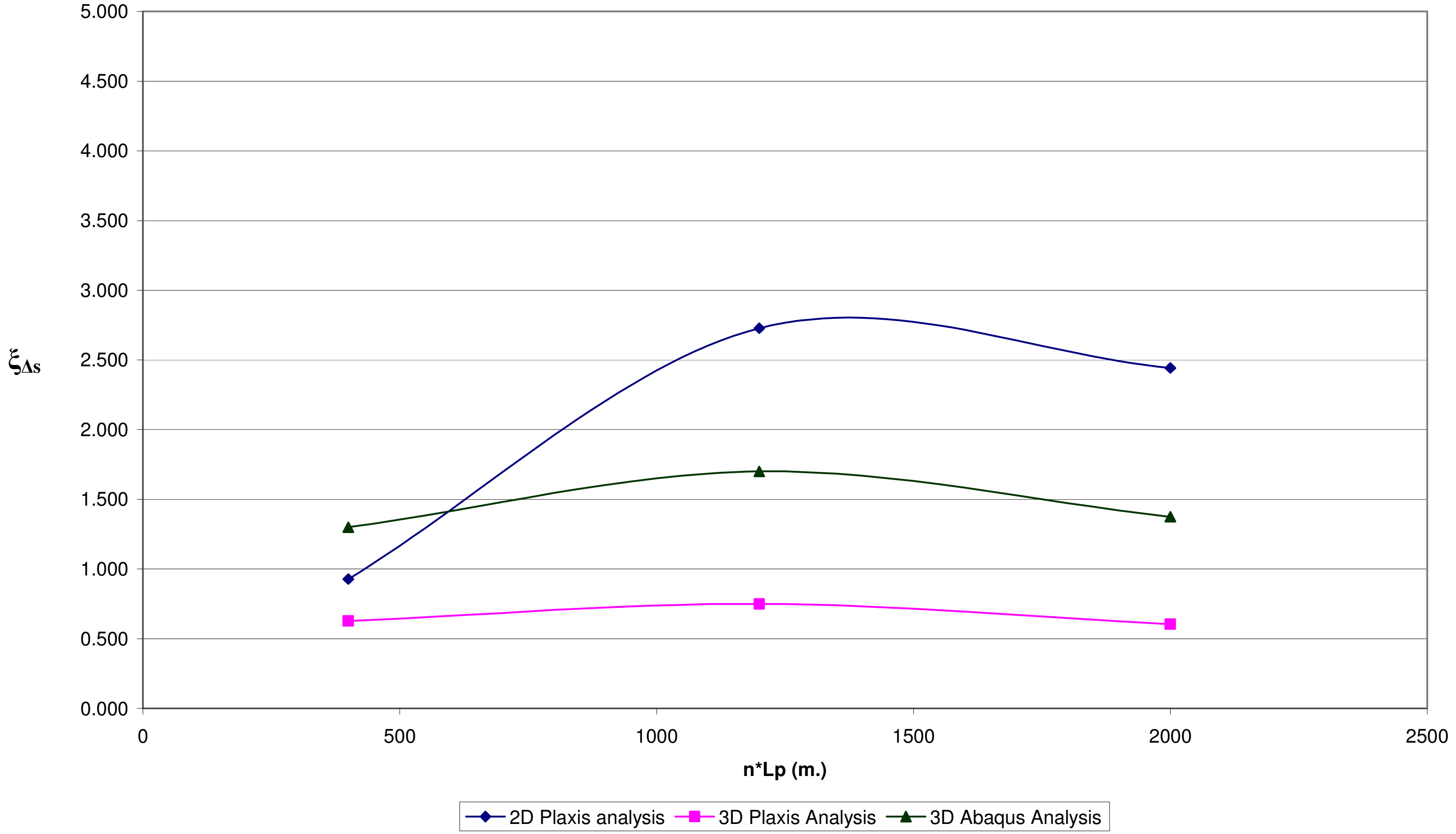

Fig D.13 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 3, n=40 & V_{ult} / P_{eff} = 5

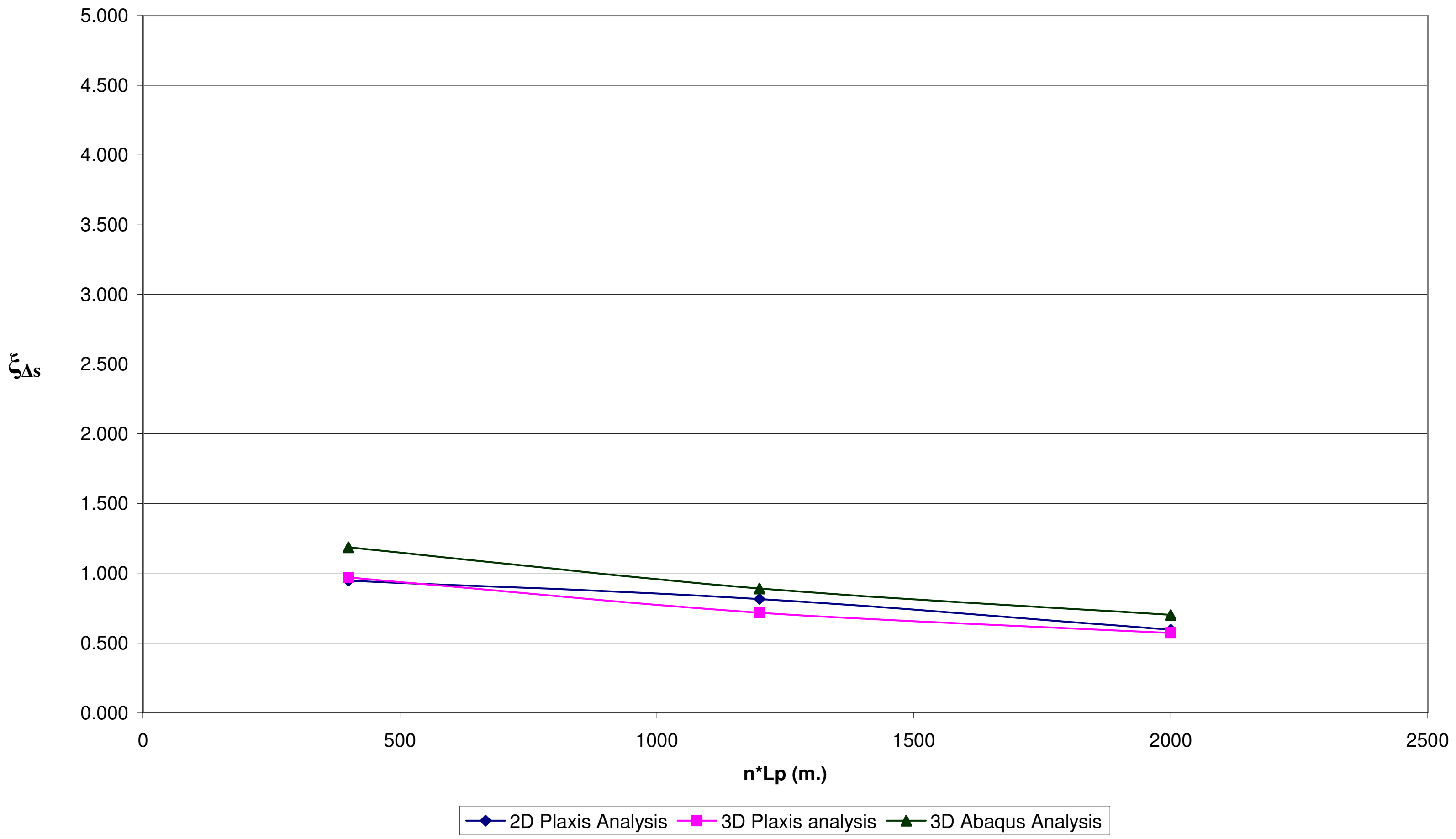

Fig D.14 $\xi_{\Delta s}$ vs. n*L_p for Pile Configuration – 3, n=40 & V_{ult} / P_{eff} = 20

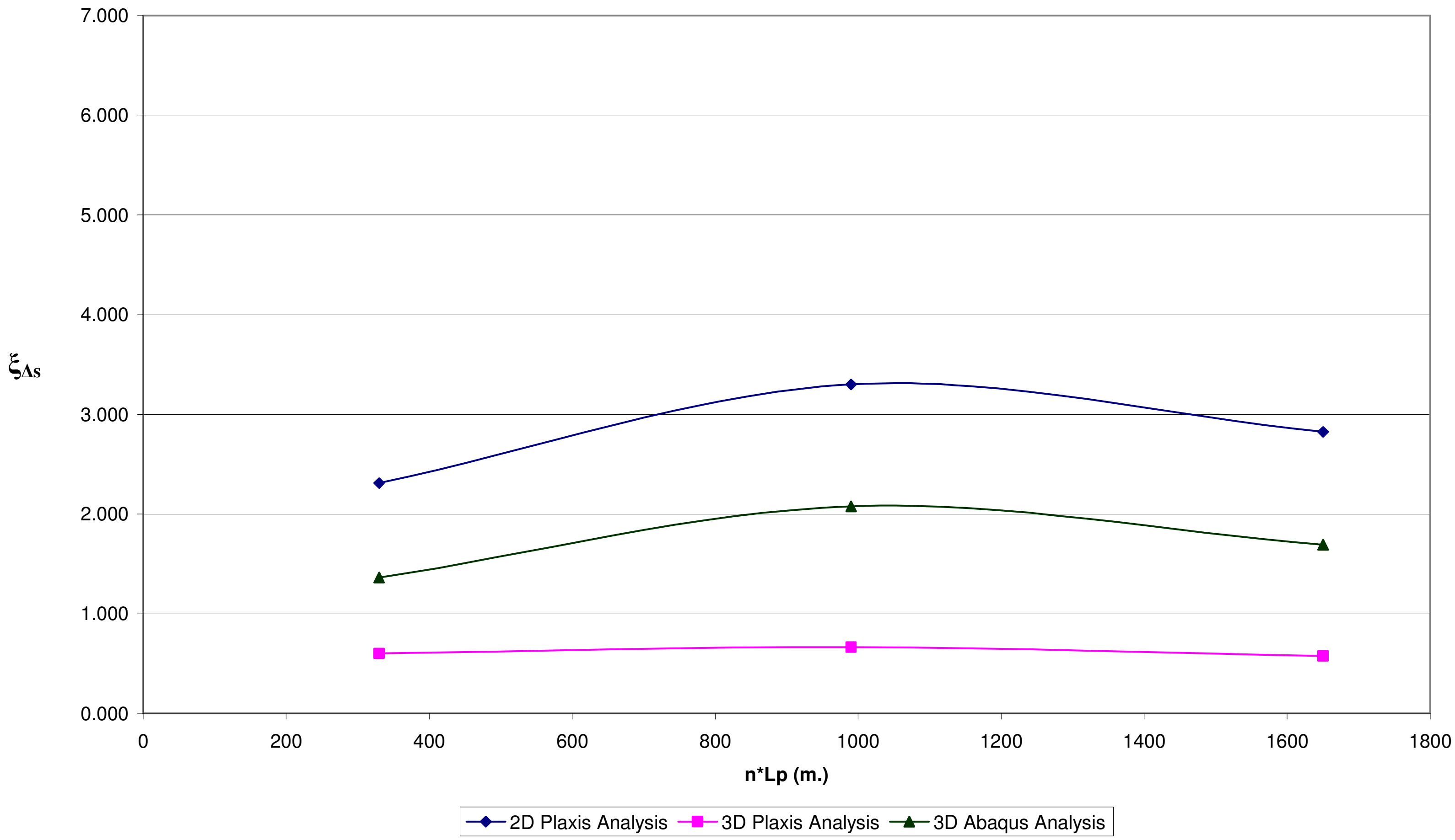

Fig D.15 $\xi_{\Delta s}$ vs. n^*L_p for Pile Configuration – 3, n=33 & V_{ult} / P_{eff} = 5

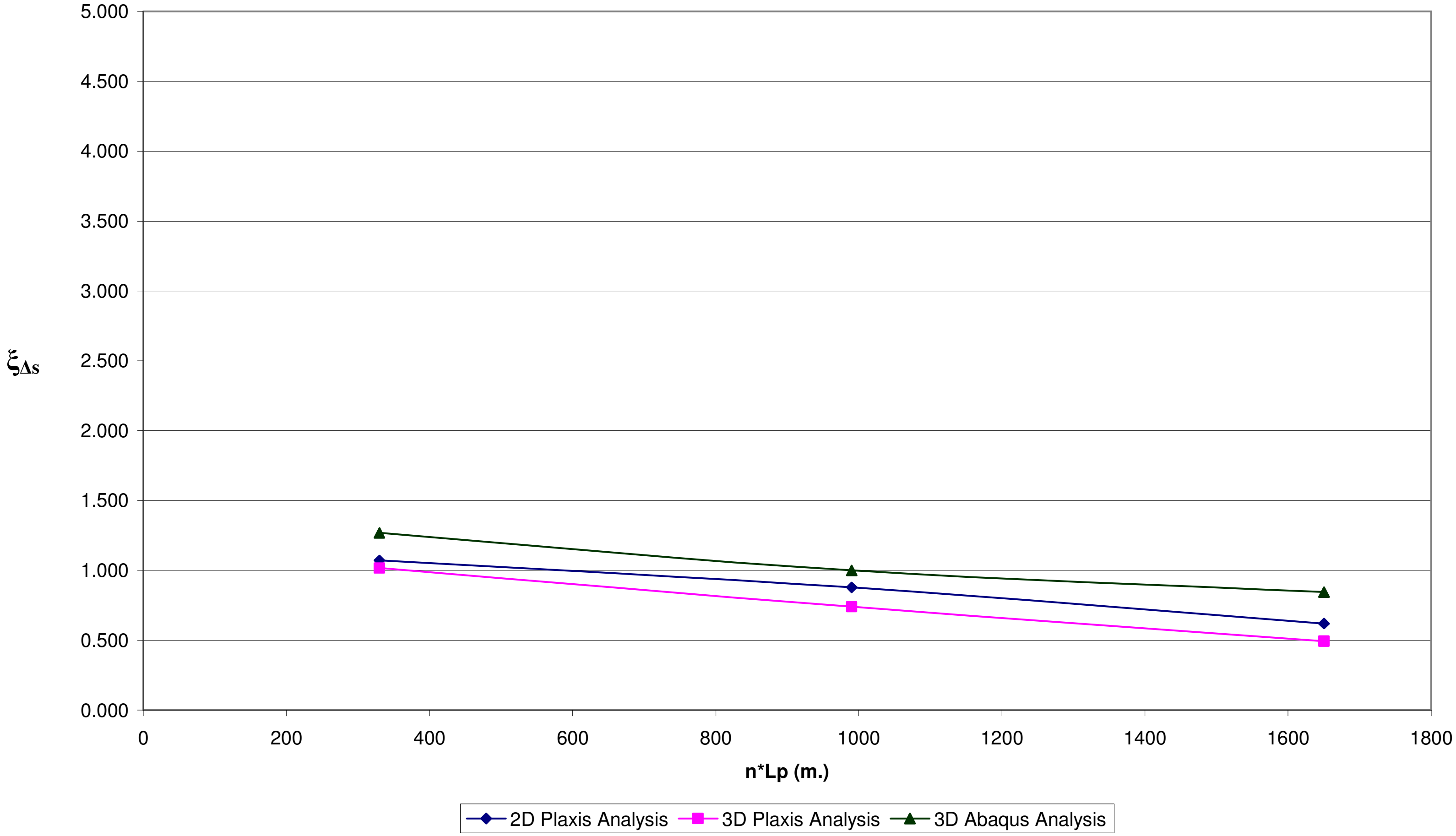

Fig D.16 $\xi_{\Delta s}$ vs. n^*L_p for Pile Configuration – 3, n=33, V_{ult}/ P_{eff} = 20