SETTLEMENT OF PILED RAFTS:
A CRITICAL REVIEW OF
THE CASE HISTORIES AND CALCULATION METHODS

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Approval of the Graduate School of Natural and Applied Sciences

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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ABSTRACT

SETTLEMENT OF PILED RAFTS:

A CRITICAL REVIEW OF

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M.S. Thesis, Department of Civil Engineering

Supervisor: Prof. Dr. M. Ufuk ERGUN

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In this study, settlement analysis of pile groups by hand calculation methods were investigated. Settlement ratio, equivalent pier, and equivalent raft methods were studied. Variations in some of the calculation methods were noted, and some suggestions were given.

More than thirty piled raft foundation case histories whose foundation and soil properties known have been found. The settlement of piled foundation in each case was solved by these methods. Results obtained from the calculations following different methods were presented for each case in the form of tables and graphs. Measured and calculated values were compared by making use of graphs and tables. Effect of type of piles was shown.

It was tried to find out that which method is suitable under different conditions. In conclusion, suggestions for methods and calculation procedures were given.

Keywords: Settlement ratio, equivalent pier, equivalent raft, settlement, pile raft foundation.

ÖZ

KAZIKLI RADYE TEMELLERİN OTURMASI: HESAP METODLARININ VE GERÇEK PROBLEMLERİN ELEŞTİREL YAKLAŞIMLA TEKRAR İNCELENMESİ

Neslihan SAĞLAM

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Bu çalışmada kazık guruplarının oturma analizlerinin elde çözüm metodları incelenmiştir. Oturma oranı, eşdeğer ayak, eşdeğer radye metodları çalışılmıştır. Bazı hesap yöntemlerindeki değişikliklere dikkat çekilmiş ve önerilerde bulunulmuştur.

Otuzun üzerinde, zemin ve temel özellikleri belirli kazıklı radye temel bulunmuştur. Her bir kazıklı temelin oturması bu metodlarla çözülmüştür. Her bir durum için sonuç tabloları ve grafikler hazırlanmıştır. Farklı yöntemlerle elde edilen neticeler her temel için tablo ve grafikler ile sunulmuştur. Bu tablo ve

grafikler kullanılarak, ölçülen ve hesaplanan değerler karşılaştırılmıştır. Kazık tipinin etkileri de gösterilmiştir.

Farklı durumlar için hangi metodun uygun olduğu bulunmaya çalışılmıştır. Sonuç olarak değişik tipteki hesap yöntemleri hakkında önerilerde bulunulmuştur.

Anahtar kelimeler: Otuma oranı, eşdeğer ayak, eşdeğer radye, oturma, kazıklı radye temel.

To My Mother

ACKNOWLEDGEMENTS

I would like to express my deepest gratitude to Prof. Dr. M. Ufuk ERGUN for his supervision, guidance and encouragement.

A very special word of thanks goes to my mother Perihan Sağlam and my family for their great support and patience.

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LIST OF SYMBOLS

LATIN SYMBOLS

A_G plan area of pile group

A_P total cross-sectional area of the piles in the group

B overall width of the group

D depth of foundation

d pile diameter

d_e equivalent pier diameter

E_b soil modulus of bearing stratum

E_d modulus of deformation at (H+D) level

E_e equivalent pier modulus

E_f modulus of deformation at foundation level

E_p pile modulus

E_s' drained soil modulus

E_u undrained soil modulus

G_l soil shear modulus at the level of pile base

$G_{l/2}$ soil shear modulus at the l/2 level

G_b soil shear modulus below the level of pile base

H thickness of the soil layer

I_p' influence factor for equivalent raft method

I_δ influence factor for equivalent pier method

k	stiffness of a single pile
K	stiffness of pile group
L	overall length of the group for equivalent raft method
	Pile length for settlement ratio and equivalent pier method
L_e	equivalent pier length
μ_d	depth factor
μ_g	geological factor
m_υ	coefficient of volume compressibility
n	number of piles in the group
P	load
P_b	base load
P_s	shaft load
P_t	total load
q_n	net foundation pressure
R_A	area ratio
r_b	radius of pile base
r_m	maximum radius
r_0	pile radius
R_s	settlement ratio
s	pile spacing
w_b	base settlement
w_s	shaft settlement
w_t	pile head settlement
z	depth

GREEK SYMBOLS

ζ	measure of radius of influence of pile
η	ratio of underream for underreamed piles
λ	pile-soil stiffness ratio
ξ	ratio of end-bearing for end-bearing piles
ξ_h	correction factor for effect of finite layer
ξ_υ	correction factor for effect of Poisson's ratio
ρ	variation of soil modulus with depth
υ_s	Poisson's ratio for drained conditions
υ_u	Poisson's ratio for undrained conditions
η_w	efficiency of pile group
δ	settlement
δ_i	immediate settlement
δ_c	consolidation settlement
δ_{oed}	oedometer settlement
σ_z	average effective vertical stress
μ_0	influence factor related to the depth of the equivalent raft
μ_1	influence factor related to the thickness of the compressible soil layer

CHAPTER 1

INTRODUCTION

Several techniques have been proposed for analyzing the settlement of pile groups. These techniques can usually be classified into one of the following three categories.

a. Estimates of settlement of pile groups are based on purely emprical data. Among the emprical approaches are those for groups in sand proposed by Skempton (in Poulos 1980) on the basis of limited number of field observations. Meyerhof (in Poulos 1980) suggests a method for a square group for driven piles and displacement caissons in sand.

b. Simplified techniques which reduce a pile group to an equivalent simpler form of foundation for analysis purposes.

Simplified procedures, which reduce a group to an equivalent raft are used. There are variations in the suggested procedures (Tomlinson (1986), Ordemir (1984)). The depth at which the equivalent raft is located depends on the nature of the soil profile.

Simplified methods which reduce the group to an equivalent pier are suggested by Poulos and Davis (1980), Poulos (1993). Two types of approximations may be made:

1. An equivalent single pier of the same circumscribed plan area as the group and of some equivalent length, L_e.

2. An equivalent single pier of the same length, L, as the piles, but having an equivalent diameter, d_e.

In the so called settlement ratio method, the settlemet of a single pile at the average load level is multiplied by settlement ratio R_s to calculate group settlement. The interaction factor approach can be used to derive theoretical values of R_s, and some values of R_s so derived are tabulated (Poulos and Davis (1980)). Randolph, and Fleming et al. (1992), has developed a very useful approximation for R_s. (Poulos (1989), Fleming et al. (1992))

c. Analytical methods which consider interaction between the piles and surrounding soil.

Methods which compute the response of a single pile and which consider pile-soil-pile interaction via interaction factors make use of some form of elastic theory (Poulos 1968, Randolph and Wroth 1979). The analysis is based on elastic soil characterized by shear modulus G which may vary with depth and a Poisson’s ratio υ. To analyze the settlement behaviour of a general pile group, superposition of the two-pile interaction factors may be employed.

Finite element method is a powerful analytical tool that can be used in settlement analyses. Non-linear soil behaviour can be modelled. Also the complete history of the pile can be simulated, i.e. the processes of installation, reconsolidation of the soil following installation, and subsequent loading of the pile. Such analyses are valuable in leading to a better understanding of the details of pile behaviour, but are unlikely to be readily applicable to practical piling problems because of their complexity and the considerable number of geotechnical parameters required (e.g. Ottoviani 1975).

Complete boundary element method, in which each pile is divided into discrete elements and pile-soil-pile interaction is considered between each of these elements is another way of analyzing settlement of pile foundation (Poulos and Davis (1980)). The boundary element methods are more economical than the finite element method in pile group analysis, but these methods require double integration of analytical point load solution that may be cumbersome and relatively time-consuming.

A modification of complete boundary element analysis, "the hybrid method", has been developed by Chow (1986) and Lee (1993). Here, a load transfer analysis is used to determine the response of a single pile, and continuum theory is employed to determine the influence of adjacent piles on this response.

This study is focused on the simpler methods namely settlement ratio, equivalent pier, and equivalent raft methods. Over thirty case histories are studied to examine them thoroughly and some suggestions are given about the use of these methods.

In Chapter 2 the simpler methods are reviewed in some detail. Case histories are presented in Chapter 3 and Appendix. All parameters used and calculations are summarized in each case. Settlement ratio, equivalent pier and equivalent raft solutions are made for all cases.

Conclusions reached and obversations made in the calculation of settlement of piled raft foundations are given in a compact form in Chapter 4.

CHAPTER 2

SIMPLIFIED DESIGN METHODS

2.1. Settlement Ratio Method

A convenient way of regarding the effects of interaction within a pile group has been suggested by Butterfield and Douglas (in Fleming, W. et al, 1992). The stiffness , K, of the pile group may be expressed as fraction η_w of the sum of the individual pile stiffness, k. Thus for a group of piles (n: number of piles),

$$K = \eta_w \, n \, k \qquad \ldots(2.1)$$

The factor η_w is the inverse of the settlement ratio, R_s, and may be thought of as an efficiency. For no interaction between piles, η_w would equal unity. The efficiency may be written as

$$\eta_w = n^{-e} \qquad \ldots(2.2)$$

Where the exponent e will lie between 0.4 and 0.6 for most pile groups (Poulos (1993)). The actual value of e will depend on

pile slenderness ratio, L/d (pile length/pile diameter)

pile stiffness ratio, $\lambda = E_p/G_l$ (pile modulus/soil shear modulus)

pile spacing ratio, s/d (pile spacing/pile diameter)

homogeneity of soil, characterised by ρ,

Poisson's ratio, υ

For a given combination of the above factors, the value of e may be estimated using the curves shown in Figure 2.1 (Fleming et al, 1992). The upper part of the figure allows a base of e to be chosen, depending on pile slenderness ratio (assuming λ=1000, s/d=3, ρ=0.75, υ =0.3). The four curves in lower part of the figure then modify this basic value of pile stiffness ratio, s, υ and ρ.

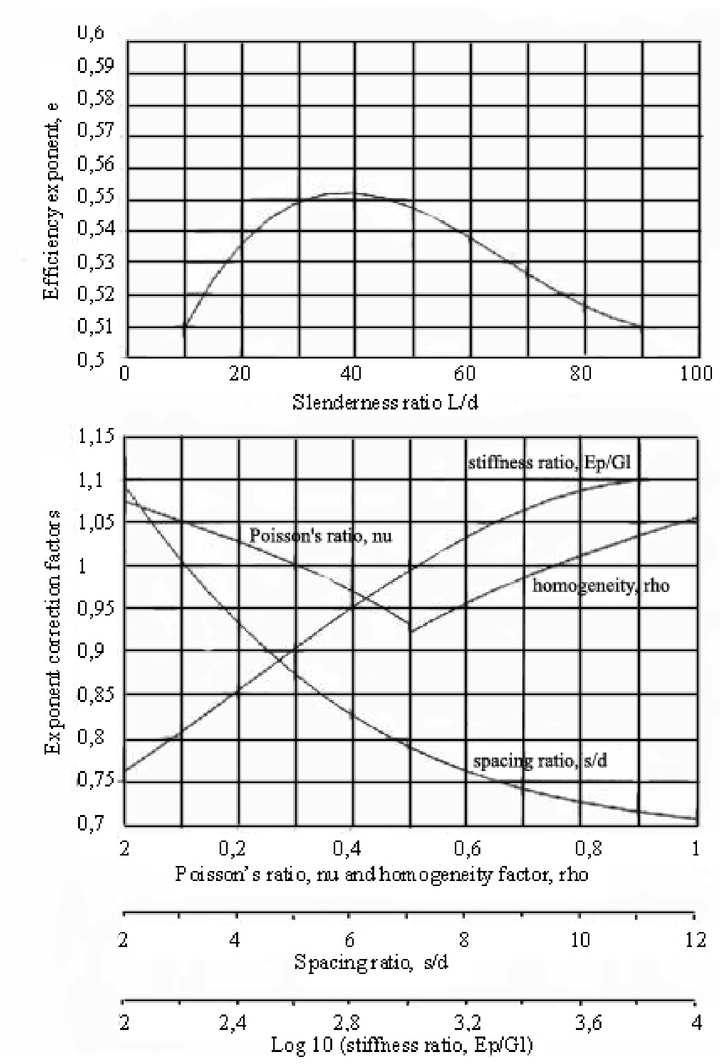

Figure 2.1: Charts for calculation of exponent e for efficiency of pile groups. (Fleming, et al, 1992)

The base settlement and shaft settlement will be similar to the settlement of pile head, w_t for a single stiff pile. The total load, P_t, may thus be written as

$$P_t = P_b + P_s = w_t \left(\frac{P_b}{w_b} + \frac{P_s}{w_s} \right) \qquad \ldots(2.3)$$

In developing a general solution for the axial response of a pile, it is convenient to introduce a dimensionless load settlement ratio for the pile. The stiffness is P_t/w_t and this may be made dimensionless by dividing by the radius of the pile and an appropriate soil modulus. It has been customary to use the value of soil modulus at the level of pile base for this purpose, written as G_l. Thus equation becomes

$$\frac{P_t}{w_t\, r_0\, G_l} = \frac{4\, r_b\, G_b}{(1-\upsilon)\, r_0\, G_l} + \frac{2\Pi G_{l/2}}{G_l}\ \frac{L}{r_0} \qquad \ldots(2.4)$$

The shear modulus variation with depth may de idealized as linear, according to $G=G_0+mz$ (where z is depth), with the possibility of sharp rise to G_b below the level of pile base (Figure 2.2) (Fleming, W.G. et al, (1992)). Defining parameters $\rho=G_{l/2}/G_l$ and $\xi=G_l/G_b$, the constant ζ has been found to fit the expressions (Randolph and Wroth, (1978))

$$\zeta = \ln \{[0.25+(2.5\rho(1-\upsilon)-0.25)\ \xi]L/r_0\} \qquad \ldots(2.5)$$

$$\zeta = \ln [2.5\rho(1-\upsilon)L/r_0] \text{ for } \xi = 1 \qquad \ldots(2.6)$$

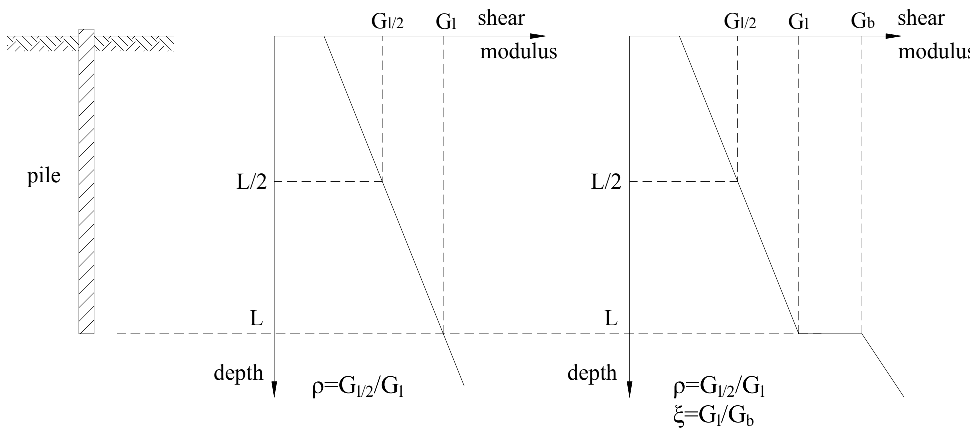

Figure 2.2: Assumed variation of soil shear modulus with depth

Substituting in the appropriate boundary conditions at the pile base yields an expression for load settlement ratio of the pile head of

$$\frac{P_t}{G_l r_0 w_t} = \frac{\dfrac{4\eta}{(1-\upsilon)\xi} + \dfrac{2\Pi\rho}{\zeta}\,\dfrac{\tanh(\mu L)}{\mu L}\,\dfrac{L}{r_0}}{1 + \dfrac{4\eta}{\Pi\lambda\,(1-\upsilon)\xi}\,\dfrac{\tanh(\mu L)}{\mu L}\,\dfrac{L}{r_0}} \qquad \ldots(2.7)$$

where, summarizing the various dimensionless parameters, (Randolph 1994, Birand 2001)

$\eta = r_b/r_0$	(ratio of underreamed for underreamed piles)
$\xi = G_l/G_b$	(ratio of end-bearing for end-bearing piles)
$\lambda = E_p/G_l$	(pile-soil stiffness ratio)
$\zeta = \ln(r_m/r_0)$	(measure of radius of influence of pile)
$\mu = (2/(\zeta\lambda))^{0,5} L/r_0$	(measure of pile compressibility)

Finally settlement of a group pile can be calculated as

$$\delta_{group} = \frac{P_{group}}{K} \qquad \ldots(2.8)$$

…(1.

where $K = \eta_w \, k \, n$

$k = P_t / w_t$

or

$$\delta_{group} = \delta_{single} R_s \qquad \ldots(2.9)$$

where $\delta_{single} = P_{single} / k$

$R_s = n^e$

The effect of different layers of soil over the depth of penetration of the piles in a group may generally be dealt with adequately by adopting suitable values of the average shear modulus for the soil, and a value for the homogeneity factor, ρ, which reflects the general trend of stiffness variation with depth. However particular attention needs to be paid to the case where a soft layer of soil occurs at some depth beneath the pile group, as shown in Figure 2.3 (Fleming, W.G. et al, (1992)). In assessing how much additional settlement may occur due to the presence of soft layer, the average change in vertical stress caused by the pile group must be estimated.

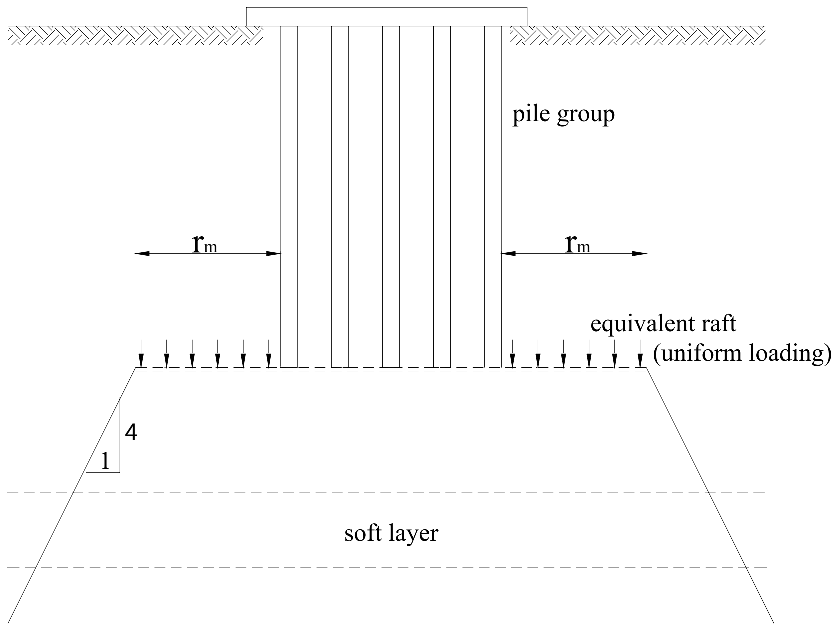

Figure 2.3: Use of equivalent raft for calculating effect of soft layer underlying pile group

Implicit in the solution for the load settlement response of a single pile is the idea of the transfer of the applied load, by means of induced shear stressess in the soil, over a region of radius r_m (Randolph and Wroth, (1978)). The average vertical stress applied to the soil at the level of the base of a group of a piles may be estimated by taking the overall applied load and distributing it over the area of the group augmented by this amount, as shown in Figure 2.3. Below the level of the pile bases, the spread of the area over which the load assumed to be shared may be taken as the usual rate of 1:4 (Tomlinson 1986)

$$r_m = [\ 0.25 + (\ 2.5\ \rho\ (\ 1\text{-}\upsilon\ ) - 0.25\ )\ \xi\ ]\ L \qquad \ldots(2.10)$$

The interaction factor approach can be used to derive theoretical values of R_s. Table 2.2 (Poulos and Davis (1980)) shows the theoretical values of R_s, for floating-pile groups in a deep layer of uniform soil, and in Table 2.3 (Poulos and Davis (1980)) for pile groups bearing on rigid stratum. These values apply to square groups of piles with a rigid cap in which the center-to-center spacing between adjacent piles in a row is s, and the length and diameter of each pile are L and d, respectively. The pile stiffness factor is K. K is defined as

$$K = \frac{E_p}{E_s} R_A \qquad \text{...(2.11)}$$

where $R_A = A_p/(\pi d^2/4)$ (Ratio of area of pile section A_p to area bounded by outer circumference of pile) (Poulos and Davis (1980))

Average values of pile-stiffness factor K, calculated for various types of pile and soil, are given in Table 2.1 (Poulos and Davis (1980)).

Table 2.1: Average values of K for solid piles (Poulos and Davis, 1980)

Soil Type	Pile Material		
	Steel	Concrete	Timber
Soft clay	60.000	6.000	3.000
Medium clay	20.000	2.000	1.000
Stiff clay	3.000	300	150
Loose sand	15.000	1.500	750
Dense sand	5.000	500	250

Table 2.2: Theoretical Values of Settlement Ratio R_s Friction Pile Groups, with Rigid Cap, in Deep Uniform Soil Mass (Poulos and Davis, 1980)

No of piles in group			4				9				16				25			
L/d	s/d	K	10	100	1000	∞	10	100	1000	∞	10	100	1000	∞	10	100	1000	∞
	2		1,83	2,25	2,54	2,62	2,78	3,80	4,42	4,48	3,76	5,49	6,40	6,53	4,75	7,20	8,48	8,68
10	5		1,40	1,73	1,88	1,90	1,83	2,49	2,82	2,85	2,26	3,25	3,74	3,82	2,68	3,98	4,70	4,75
	10		1,21	1,39	1,48	1,50	1,42	1,76	1,97	1,99	1,63	2,14	2,46	2,46	1,85	2,53	2,95	2,95
	2		1,99	2,14	2,65	2,87	3,01	3,64	4,84	5,29	4,22	5,38	7,44	8,10	5,40	7,25	9,28	11,25
25	5		1,47	1,74	2,09	2,19	1,98	2,61	3,48	3,74	2,46	3,54	4,96	5,34	2,95	4,48	6,50	7,03
	10		1,25	1,46	1,74	1,78	1,49	1,95	2,57	2,73	1,74	2,46	3,42	3,63	1,98	2,98	4,28	4,50
	2		2,43	2,31	2,56	3,01	3,91	3,79	4,52	5,66	5,58	5,65	7,05	8,94	7,26	7,65	9,91	12,66
50	5		1,73	1,81	2,10	2,44	2,46	2,75	3,51	4,29	3,16	3,72	5,11	6,37	3,88	4,74	6,64	8,67
	10		1,38	1,50	1,78	2,04	1,74	2,04	2,72	3,29	2,08	2,59	3,73	4,65	2,49	3,16	4,76	6,04
	2		2,56	2,31	2,26	3,16	4,43	4,05	4,11	6,15	6,42	6,14	6,50	9,92	8,48	8,40	10,25	14,35
100	5		1,88	1,88	2,01	2,64	2,80	2,94	3,38	4,87	3,74	4,05	4,98	7,54	4,68	5,18	6,75	10,55
	10		1,47	1,56	1,76	2,28	1,95	2,17	2,73	3,93	2,45	2,80	3,81	5,82	2,95	3,48	5,00	7,88

Table 2.3: Theoretical Values of Settlement Ratio R_s End-Bearing Pile Gr., with Rigid Cap, Bearing on a Rigid Stratum (Poulos and Davis,1980)

No of piles in group		4				9				16				25			
L/d	s/d K	10	100	1000	∞	10	100	1000	∞	10	100	1000	∞	10	100	1000	∞
	2	1,52	1,14	1,00	1,00	2,02	1,31	1,00	1,00	2,38	1,49	1,00	1,00	2,70	1,63	1,00	1,00
10	5	1,15	1,08	1,00	1,00	1,23	1,12	1,02	1,00	1,30	1,14	1,02	1,00	1,33	1,15	1,03	1,00
	10	1,02	1,01	1,00	1,00	1,04	1,02	1,00	1,00	1,04	1,02	1,00	1,00	1,03	1,02	1,00	1,00
	2	1,88	1,62	1,05	1,00	2,84	2,57	1,16	1,00	3,70	3,28	1,33	1,00	4,48	4,13	1,50	1,00
25	5	1,36	1,36	1,08	1,00	1,67	1,70	1,16	1,00	1,94	2,00	1,23	1,00	2,15	2,23	1,28	1,00
	10	1,14	1,15	1,04	1,00	1,23	1,26	1,06	1,00	1,30	1,33	1,07	1,00	1,33	1,38	1,08	1,00
	2	2,49	2,24	1,59	1,00	4,06	3,59	1,96	1,00	5,83	5,27	2,63	1,00	7,62	7,06	3,41	1,00
50	5	1,78	1,73	1,32	1,00	2,56	2,56	1,72	1,00	3,28	3,38	2,16	1,00	4,04	4,23	2,63	1,00
	10	1,39	1,43	1,21	1,00	1,78	1,87	1,46	1,00	2,20	2,29	1,71	1,00	2,62	2,71	1,97	1,00
	2	2,54	2,26	1,81	1,00	4,40	3,95	3,04	1,00	6,24	5,89	4,61	1,00	8,18	7,93	6,40	1,00
100	5	1,85	1,84	1,67	1,00	2,71	2,77	2,52	1,00	3,54	3,74	3,47	1,00	4,33	4,68	4,45	1,00
	10	1,44	1,44	1,46	1,00	1,84	1,99	1,98	1,00	2,21	2,48	2,53	1,00	2,53	2,98	3,10	1,00

R_s values for other numbers of piles may be interpolated from Table 2.2 and 2.3. For groups containing more than 16 piles, it has been found that R_s varies approximately linearly with the square root of the number of piles in the group. Thus, for a given value of pile spacing, K and L/d, R_s may be extrapolated from the values for a 16-pile group and a 25-pile group as follows:

$$R_s = (R_{25}-R_{16})\,(n^{0.5}-5)+R_{25} \qquad \ldots(2.12)$$

where R_{25}:value of R_s for 25-pile group

R_{16}: value of R_s for 16-pile group

n: number of piles in group

For floating pile groups, the underlying rigid base below the soil layer tends to reduce the settlement ratio R_s. An indication of the extent of this decrease is given in Figure 2.4 (Poulos and Davis (1980)), in which, for typical groups, a reduction coefficient, ξ_h, is plotted against the ratio of layer depth h to pile-length L, ξ_h being defined as,

$\xi_h = R_s$ for finite layer of depth/R_s for infinitely deep layer

The effect of finite layer is more pronounced as the size of the group increases. As L/d increases, the effect of the finite layer becomes less significant.

As the relative stiffness of the bearing stratum E_b/E_s (modulus of bearing stratum/modulus of the soil along the pile shaft) increases R_s decreases, this effect being most pronounced for shorter stiffer piles. For slender piles (e.g. L/d =100) unless the piles are quite stiff (K>1000), the bearing stratum has little effect on settlements, because little load reaches the pile tip under normal working load conditions.

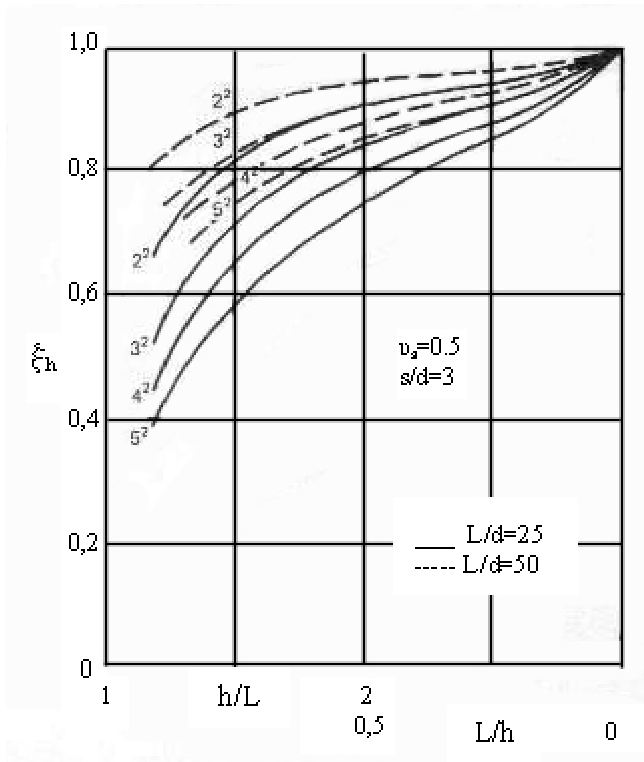

Figure 2.4: Reduction coefficient ξ_h for effect of finite layer (Poulos and Davis, 1980)

The effect of υ_s on R_s is shown in Figure2.5 (Poulos and Davis (1980)), in which factor ξ_υ is plotted for a typical cases, ξ_υ being defined as

$$\xi_\upsilon = R_s \text{ for specified value of } \upsilon_s/R_s \text{ for } \upsilon=0.5$$

The effect of υ_s becomes more pronounced as the number of piles in the group increases.

Figure 2.6 (Poulos and Davis (1980)) shows the effect of the distribution of soil modulus on R_s for typical case. Larger values of R_s occur for the uniform soil, the difference becoming greater as the number of piles increases.

As the spacing increases , the pile cap has an increasing effect, but for practical pile spacing, the influence of the cap appears to be negligible.

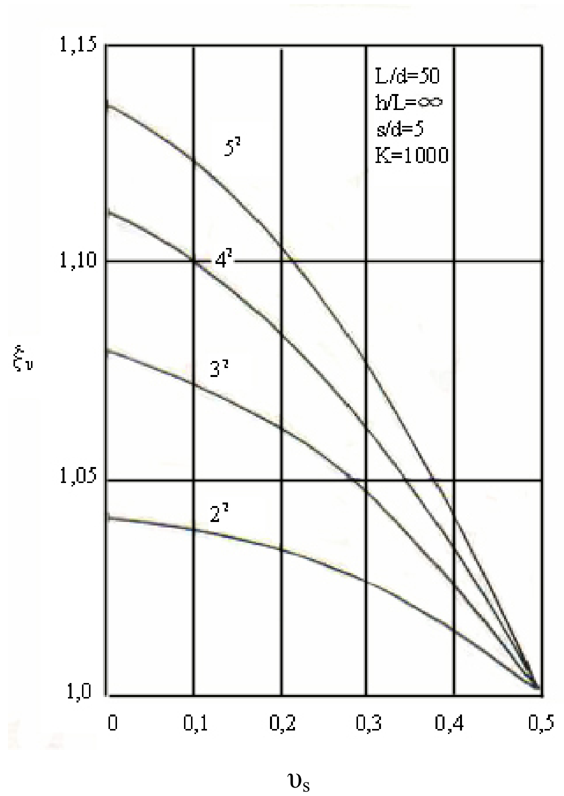

Figure 2.5: Correction factor ξ_υ for effect of υ_s (Poulos and Davis, 1980)

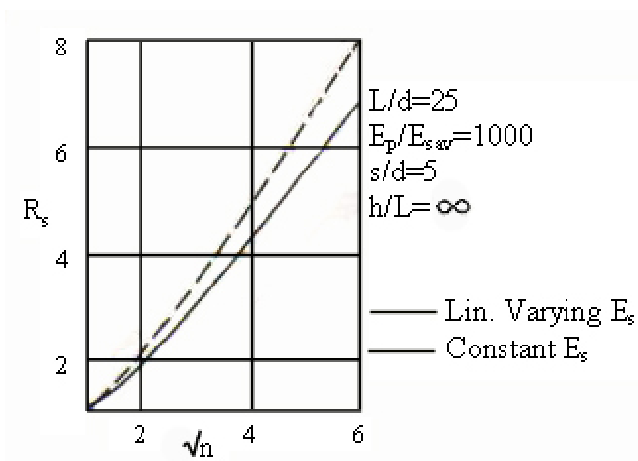

Figure 2.6: Effect of distribution of E_s on settlement ratio (Poulos and Davis, 1980)

2.2. Equivalent Pier Method

This method has been suggested by Poulos and Davis (1980) and illustrated in Figure2.7. (Poulos (1993)). The pile group is replaced by a single pier of equivalent diameter, d_e (or length, L_e) and equivalent stiffness.

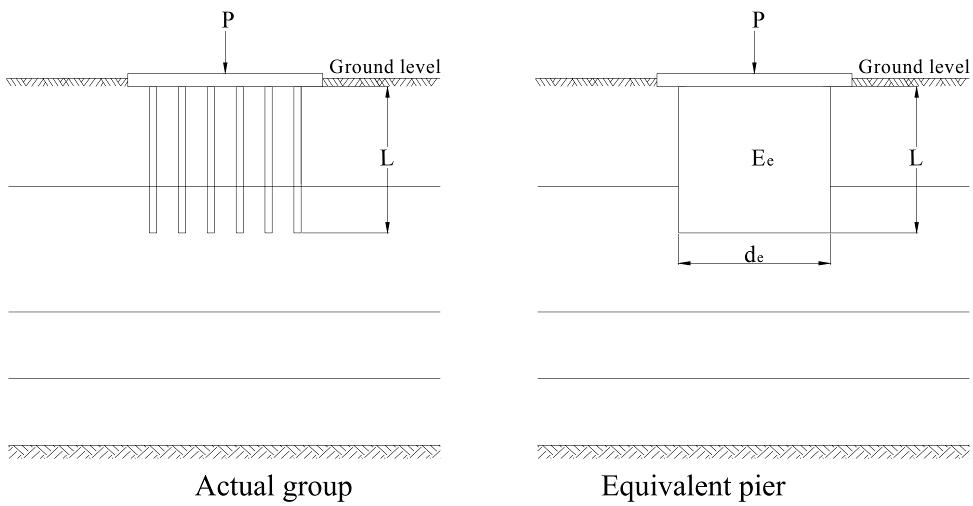

Figure2.7: Equivalent pier concept

L_e is prefered for incompressible floating groups. d_e is more appropriate when the piles pass through layered soils or founded on very different material. For incompressible floating groups, for most practical cases, L_e/L lies between 0.9 and 0.6. For layered soils;

For friction piles;

$$d_e=1.27A_G^{0.5} \quad \ldots(2.13)$$

For end-bearing piles;

$$d_e=1.13A_G^{0.5} \quad \ldots(2.14)$$

where A_G: Plan area of pile group. Poulos (1993), Randolph (1994)

The equivalent pier modulus, E_e, is approximated as;

$$E_e = E_p \frac{A_p}{A_G} + E_s \left(1 - \frac{A_p}{A_G}\right) \qquad \ldots(2.15)$$

Where E_p: Young's modulus of piles

E_s: average Young's modulus of soil within the group

A_p: total cross-sectional area of the piles in the group.

Having reduced the group to an equivalent pier, theoretical solutions for the settlement of a single pile may then be used to estimate the settlement (e.g. Randolph and Wroth, (1979); Poulos and Davis, (1980)).

For incompressible floating groups, values of L_e/L obtained by Poulos (1968), are shown in Figure 2.8. Poulos and Davis (1980). L_e/L depends both on spacing and L/d, but virtually independent of the number of piles in the group.

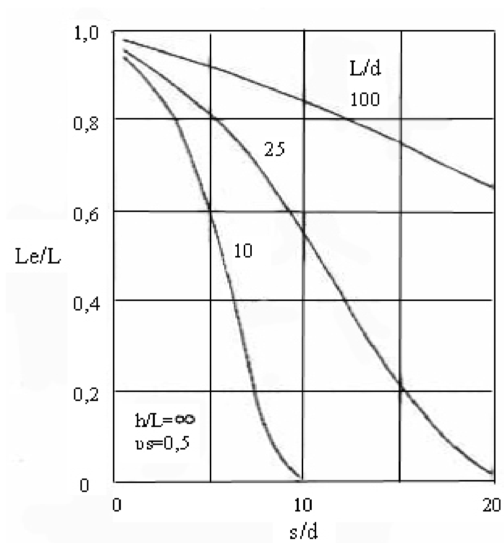

Figure 2.8: Equivalent length of single pier for same settlement as pile group (Poulos and Davis, 1980)

Relationships between d_e/B and s/d are plotted in Figure 2.9 (Poulos and Davis(1980)) for floating piles. B is the width of the raft. Like L_e/L, d_e/B is almost independent of the group's size, but it does depend on L/d. The ratio d_e/B tends to decrease with increasing pile compressibility. It should be noted that the equivalent pier in Figure 2.9 has the same value of pile stiffness factor, K (equation 2.11) as the pile in the group.

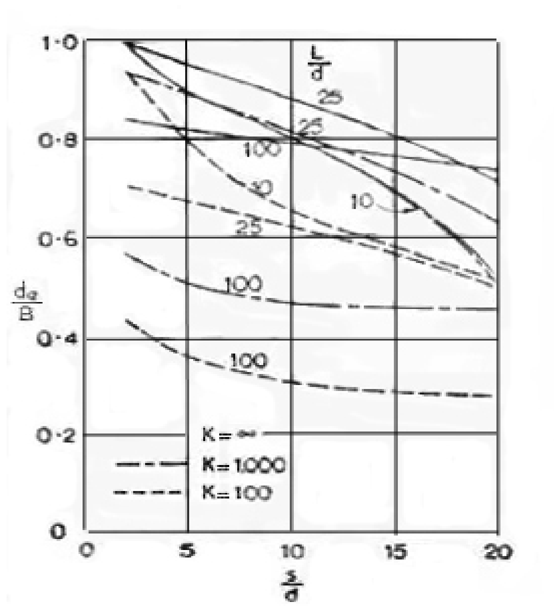

Figure 2.9: Diameter of equivalent pier to represent pile group (Poulos and Davis, 1980)

Figure 2.10 presents dimensionless solutions for a pier in a homogeneous soil, bearing on a stratum of equal or greater stiffness. The compresibility of the pier has been chosen to be representative of the average value of a pile and soil block with piles at spacing of about 3 diameters. For short piers, the relative compressibility is unimportant unless the pier is very compressible, or unless it is founded on a very stiff stratum. Figure 2.10 may be used with sufficient accuracy for a pier in non-homogeneous soil, by using an average soil modulus along the shaft of the pier (Poulos (1972)) .

The I_δ, displacement influence factor, depends on slenderness ratio, pile material, soil homogeneity and relative soil-pile stiffness which are given in equation 2.16 (Randolph and Wroth, (1978), (1979)).

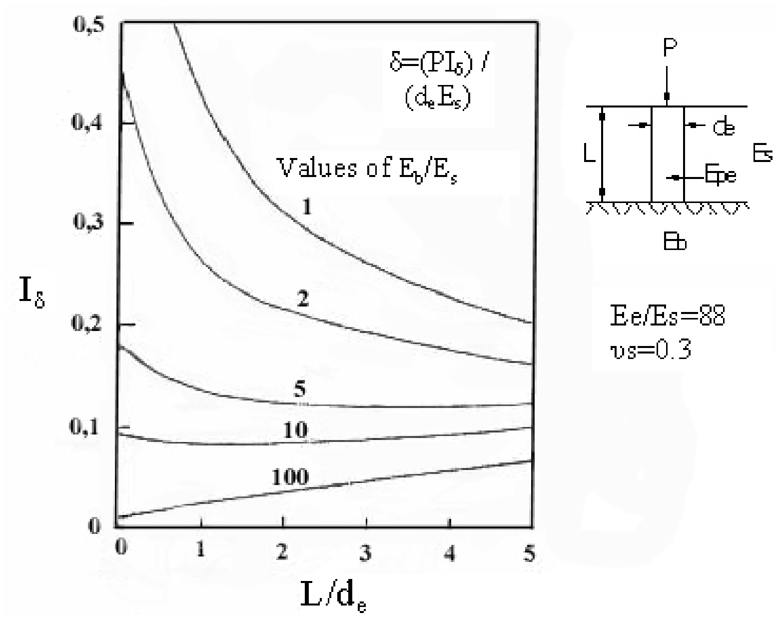

Figure 2.10: Settlement of equivalent pier in soil layer (Poulos, 1972)

$$I_\delta = 4(1+\upsilon_s)\frac{1 + \dfrac{1}{\Pi\lambda}\,\dfrac{8}{(1-\upsilon_s)}\,\dfrac{\eta}{\xi}\,\dfrac{\tanh(\mu L)}{\mu L}\,\dfrac{L}{d}}{\dfrac{4}{(1-\upsilon_s)}\,\dfrac{\eta}{\xi} + \dfrac{4\Pi\rho}{\zeta}\,\dfrac{\tanh(\mu L)}{\mu L}\,\dfrac{L}{d}} \qquad \ldots(2.16)$$

In deep homogeneous soil Randolph (1994) reported that, in order to improve the accuracy of equation 2.7 for relatively short and thick piers, the maximum radius of influence, r_m, should emprically be increased giving revised equation for ζ of (Horikoshi and Randolph (1999)):

$$\zeta = \ln[A+2,5(1-\nu)L_p/r_p] \quad (A=5, \text{ for small } L_p/r_p) \qquad \ldots(2.17)$$

Horikoshi (in Horikoshi and Randolph (1999)) discussed the applicability of equation 2.17 to piers in deep non-homogeneous soil where the soil modulus increase linearly with depth. He found that for piers installed in non-homogeneous soil, the following equation is suitable :

$$\zeta = \ln\{A+[0,25+(2,5\rho(1-\nu)-0,25)\xi]L_p/r_p\} \quad (A=5 \text{ for small } L_p/r_p) \qquad \ldots(2.18)$$

In practical cases in which the soil profile is layered and compressible strata are present below the piles, the settlement caused by these strata must be considered in calculating the overall settlement of the group. The settlement of compressible stratas are given approximately as;

$$\delta_{layered} = \frac{P}{L}\left(\sum_{k=2}^{m-1} \frac{I_k - I_{k+1}}{E_{sk}}\right) \qquad \ldots(2.19)$$

where I_k: displacement influence factor I_ρ on the pile axis at level of the top of layer j; E_{sk}: Young modulus of layer k; m: number of layers of different soils.

For application of equation (2.19), it is convenient to have values of influence factor I_ρ on the axis plotted against depth, and such a plot is shown in

Figure 2.11 (Poulos and Davis (1980)) for three values of L/d and for υ_s=0.5. The effect of L/d becomes insignificant for H/L>1.75.

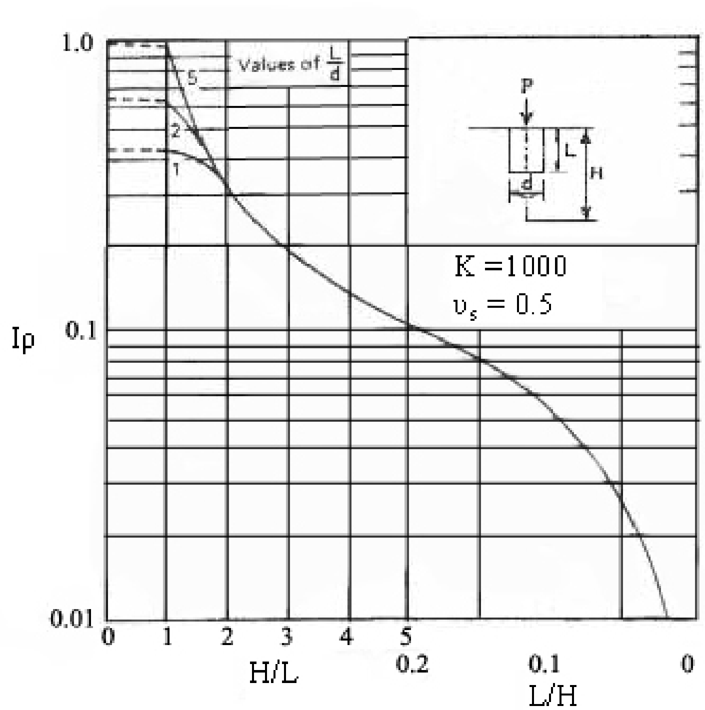

Figure 2.11: Influence factors for settlement beneath center of a pier. (Poulos and Davis, 1980)

2.3. Equivalent Raft Method

This approach is described in many foundation engineering texts, but there are some differences in the suggested procedure for reducing the group to an equivalent raft.

a) The depth at which the equivalent raft is located depends on the nature of the soil profile and ranges from 2l/3 for friction pile groups to l for end-bearing pile groups, where l is the pile length. It is assumed that pressure is distributed at 2V:1H slope (Figure 2.12). If the end-bearing piles rest on a rock or a very hard layer that is thick enough, the settlement analysis is not necessary Ordemir (1984).

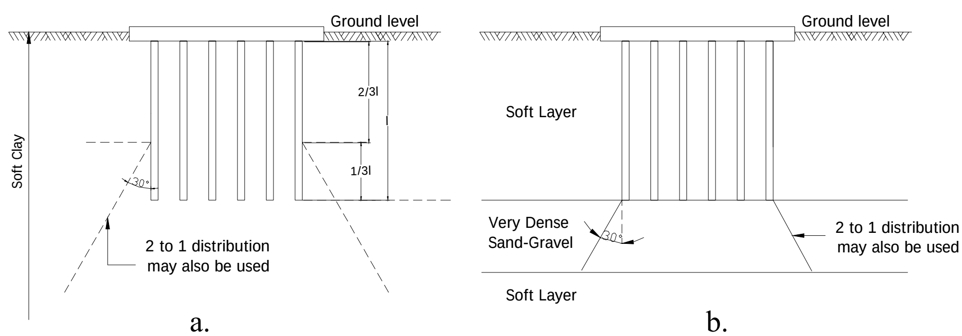

Figure 2.12: Settlement of a group of piles. **a.** Settlement analysis of a group of friction piles in clay **b.** Stresses on top of a compressible layer for calculating settlement of a group of end-bearing piles.

b) The procedure suggested by Tomlinson (1986) is illustrated in Figure 2.13. (Poulos (1993). Load transfer in skin friction from the pile shaft to the surrounding soil is allowed for by assuming that the load is spread from the shafts of friction piles at an angle of 1 in 4 from the vertical. Three cases of load transfer are shown in Figure 2.13.a to c.

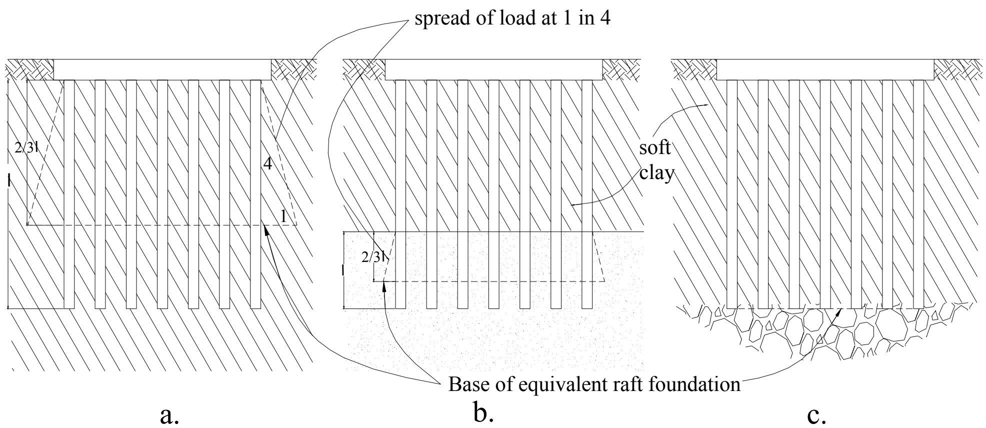

Figure 2.13: Load transfer to soil from pile group. **a.** Group of piles supported predominantly by skin friction. **b.** Group of piles driven through soft clay to combined skin friction and end bearing in stratum of dense granular soil. **c.** Group of piles supported in end bearing on hard rock stratum

In order to obtain more accurate settlement prediction, Brzezinski (in Blanchet, Tavenas, and Garneau (1980)) suggested that the theoretical footing is assumed to be located at the tip of the piles if the pile spacing is large or if a significant number of the piles are battered.

The settlement of piles in cohesive soils primarily consists of the sum of the following two components:

1. Short-term settlement occuring as the load is applied.
2. Long-term consolidation settlement occuring gradually as the excess pore pressures generated by loads are dissipated.

Long-term settlement will be computed by using the drained Young's modulus of the soil. For highly overconsolidated clays, long-term consolidation settlement does not occur. Calculation of short-term (undrained) settlements in clays would require the use of the undrained Young's modulus together with the strain factors for the undrained values of Poisson's ratio.

The average immediate settlement of a foundation at depth D below the surface is;

$$\delta_i = \frac{\mu_i \, \mu_0 \, q_n \, B}{E_u} \qquad \text{...(2.20)}$$

In the above equation Poisson's ratio is assumed to be equal to 0.5. The factors μ_i and μ_0, which are related to the depth of equivalent raft, the thickness of compressible soil layer and the length/width ratio of the equivalent raft foundation, are shown in Figure 2.14.

The influence values in Figure 2.14 are based on the assumption that the deformation modulus is constant with depth. However, in most natural soil and rock formations the modulus increases with dept such that calculations for the conditions based on a constant modulus give exaggerated estimates of settlement.

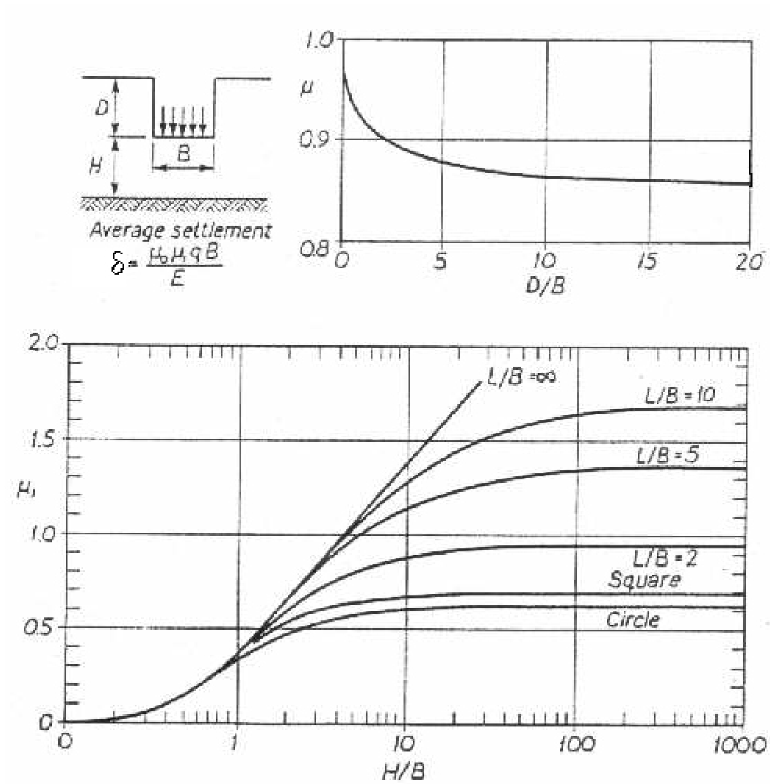

Figure 2.14: Influence factors for calculating immediate settlements of flexible foundations of width B at depth D below ground surface (after Christian and Carrier, 1978)

Butler (1974) developed a method for settlement calculations for the conditions of a deformation modulus increasing linearly with depth within a layer of finite thickness. The value of modulus at a depth z below foundation level is given by the equation;

$$E_d = E_f(1 + k\,z / B) \qquad \text{…(2.21)}$$

and

$$\delta_i = \frac{q_n\,B\,I'_p}{E_f} \qquad \text{…(2.22)}$$

where E_f is the modulus of deformation at foundation level (the base of the equivalent raft) and δ_i is the settlement at the corner of the loaded area. Having obtained k, the appropriate factor for I'_p is obtained from Butler's curves shown in Figure 2.15. These are different ratios for L/B at the level of the equivalent raft, and a reapplicable for a compressible layer thickness not more than 9*B. The curves are based on the assumption of a Poisson's ratio of 0.5 for undrained conditions, this is for immediate application of the load.

The consolidation settlement δ_c, is calculated from the results from oedometer tests made on clay samples in the laboratory. Having obtained a represantative value of m_v for each soil layer stressed by the pile group, the odeometer settlement δ_{oed} for this layer at the centre of the loaded area is calculated from the equation

$$\delta_{oed} = \mu_d \, m_v \, \sigma_z \, H \qquad \ldots(2.23)$$

where μ_d is a depth factor, σ_z is the average effective vertical stress imposed on the soil layer due to the net foundation pressure q_n at the base of the equivalent raft foundation and H is the thickness of the soil layer. The depth factor is obtained from Fox's correction curves shown in Figure 2.16. To obtain the average vertical stress σ_z at the centre of each soil layer the coefficients in Figure 2.17 (Tomlinson (1986)) should be used. The oedometer settlement must now be corrected to obtain the field value of the consolidation settlement, where

$$\delta_c = \delta_{oed} \, \mu_g \qquad \ldots(2.24)$$

Published values of μ_g have been based on comparisons of the settlement of actual structures with computations made from laboratory oedometer tests. Values established by Skempton and Bjerrum (1957) are shown in Table2.4.

Table 2.4: Value of geological factor μ_g (Skempton and Bjerrum, 1957)

Type of Clay	μ_g value
Very sensitive clays (soft alluvial, estuarine and marine clays)	1,0-1,2
Normally-consolidated clays	0,7-1,0
Over-consolidated clays (London clay, Weald, Kimmeridge, Oxford and Lias clays)	0,5-0,7
Heavily over-consolidated clays (unweathered glacial till, Keuper Marl)	0,2-0,5

In layered soils with different values of the deformation modulus E_u in each layer or soils which show progresively increasing modulus with increases in depth, the strata below the base of the equivalent raft are divided into a number of representative horizontal layers and average value of E_u is assigned to each layer. The dimensions L and B in Figure 2.14 are determined on the assumption that the load is spread to the surface of each layer at an angle of 30 from the edges of the equivalent raft (Figure 2.18) (Tomlinson (1986). The total settlement of piled foundation is then sum of the average settlements calculated for each soil layer from equation 2.20.

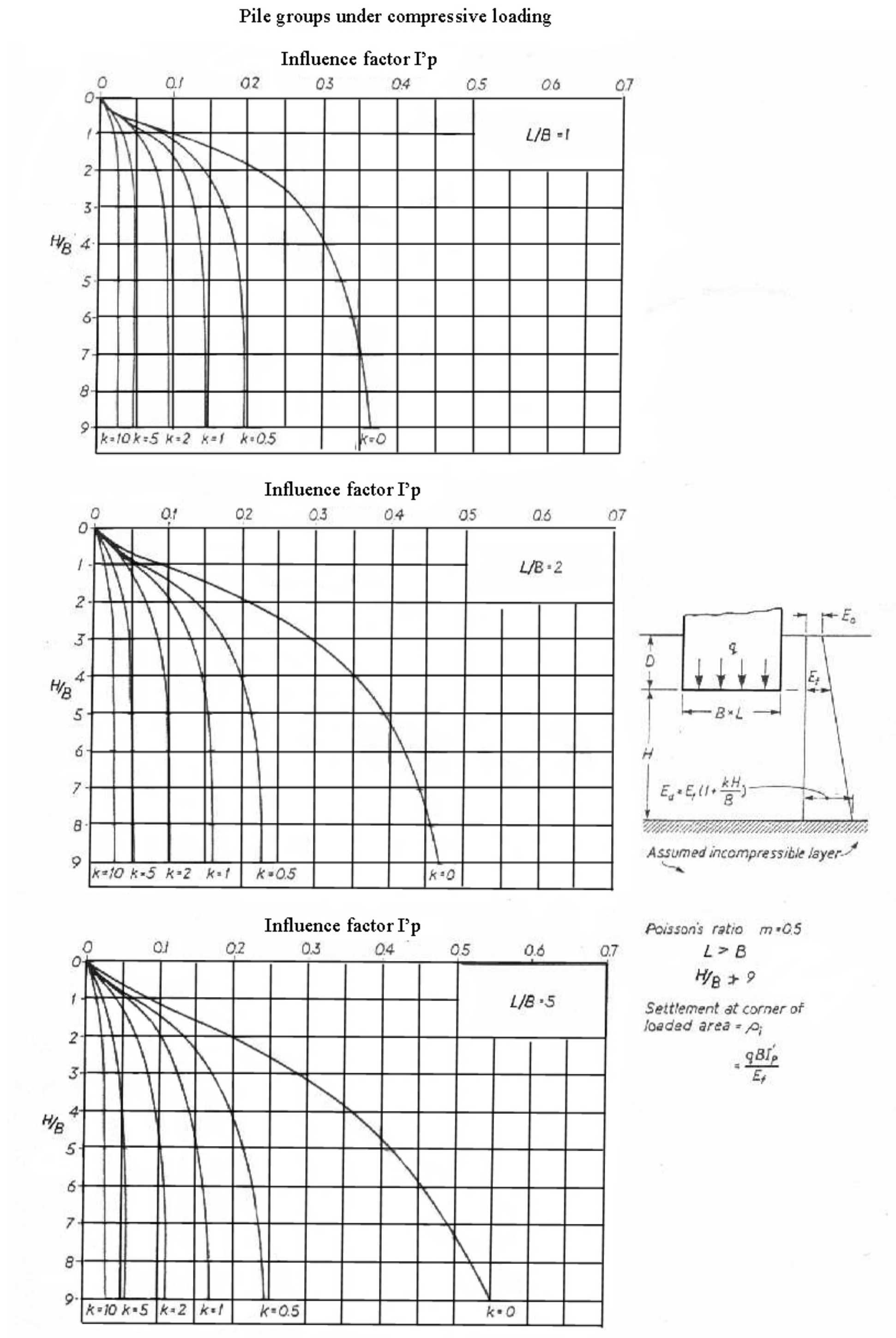

Figure 2.15: Values of the influence factor I'_p for deformation modulus increasing linearly with depth and modular ratio of 0.5 (after Butler. 1974)

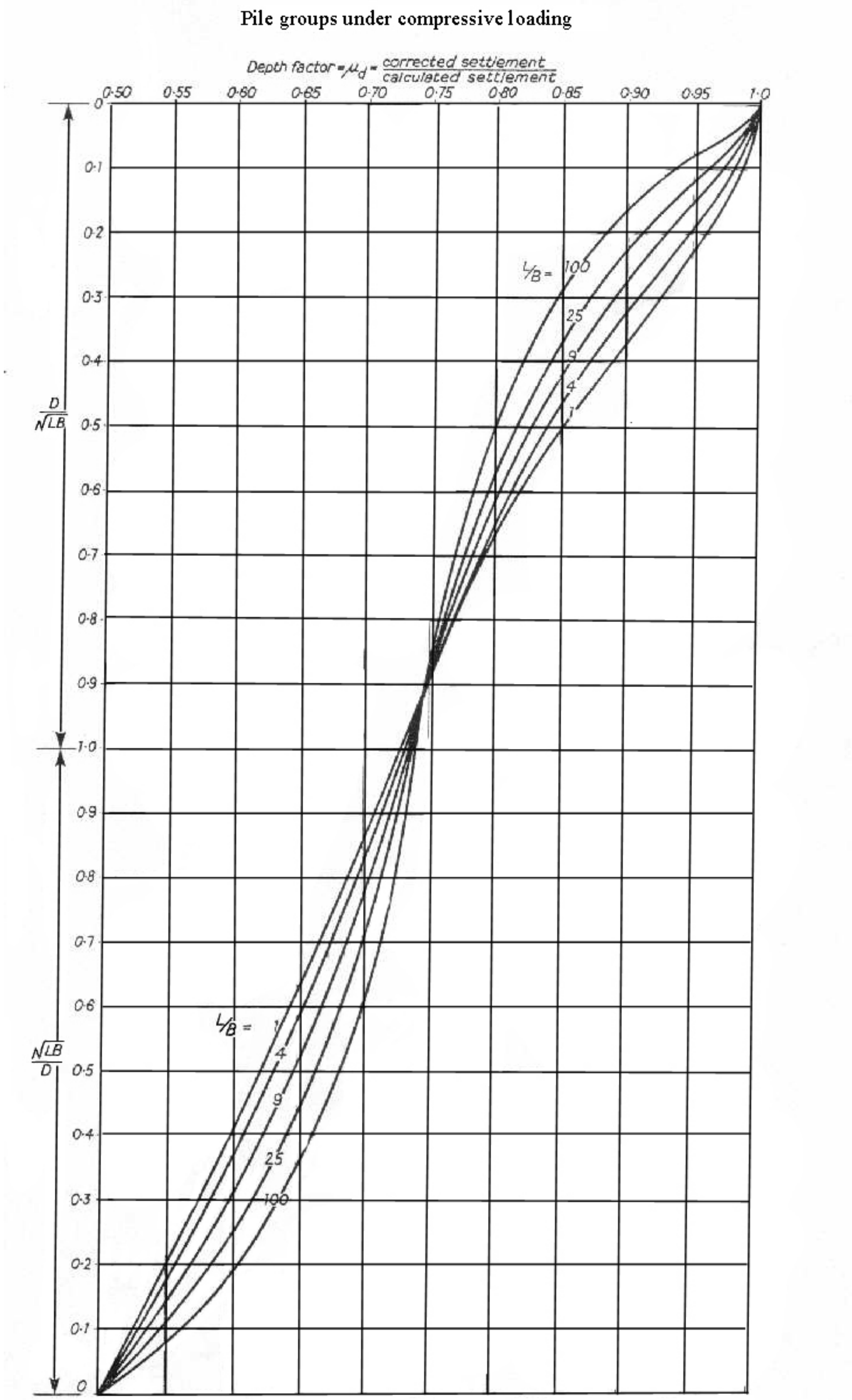

Figure 2.16: Depth factor μ_d for calculating oedometer settlements (after Fox, 1948)

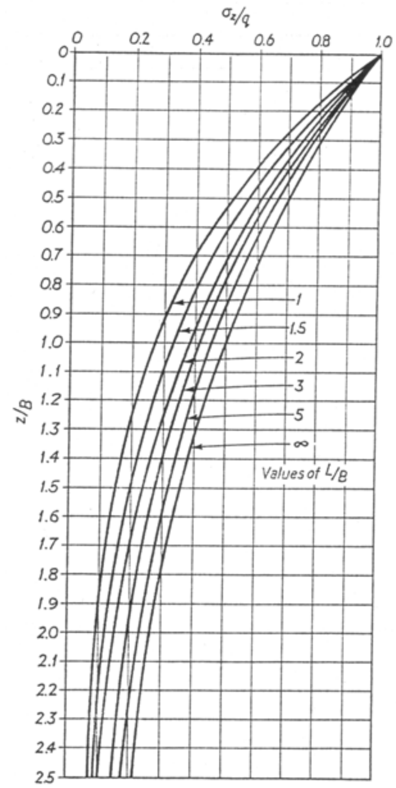

Figure 2.17: Calculating of mean vertical stress (σ_z) at depth z beneath rectangular area a*b on surface loaded at uniform pressure q (Tomlinson, 1986)

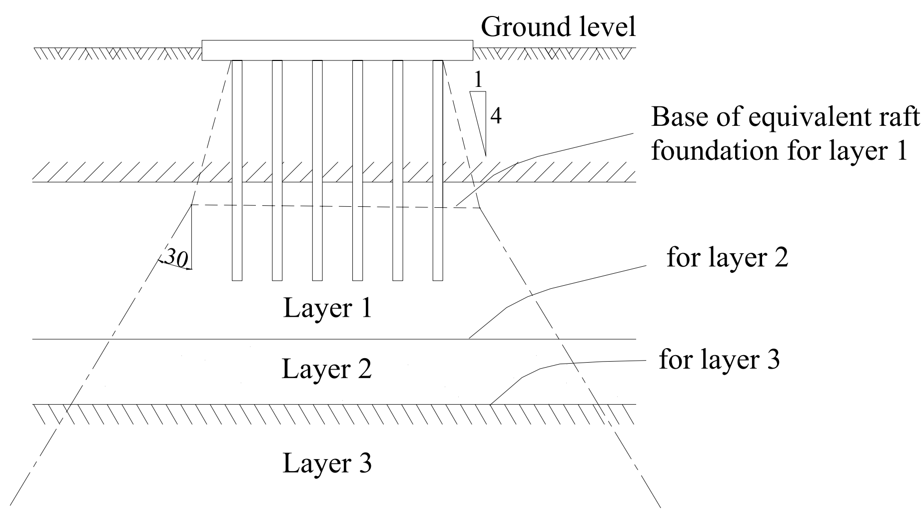

Figure 2.18: Load distribution beneath pile group in layered soil formation

For friction piles driven into sand; the load settlement relationship of a single pile driven into coarse granular soils can be determined by pile load tests. If the settlement of the test pile is within permissible limits, the settlement of the pile group will also be within permissible limits, because the granular soil between the piles will be compacted by pile driving and the soil will be more dense and less compressible. Therefore, no settlement analysis for driven piles in sand is required. For the pile group terminating in rock, anticipated settlement is 0,01-0,05% of the group width.

CHAPTER 3

AN EXEMPLARY CASE HISTORY

3.1. Messeturm Tower (n=64)

The building has a basement with two underground floors, 58,8 m square in plan, and a 60-storey core shaft (41 m* 41 m in plan) up to height of 210 m. The estimated total load of the building is 1880 MN. At the site of the Messeturm building there are gravels and sands with a thickness of 8 m, followed by Frankfurt Clay to a depth of more than 100 m below the ground surface.

In order to reduce settlements and tilt, the foundation system comprised a base slab or raft supported and stabilised against tilt by 64 large diameter bored piles. The raft is founded at a depth of 14 m below the ground surface on the Frankfurt Clay, and is 9 m below the grounwater table. The thickness of the raft decrease from 6.0 m at the centre to 3.0 m at the edges. The bored piles have a diameter of 1.3 m and are arranged in three concentric circles below the raft. The distance between the piles varies from 3,5 to 6 pile diameters. The pile length varies from 26.9 m for the 28 piles in the outer circle to 30.9 m for the 20 piles in the middle circle, and to 34.9 m for the 16 piles in the inner circle. Calculated range of settlement is 150-200 mm using different methods. (Katzenbach, R. et al., 2000, Poulos, H.G., 2000, Poulos, H.G., 2001)

a) Settlement Ratio Method

n= 64 d= 1,3 m r_0= 0,65 m s= 4,75 m

P=1.880 MN L= 30,9 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,1 υ_s = 0,3 Frankfurt Clay

$\lambda=E_p/G_l$=30000/56,9 ≈ 572,24

L/d=23,769→0,54 (Fig. 2.1)

$\rho=G_{l/2}/G_l$=0,728→0,99 (Fig. 2.1)

logλ=2,722→0,93 (Fig. 2.1)

s/d=4,75→0,88 (Fig. 2.1)

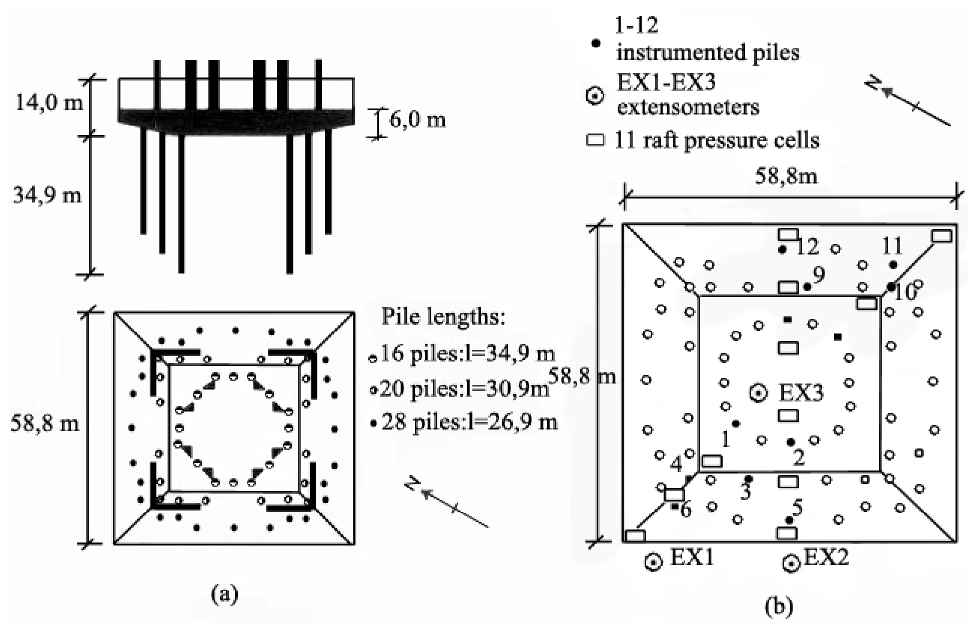

Figure 3.1: Piled raft foundation for Messeturm building, **(a)** plan and cross-section **(b)** location of instrumentation (Katzenbach, 2000, Poulos, 2000, Poulos, 2001)

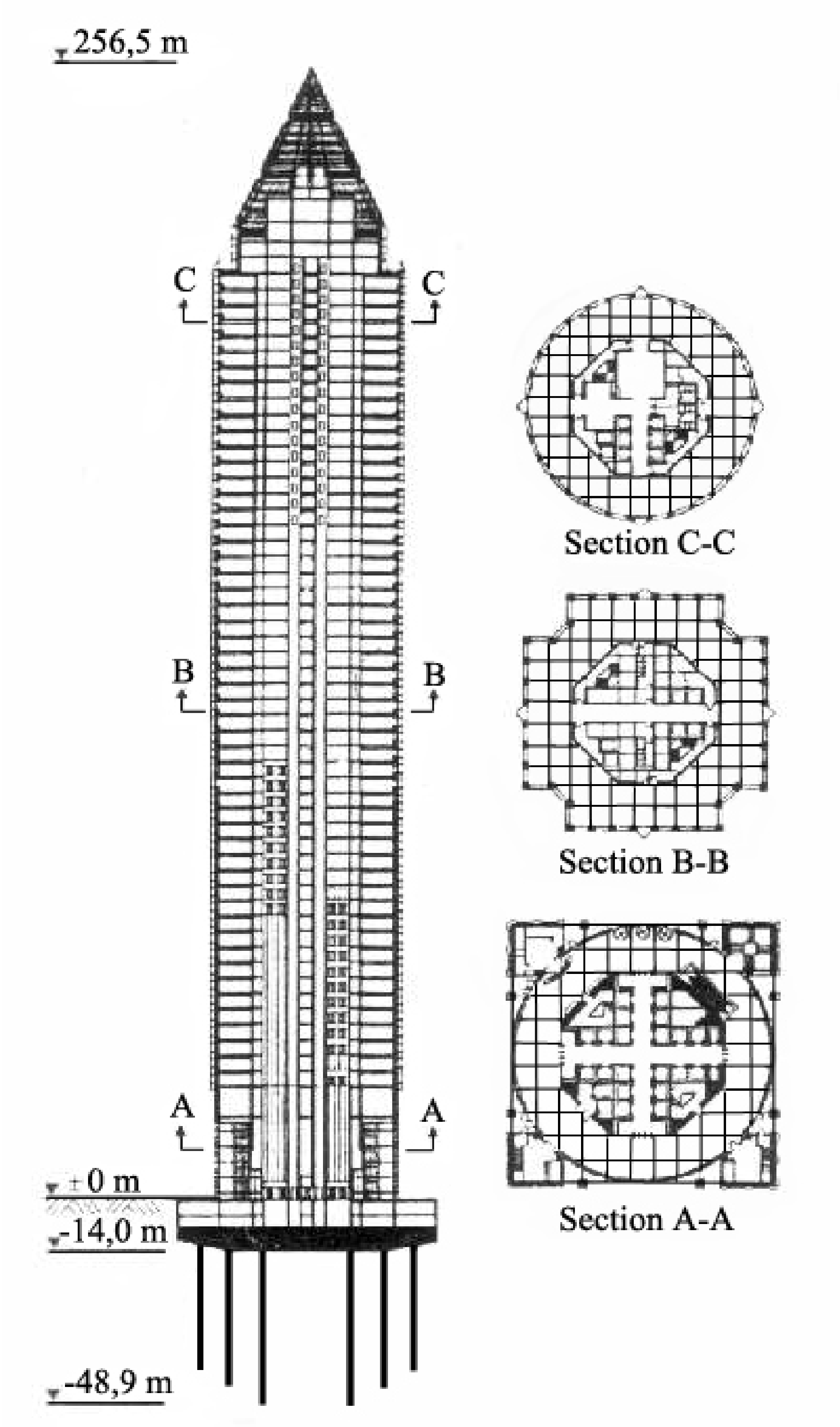

Figure 3.2: Messeturm building, cross-sections (Katzenbach, 2000, Poulos, 2000, Poulos, 2001)

υ_s=0,1→1,05 (Fig. 2.1)

υ_s=0,3→1

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=1880000/64=29375 KN

$\delta_{single}=P_{single}/k$ (mm)

	e	η_w	R_s	ζ	μL	tanμL L/(μL r_0)	$P_t/(w_tG_lr_0)$
υ_s=0,1	0,459	0,147	6,756	4,356	1,403	30,022	33,309
υ_s=0,3	0,437	0,162	6,169	4,104	1,445	29,431	34,983

	P_t/w_t	$K=n\eta_wk$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_sR_s$
υ_s=0,1	1231,954	11668,88	161,11	23,84	161,11
υ_s=0,3	1293,872	13422,63	140,06	22,70	140,06

$\delta_{measured}$=130 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =58,8 m

$A_P=\Pi d^2n/4$=84,9487 m^2

E_p=30000 MPa

E_s'=125,18 MPa E_u=170,7 MPa

d_e=1,27 $A_G^{0,5}$=74,676 (for friction piles)

ρ=0,728   L=30,9 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\lambda = E_e/G_l$=859,199/56,9 ≈ 15,10

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	859,199	15,100	0,304	0,545	0,377	0,298	60,08
			1,849	0,221	0,407	0,733	147,43
υ_s=0,3	881,4	15,49	0,053	1,285	0,276	0,104	17,80
			1,800	0,221	0,407	0,732	124,55

Method 2

L/d_e=30,9/74,676 =0,413 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	100,55	85,08

K ≈ 200 (pile stiffness factor)   s/d ≈ 4,75   L/d ≈ 23,769   B=58,8 m

d_e/B ≈ 0,77 assumed, then d_e ≈ 45,276 m   (Fig.2.9)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	859,199	15,100	0,805	0,553	0,620	0,427	141,70
			1,979	0,353	0,655	0,660	219,20
υ_s=0,3	881,4	15,49	0,554	0,659	0,598	0,380	106,69
			1,908	0,355	0,655	0,677	190,12

Method 2

L/d_e=30,9/45,276=0,68 → I_δ=0,47 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	155,90	131,91

$\delta_{measured}$=130 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
62	62	40	1	0,645	0,558
82	82	40	1	0,487	0,909

P=1880000 KN υ_s= 0,1 υ_u= 0,5

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_1, μ_0 → Fig. 2.14

μ_0	μ_1	E_{uave}	q	δi
0,93	0,23	199,8	489,07	32,46
0,92	0,18	320,4	279,59	11,84

$\delta_{i\ ave}$= 44,31 mm

$m_{\upsilon}=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}=0,558 \rightarrow \mu_d=0,83$ (Fig. 2.16)

Frankfurt Clay → $\mu_g=0,7$ (Table 2.4)

$\delta_c=m_{\upsilon}\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
146,52	0,0066	322,78	50,06
234,96	0,0041	134,49	13,00

δ_c= 63,06 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 107,38 mm

$\delta_{measured}$=130 mm

Two different values were used for Poisson's ratio in the calculations. These were advised as upper and lower values for the soil. Results which were obtained by using lower Poisson's ratio were used in the graphs.

If L/re was greater than 1 results which were obtained from d_{e1} formulations (I (Fig. 2.10) and I (equation 2.16-A=0) were used in the graphs for equivalent pier method. If L/r_e was less than 1 then results which were obtained from formulations d_{e1} (I (Fig. 2.10) and I (equation 2.16-A=5) and d_{e2} (I (equation 2.16-A=0) were used in the graphs. For end bearing piles results for d_{e2} (I (equation 2.16-A=5) were used.

Calculations were made for different pressure distributions and compressible layer thickness (H) values for equivalent raft method. For graphs ¼ pressure distribution and H=2B (B:width of equivalent raft) results were used.

Table 3.1: Measured and computed settlements for Messeturm Building (mm)

<table>
<tr><th rowspan="4"></th><th colspan="10">Settlement (mm)</th></tr>
<tr><th rowspan="3">Set. Ratio</th><th colspan="4">Equivalent Pier</th><th colspan="4">Equivalent Raft</th><th rowspan="3">Mea.</th></tr>
<tr><th colspan="2">d_{e1}</th><th colspan="2">d_{e2}</th><th>H=80 m</th><th>H=71 m (at the tip)</th><th>H=80 m (1/6)</th><th>H=80 m (1/8)</th></tr>
<tr><th>Met1</th><th>Met2</th><th>Met1</th><th>Met2</th><th>Ave.</th><th>Ave.</th><th>Ave.</th><th>Ave.</th></tr>
<tr><td rowspan="2">υs=0,1</td><td rowspan="2">161,11</td><td>60,08</td><td rowspan="2">100,55</td><td>141,7</td><td rowspan="2">155,9</td><td rowspan="2">107,38</td><td rowspan="2">113,75</td><td rowspan="2">115,21</td><td rowspan="2">120,88</td><td rowspan="4">130</td></tr>
<tr><td>147,43</td><td>219,2</td></tr>
<tr><td rowspan="2">υs=0,3</td><td rowspan="2">140,06</td><td>17,8</td><td rowspan="2">85,08</td><td>106,69</td><td rowspan="2">131,91</td><td rowspan="2">84,85</td><td rowspan="2">90,75</td><td rowspan="2">91,24</td><td rowspan="2">95,91</td></tr>
<tr><td>124,55</td><td>190,12</td></tr>
</table>

Messe Turm

Rs	161,11	Mea.	130
Pier	147,43		
	100,55		
	141,70		
Raft	107,38		

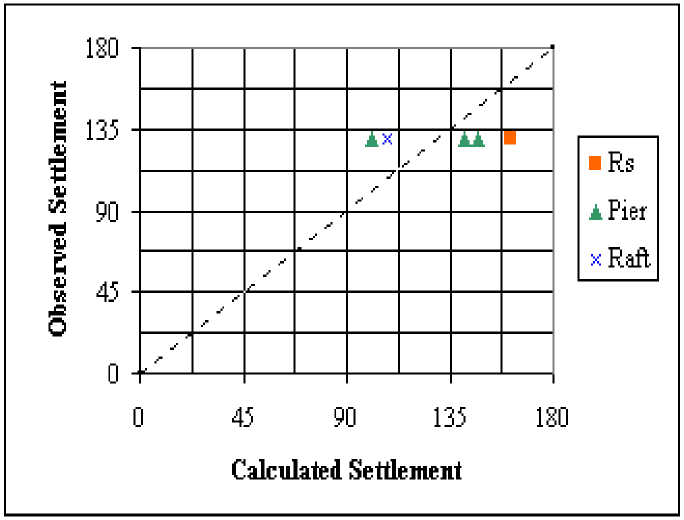

Figure 3.3: Measured and computed settlements for Messeturm Building (mm)

CHAPTER 4

SUMMARY AND CONCLUSION

The conclusions reached are enumerated below, but an explanatory introduction may be needed. The calculation methods have been summarized in some detail in Chapter 2. It is possible to calculate different settlement values by the equivalent pier method depending on the selection of displacement influence factor I_δ and equivalent diameter d_e. I_δ is either selected based on equation. 2.16 (Method 1), or using Fig.2.10 (Method 2). Also two different equivalent pier diameters are obtained by using equations 2.13, 2.14 (d_{e1}) and Figure 2.9 (d_{e2}). ς, measure of radius of influence of pile, which is used in equation 2.16 to obtain I_δ, can be calculated by equation 2.5 (inferred A=0) and equation 2.18 (A=5). As it is recalled coefficient A is an empirical coefficient. As a result there are three displacement influence factors I_δ can be obtained for each equivalent pier diameter by using Figure 2.10 and equation 2.16 with equations 2.5 and 2.18 (A=0, A=5). Also L/r_e and type of the pile (friction vs. end –bearing) are the additional factors to be considered in the interpretation. Fig. 4.1 summarizes the types of solutions.

Total settlements by equivalent raft method are obtained by adding initial settlements and consolidation settlements. Average consolidation settlements are estimated by the conventional procedure and the initial average settlements are estimated by Christian and Carrier (1978) for constant E_u.

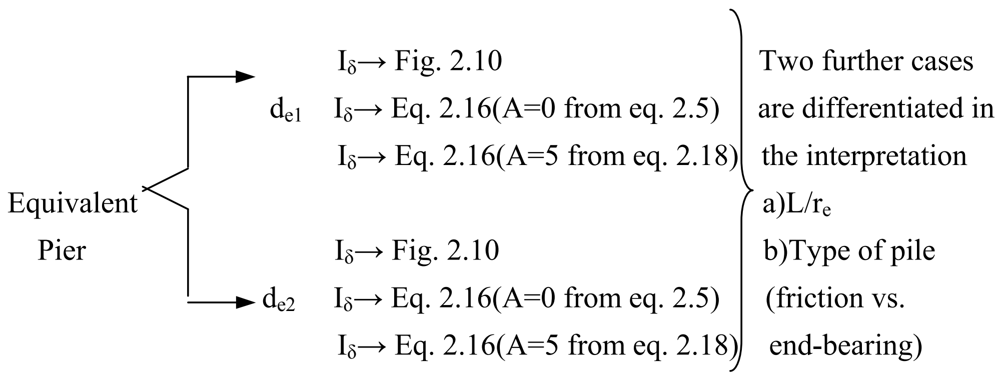

Figure 4.1: Equivalent Pier Method - Summary variations in the calculation procedures.

A flow chart is provided in Fig. 4.2 to make an easier selection of the proper method for a case

1. In general settlement ratio method gives overestimated settlement values (Fig. 4.3 and Fig. 4.4).

2. It is not possible to get reasonable results by using settlement ratio method if number of piles is less than 16, (Table 4.3). For small pile groups, equivalent raft method gives better predictions (9-pile group, Test of Kaino). It may be possible to get an idea for small groups by using equivalent pier method.

3. It is proposed that a relationship can be described between settlements calculated by the settlement ratio method and p values (Figures 4.6 – 4.7) where

$$p = \frac{P_{Total}\,(kN)}{1000\,(kN).e} \qquad ...(4.1)$$

e: Efficiency exponent

P_{Total}: Total load (kN)

p: Dimensionless parameter

It is observed that settlement ratio method gives better correlations when p is greater than 50 and less than 1000 (Fig. 4.6).

4. It is observed that the settlement ratio method is not suitable when the shape of the piled rafts is not regular and when the raft area is larger than plan area of pile group (Pollux, Kastor, etc.) (Fig. 4.5). For such pile groups, equivalent raft method gives better results.

5. Equivalent pier diameter from Fig. 2.9 (d_{e2}) is always lower than d_{e1} from equation 2.13, 2.14. Therefore higher settlement values are calculated by using d_{e2}.

6. If L/r_e is less than 1, I_δ, from Fig 2.10 for d_{e1} gives the best results (Fig. 4.8). Another alternative is to obtain I_δ from equation 2.16 (A=0 from 2.18) for d_{e2} (Fig. 4.9) and this correlation is not as good as the former.

7. For friction piles, when L/r_e is greater than 1, d_{e1} formulation should be used. Reasonable results can be calculated by using I_δ, from equation 2.16 with A=0 and, from Fig. 2.10 (Figures 4.10 –4.11).

8. As it is seen in Figures 4.12 and 4.13 that for end-bearing piles, using d_{e2} formulation and I_δ (equ. 2.16) with equation 2.18 (A=5) is the only way to get reasonabele results. The rest of the equivalent pier procedures give underestimated settlement values.

9. When a rock layer exists at the pile tip (Commerz Bank, Main Tower, Japan Centre) very high settlement values are calculated by the settlement ratio method (Table 4.3). On the other hand for the same situation, equivalent raft method tends to give underestimated results (Table 4.9).

10. It is observed that when L/r_e is greater than 5, equivalent pier method does not give reasonable results (Test of Kaino, Field test on five pile group, Frame type building 2-3-7).

11. In general, the best correlations between calculated and observed settlements are obtained from the equivalent raft method (Fig. 4.14). Correlations for friction piles are better than those of end-bearing piles (Fig. 4.15-4.16).

12. It can be seen from Fig 4.17 that s/d is one of the important parameters for equivalent raft method. Calculated settlement values increase as s/d decreases for friction piles (Fig. 4.17).

13. If s/d greater is than 4 and either L_{pile} is greater than 25 m or $B_{raft/L}$ pile is less than 1.2 then equivalent raft can be best assumed using 8V:1H pressure distribution (Fig. 4.18).

14. It is considered that practically consolidation settlement does not exist under lightly loaded small pile groups in sandy soils. Time dependent settlements are observed under heavily loaded large groups in sandy soils. Therefore settlement calculations for large groups may be performed like in clayey soils by equivalent raft method (Test of Kaino, Five Storey Building, 19-Storey, Hotel Japan, Treptowers).

Results obtained from all the methods are presented together as best lines for friction and end-bearing piles in Figs. 4.19 and 4.20 respectively.

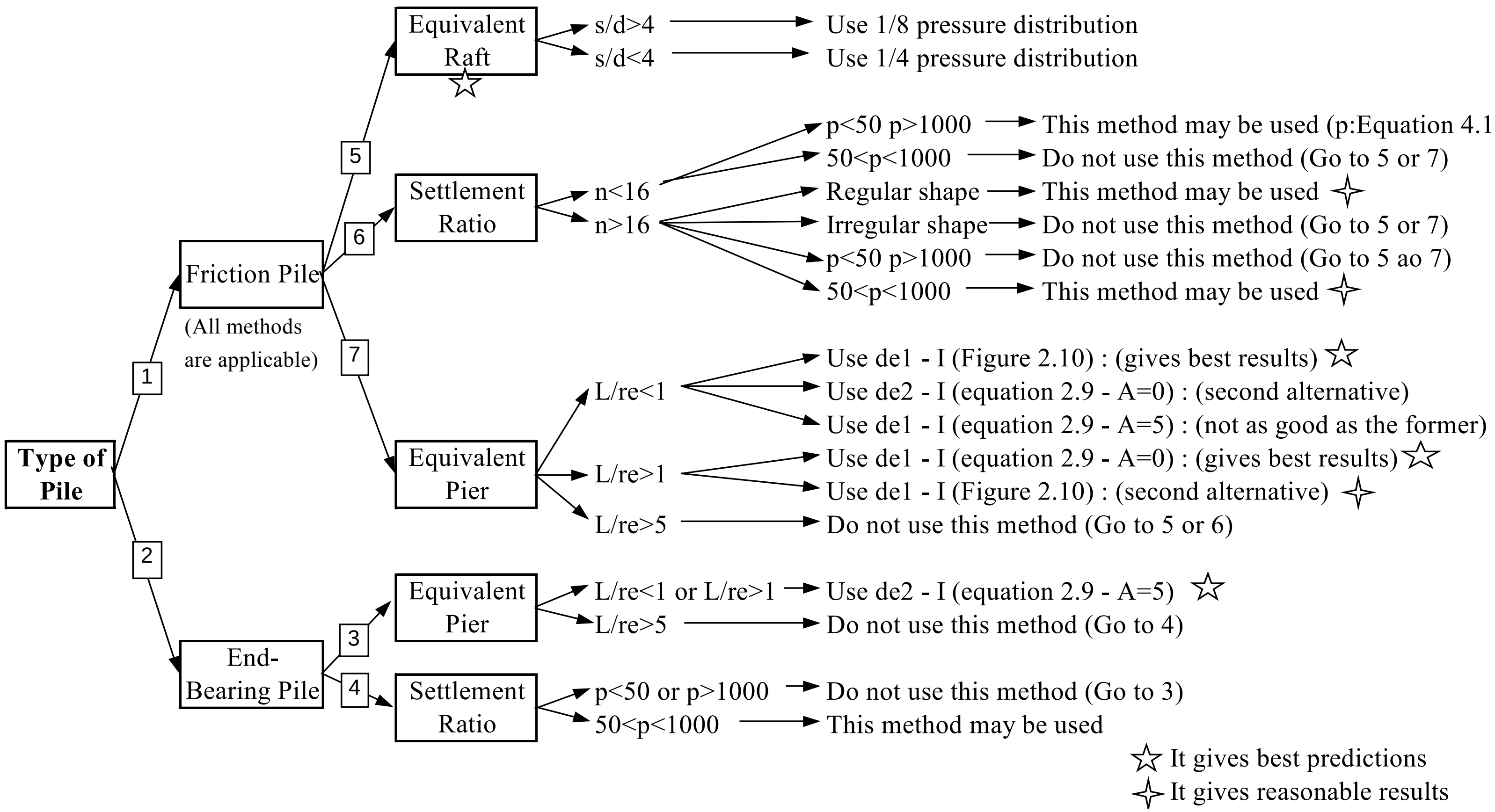

Figure 4.2: Selection of the proper method presented by a flow chart

Table 4.1: Calculated and observed settlement values for settlement ratio method (mm)

	Settlement Ratio	Cal.	Mea.	(Cal.-Mea.) / Mea.*100			Cal.	Mea.	(Cal.-Mea.) / Mea.*100
1	Field Test	7,59	38,1	80,08	17	Molasses Tank	25,34	29,5	14,10
2	Test of Kaino	9,48	3,8	149,47	18	Messeturm	161,11	130	23,93
3	Frame-type 2	29,61	13	127,77	19	New Court II	30,56	31,5	2,98
4	Frame-type 3	21,58	5	331,60	20	New Court I	36,67	28,1	30,50
5	9-Pile group	2,3	0,9	155,56	21	New Court III	29,09	25,1	15,90
6	Frame-type 7	13,26	4	231,50	22	Congress Office	71,46	45	58,80
7	Five-storey	13,7	12,65	8,30	23	Congress Hotel	105,45	50	110,90
8	Eurotheum	35,05	32	9,53	24	Commerz Bank	36,41	17	114,18
9	Japan Centre	74,89	50	49,78	25	Main Tower	51,1	20	155,50
10	Forum Kastor	156,40	75	108,53	26	Cambridge Road	31,42	27,5	14,25
11	Forum Pollux	139,59	80	74,49	27	19-Storey	77,12	64	20,50
12	American Express	291,4	55	429,87	28	Hotel Japan	17,14	17,5	2,06
13	Westend I Tower	165,7	110	50,66	29	İzmir Hilton	83,3	69,6	19,68
14	Messe-Torhaus	47,48	45	5,51	30	Frame-type 6	79,69	19	319,42
15	Gratham Road	32,84	30	9,47	31	Stonebridge	29,34	25	17,36
16	Treptowers	98,4	63	56,19	32	Dashwood	35,29	33	6,94
					33	Ghent Grain	119,14	185	35,60

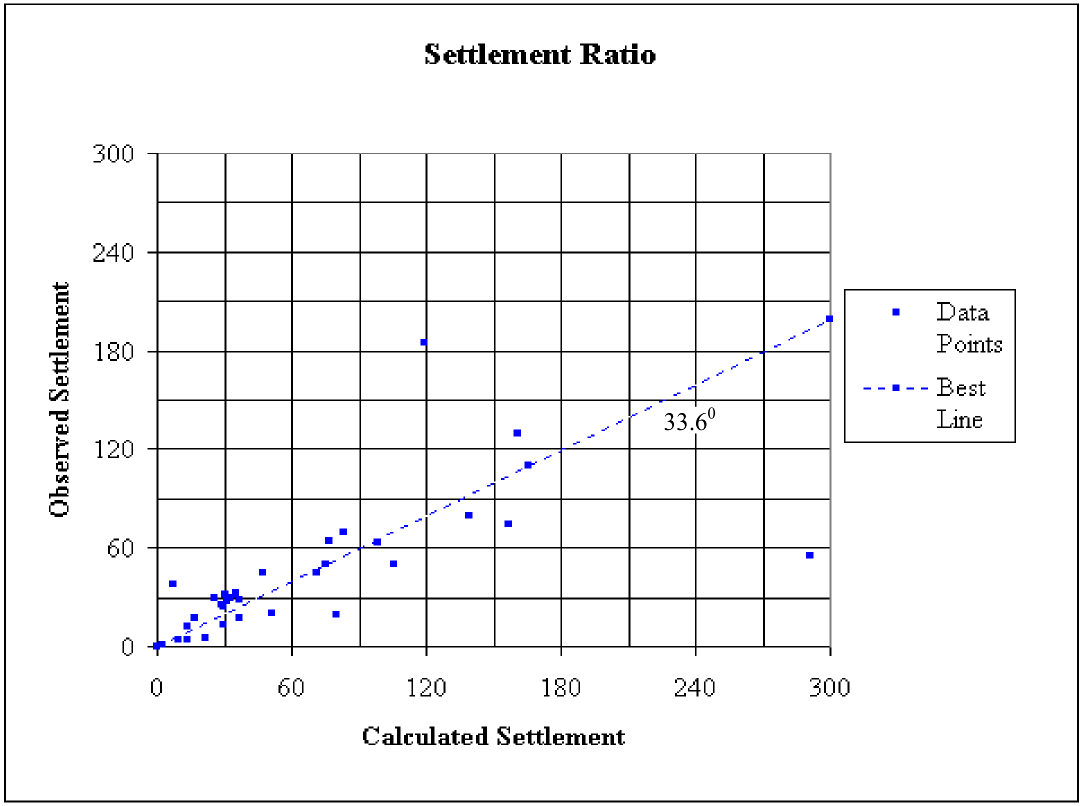

Figure 4.3: Calculated and observed settlement values for all cases (mm)

Table 4.2: Calculated and observed settlement values for friction and end-bearing piles settlement ratio method (mm)

	Friction Piles	Cal.	Mea.			Cal.	Mea.
1	Field Test	7,59	38,1	14	Molasses Tank	25,34	29,5
2	Test of Kaino	9,48	3,8	15	Messeturm	161,11	130
3	Frame-type 2	29,61	13	16	Congress Office	71,46	45
4	Frame-type 3	21,58	5	17	Congress Hotel	105,45	50
5	9-Pile group	2,3	0,9	18	Cambridge Road	31,42	27,5
6	Frame-type 7	13,26	4	19	19-Storey	77,12	64
7	Five-storey	13,7	12,65	20	Hotel Japan	17,14	17,5
8	Forum Kastor	156,40	75	21	İzmir Hilton	83,3	69,6
9	Forum Pollux	139,59	80	22	Frame-type 6	79,69	19
10	American Express	291,43	55	23	Stonebridge	29,34	25
11	Westend I Tower	165,73	110	24	Dashwood	35,29	33
12	Messe-Torhaus	47,48	45	25	Ghent Grain	119,14	185
13	Gratham Road	32,84	30				
	End-Bearing Piles						
1	Eurotheum	35,05	32	5	New Court I	36,67	28,1
2	Japan Centre	74,89	50	6	New Court III	29,09	25,1
3	Treptowers	98,4	63	7	Commerz Bank	36,41	17
4	New Court II	30,56	31,5	8	Main Tower	51,1	20

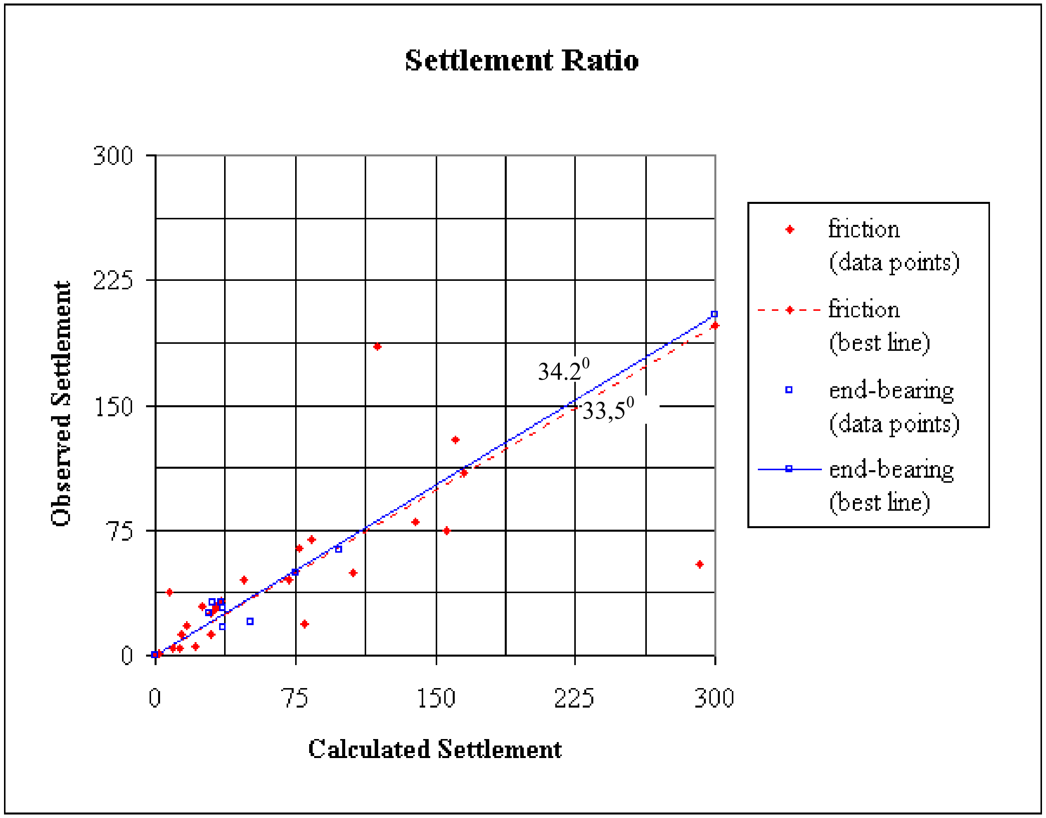

Figure 4.4: Calculated and observed settlement values for friction and end-bearing piles (mm)

Table 4.3: Calculated and observed settlement values for friction piles (n>16, n<16, regular, irregular shapes), end-bearing piles (n>16) -settlement ratio method (mm) (A,C:regular shapes; B:irregular shapes)

A	**Friction piles**	**Cal.**	**Mea.**	(n>16)	**B**	**Friction piles**	**Cal.**	**Mea.**	(n>16)
1	Five-storey B.	13,7	12,65	20	1	Forum Kastor	156,40	75	22
2	Westend I Tower	165,73	110	40	2	Forum Pollux	139,59	80	26
3	Messe-Torhaus	47,48	45	42	3	American Exp.	291,43	55	35
4	Gratham Road	32,84	30	48	4	Congress Office	71,46	45	43
5	Molasses Tank	25,34	29,5	55	5	Congress Hotel	105,45	50	98
6	Messeturm Tower	161,11	130	64	**D**	**End-bearing piles**	(n>16)		n
7	Cambridge Road	31,42	27,5	116	1	Eurotheum	35,05	32	25
8	19-Storey B.	77,12	64	132	2	Treptowers	98,4	63	54
9	Hotel Japan	17,14	17,5	157	3	New Court II	30,56	31,5	77
10	İzmir Hilton	83,3	69,6	189	4	New Court I	36,67	28,1	82
11	Frame-type 6	79,69	19	192	5	New Court III	29,09	25,1	82
12	Stonebridge Park	29,34	25	351	**6**	Japan Centre	**74,89**	**50**	25
13	Dashwood House	35,29	33	462	**7**	Commerz Bank	**36,41**	**17**	111
14	Ghent Grain	119,14	185	697	**8**	Main Tower	**51,1**	**20**	112
C	**Friction piles**			(n<16)					
1	Field Test	7,59	38,1	5	4	Frame-type 3	21,58	5	9
2	Test of Kaino	9,48	3,8	5	5	9-Pile group	2,3	0,9	9
3	Frame-type 2	29,61	13	6	6	Frame-type 7	13,26	4	16

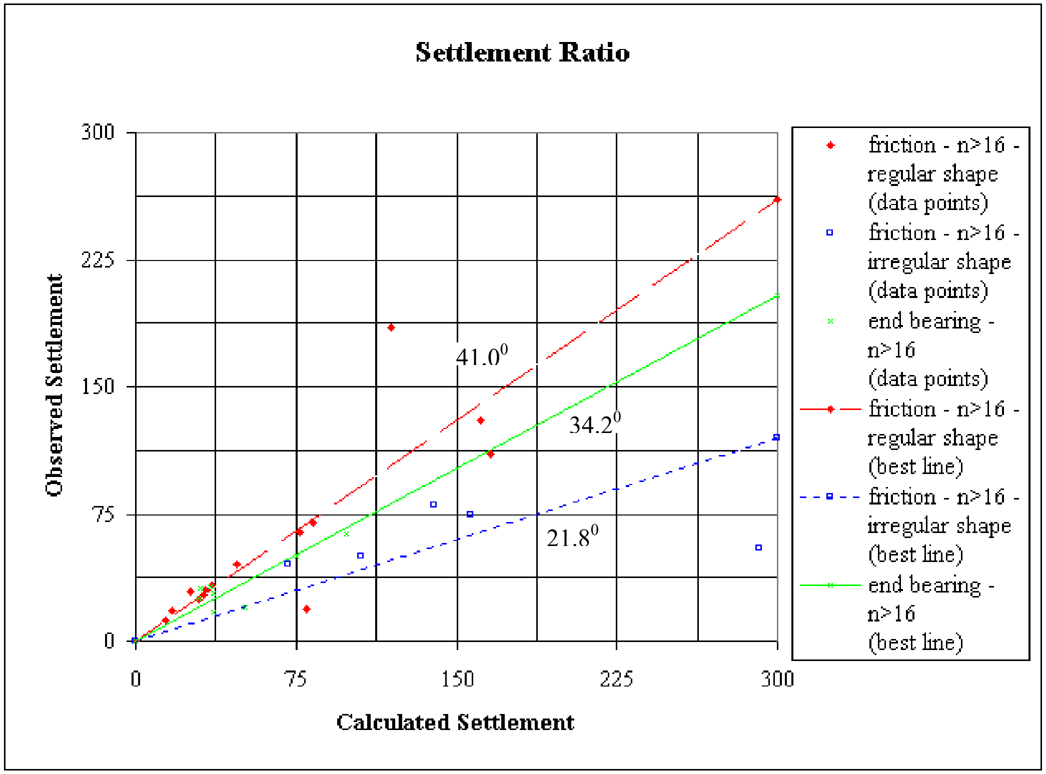

Figure 4.5: Calculated and observed settlement values for friction piles (n>16, regular, irregular shapes), end-bearing piles (n>16) (mm)

Table 4.4: Calculated and observed settlement values (50<p<1000, p<50 p>1000)-settlement ratio met. (mm)

		Cal.	Mea.	50<p<1000
1	Five-storey B.	13,7	12,65	53,23
2	Messe-Torhaus	47,48	45	336,9
3	Gratham Road	32,84	30	185,8
4	Molasses Tank	25,34	29,5	59,6
5	New Law II	30,56	31,5	769,8
6	New Law I	36,67	28,1	951,2
7	New Law III	29,09	25,1	702,1
8	Cambridge	31,42	27,5	239,3
9	19-Storey B.	77,12	64	392,6
10	Hotel Japan	17,14	17,5	371,2
11	Frame-type 6	79,69	19	491,7
12	Stonebridge	29,34	25	297
13	Dashwood	35,29	33	516,4

		Cal.	Mea.	p<50 p>1000
1	Field Test	7,59	38,1	5,04
2	Test of Kaino	9,48	3,8	12,31
3	Frame-type 2	29,61	13	22,43
4	Frame-type 3	21,58	5	12,74
5	9-Pile group	2,3	0,9	4
6	Frame-type 7	13,26	4	28,58
7	Eurotheum	35,05	32	1141
8	Japan Centre	74,89	50	2215
9	Forum Kastor	156,40	75	1991
10	Forum Pollux	139,59	80	2167
11	American Express	291,43	55	2141
12	Westend I Tower	165,73	110	3107
13	Treptowers	98,4	63	1474
14	Messeturm Tower	161,11	4092	130
15	Congress Office	71,46	1162	45
16	Congress Hotel	105,45	2648	50
17	Commerz Bank	36,41	2645	17
18	Main Tower	51,1	4201	20
19	İzmir Hilton	83,3	1549	69,6
20	Ghent Grain	119,14	1868	185

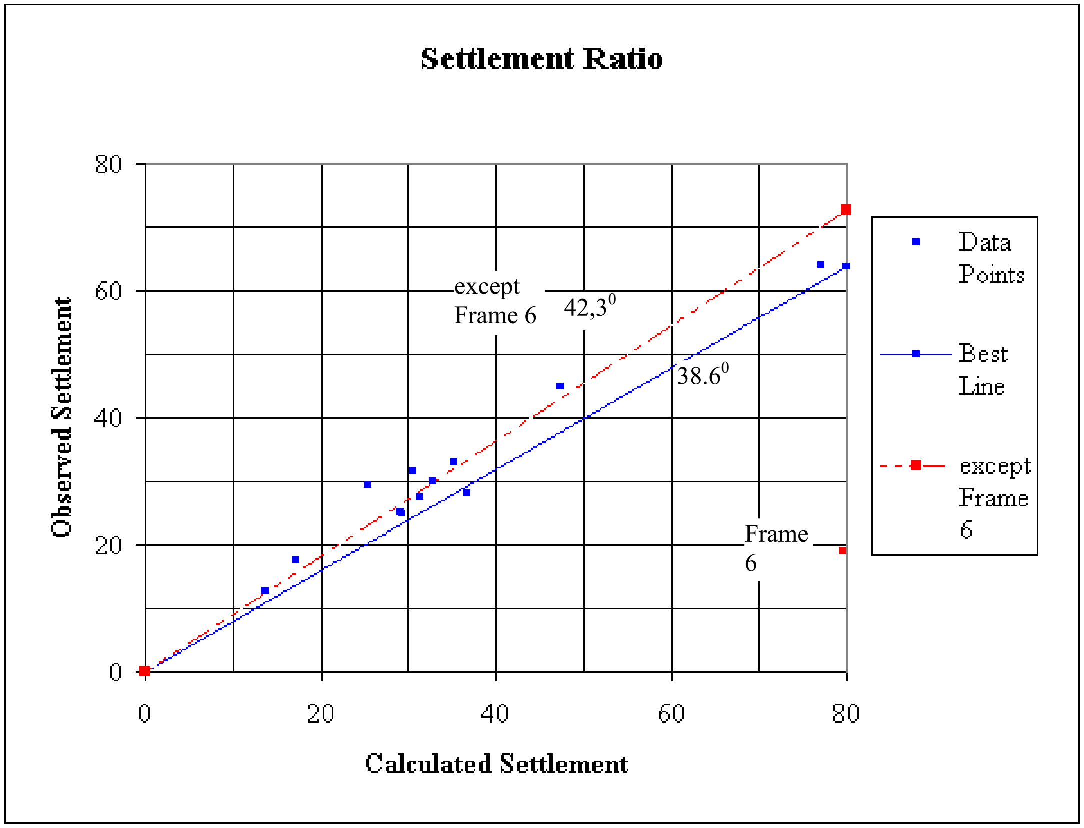

Figure 4.6: Calculated and observed settlement values (50<p<1000) (mm)

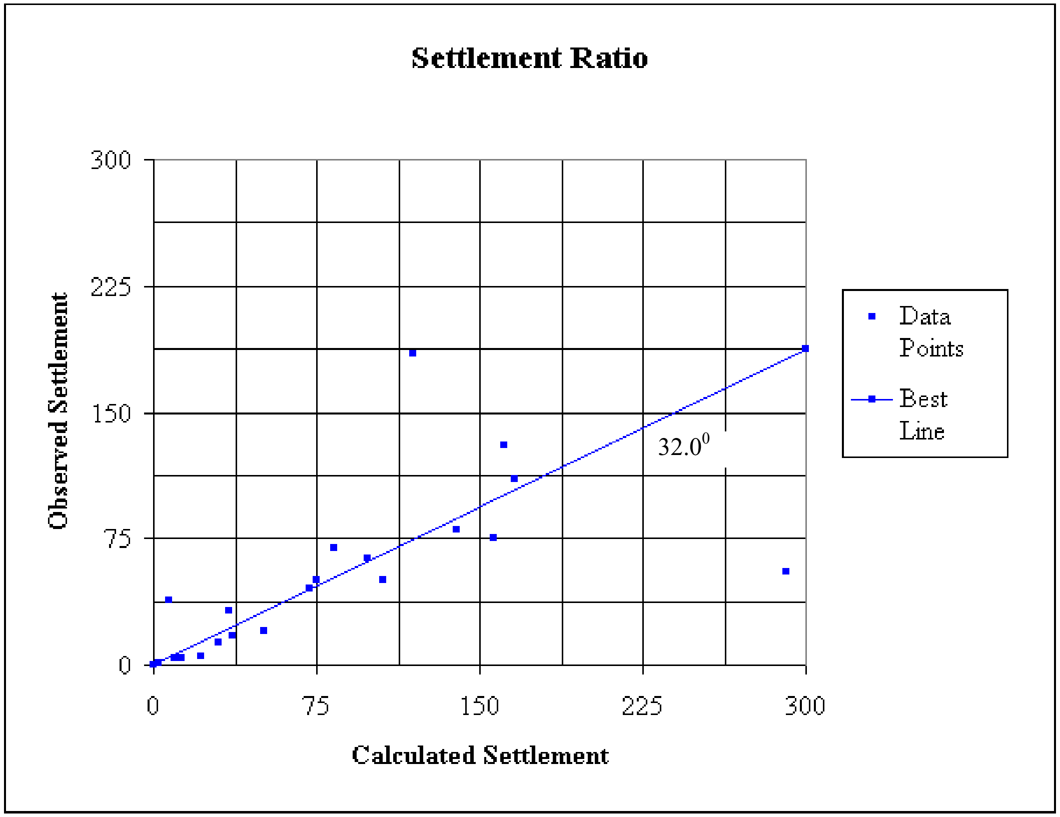

Figure 4.7: Calculated and observed settlement values (p>1000 or p<50) (mm)

Table 4.5: Calculated and observed settlement values for equivalent pier method (friction piles L/re<1) (mm)

	Friction Piles ($L/r_e<1$)	Equivalent Pier						Mea.	(Cal-Mea.)/Mea*100					
		d_{e1} (Equation 2.13-14)			d_{e2} (Figure 2.9)				d_{e1}			d_{e2}		
		A=0	A=5	I (Fig. 2.10)	A=0	A=5	I (Fig. 2.10)		A=0	A=5	I (Fig. 2.10)	A=0	A=5	I (Fig. 2.10)
1	Five-storey B.	4,94	10,84	7,78	10,55	15,89	11,4	12,65	60,9	14,3	38,5	16,6	25,6	9,9
2	American Exp.	59,2	163,7	107,1	148,1	240,2	169,95	55	7,6	197,7	94,6	169,3	336,7	209,0
3	Westend I T.	76,21	145,4	103,6	149	213,1	148,42	110	30,7	32,1	5,8	35,4	93,7	34,9
4	Messeturm T.	60,08	147,4	100,6	141,7	219,2	155,9	130	53,8	13,4	22,7	9,0	68,6	19,9
5	Congress O.	21,60	56,58	38,13	55,65	86,22	61,98	45	52,0	25,7	15,3	23,7	91,6	37,7
6	Congress H.		93,46	58,89	68,42	143,59	99,72	50		86,9	17,8	36,8	187,2	99,4
7	Cambridge R.	24,89	42,42	34,24	42,7	58,88	47,95	27,5	9,5	54,3	24,5	55,3	114,1	74,4
8	19-Storey B.		87,69	52,03		134	82,59	64		37,0	18,7		109,3	29,0
9	Hotel Japan		17,05	11,51		23,08	16,61	17,5		2,6	34,2		31,9	5,1
10	İzmir Hilton	8,71	76,29	52,39	57,31	109,1	78,19	69,6	87,5	9,6	24,7	17,7	56,8	12,3
11	Stonebridge P.	9,47	35,18	24,44	28,21	49,52	38,8	25	62,1	40,7	2,2	12,8	98,1	55,2
12	Dashwood H.	14,79	42,47	30,6	35,59	59,39	48,58	33	55,2	28,7	7,3	7,8	80,0	47,2
13	Ghent Grain		232,6	205,2	176,5	271	235,37	185		25,7	10,9	4,6	46,5	27,2

Table 4.6: Calculated and observed settlement values for friction piles (L/re>1) - equivalent pier method (mm)

	Friction Piles	**Equivalent Pier**						**Mea.**	**(Cal-Mea.)/Mea*100**					
		d_{e1} (Equation 2.13-2.14)			d_{e2} (Fig. 2.9)				d_{e1}			d_{e2}		
		A=0	A=5	I (Fig. 2.10)	A=0	A=5	I (Fig. 2.10)		A=0	A=5	I (Fig. 2.10)	A=0	A=5	I (Fig. 2.10)
1	Field Test	6,01	6,53	4,28	8,94	9,36	6,71	38,1	84,2	82,9	88,8	76,5	75,4	82,4
2	Test of Kaino	8,01	9,01	6,83	9,71	10,59	8,26	3,8	110,8	137,1	79,7	155,5	178,7	117,4
3	Frame-type 2	27,02	29,38	28,99	31,91	33,9	34,05	13	107,8	126,0	123,0	145,5	160,8	161,9
4	Frame-type 3	17,56	18,71	17,58	22,4	23,3	24,86	5	251,2	274,2	251,6	348,0	366,0	397,2
5	9-Pile group	1,44	1,55	0,68	2,58	2,71	1,04	0,9	60,0	72,2	24,4	186,7	201,1	15,6
6	Frame-type 7	11,01	11,98	9,53	15,33	16,13	13,11	4	175,3	199,5	138,3	283,3	303,3	227,8
7	Forum Kastor	64,64	121,15	84,17	126,00	176,22	124,95	75	13,8	61,5	12,2	68,0	135,0	66,6
8	Forum Pollux	77,67	123,17	88,09	141,03	180,96	125,88	80	2,9	54,0	10,1	76,3	126,2	57,4
9	Messe-Torhaus	30,7	45,61	36,71	47,68	61,61	45,91	45	31,8	1,4	18,4	6,0	36,9	2,0
10	Gratham Road	22,5	36,99	30,5	35,19	48,74	42,42	30	25,0	23,3	1,7	17,3	62,5	41,4
11	Molasses Tank	19,27	24,24	14,91	57,82	61,68	27,39	29,5	34,7	17,8	49,5	96,0	109,1	7,2
12	Frame-type 6	63,31	86,17	76,17	97,59	118,45	97,77	19	233,2	353,5	300,9	413,6	523,4	414,6

Table 4.7: Calculated and observed settlement values for end-bearing piles - equivalent pier method (mm)

	End-Bearing Piles	**Equivalent Pier**						**Mea.**	**(Cal-Mea.)/Mea*100**					
		d_{e1} (Equation 2.13-2.14)			d_{e2} (Fig. 2.9)				d_{e1}			d_{e2}		
		A=0	A=5	I (Fig. 2.10	A=0	A=5	I (Fig. 2.10		A=0	A=5	I (Fig. 2.10	A=0	A=5	I (Fig. 2.10
1	Eurotheum B.		10,42	3,88		20,46	6,75	32		67,4	87,9		36,1	78,9
2	Japan Centre		22,24	5,52		47,51	10,39	50		55,5	89,0		5,0	79,2
3	Treptowers B.		57,27	49,22		80,95	66,02	63		9,1	21,9		28,5	4,8
4	New Court I	7,91	21,29	20,55	16,17	27,91	26,43	31,5	74,9	32,4	34,8	48,7	11,4	16,1
5	New Court II	8,99	26,02	25,39	19,24	33,95	32,66	28,1	68,0	7,4	9,6	31,5	20,8	16,2
6	New Court III	2,84	19,61	18,3	12,76	25,87	24,1	25,1	88,7	21,9	27,1	49,2	3,1	4,0
7	Commerz Bank		7,49	5,29		14,44	9,33	17		55,9	68,9		15,1	45,1
8	Main Tower		10,37	4,53		20,08	7,3	20		48,2	77,4		0,4	63,5

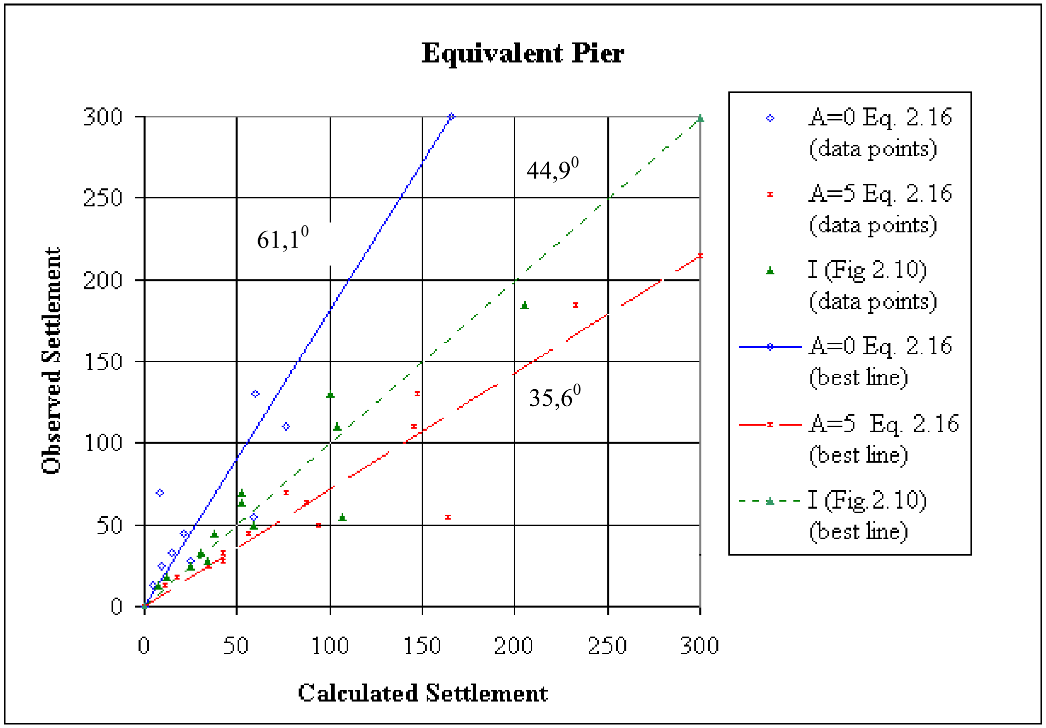

Figure 4.8: Calculated and observed values for friction piles (L/re<1 and de1) (mm)

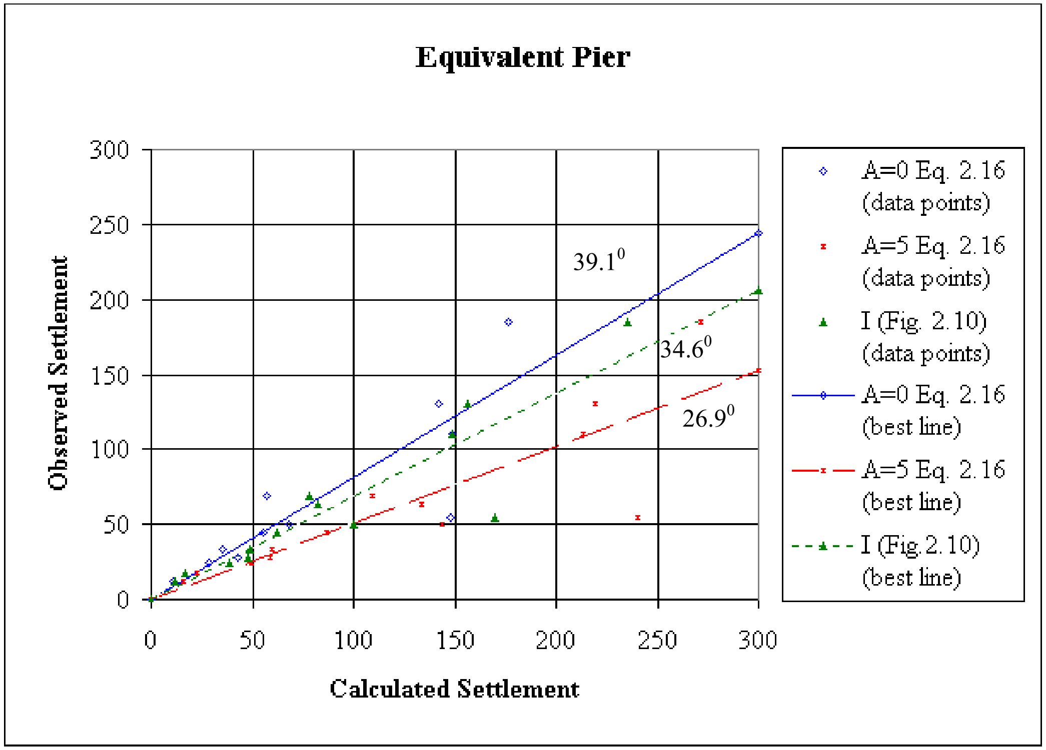

Figure 4.9: Calculated and observed values for friction piles (L/re<1 and de2) (mm)

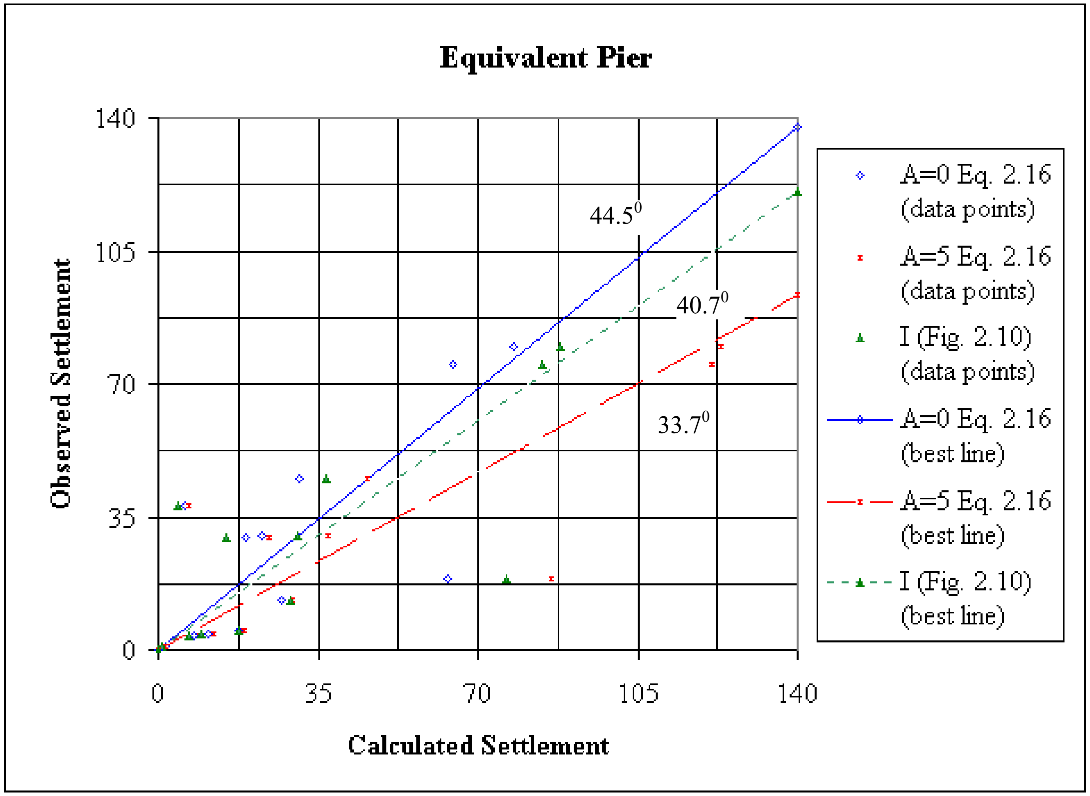

Figure 4.10: Calculated and observed values for friction piles (L/re>1 and de1) (mm)

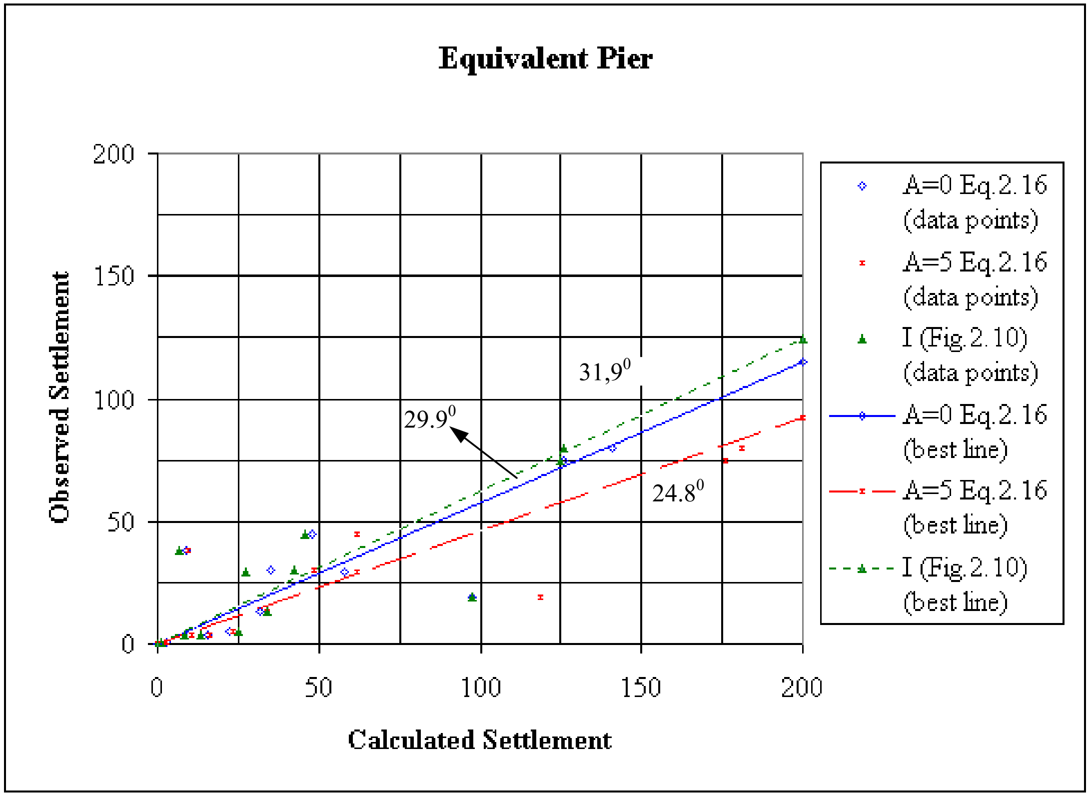

Figure 4.11: Calculated and observed values for friction piles (L/re>1 and de2) (mm)

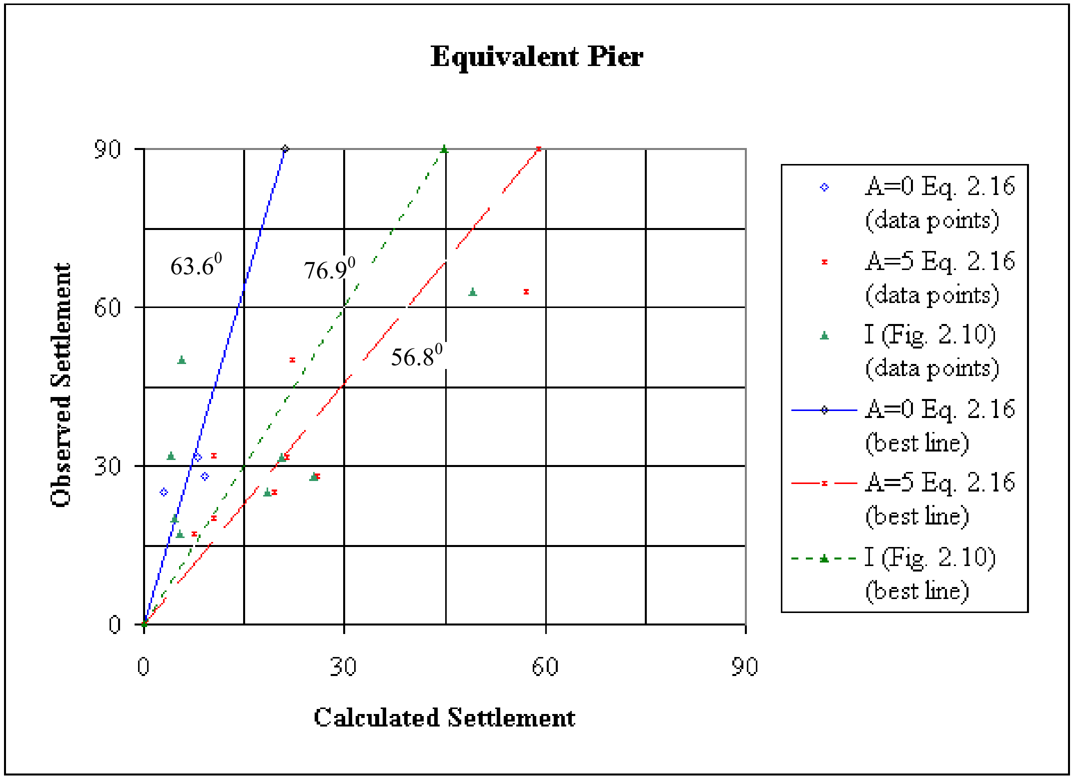

Figure 4. 12: Calculated and observed values for end-bearing piles (de1) (mm)

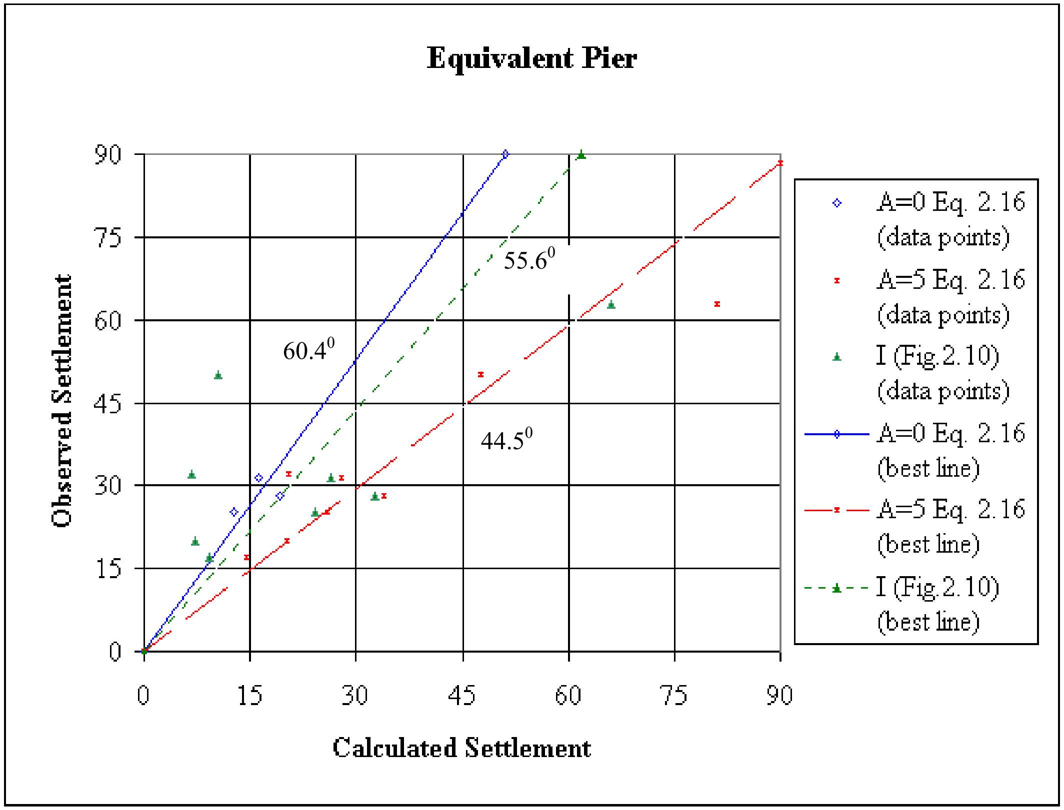

Figure 4.13: Calculated and observed values for end-bearing piles (de2) (mm)

Table 4.8: Calculated and observed settlement values for equivalent raft method (mm)

	Equivalent Raft	Ave.	Mea.	(Cal.-Mea.)/ Mea.*100			Ave.	Mea.	(Cal.-Mea.)/ Mea.*100
1	Field Test	3,56	38,1	90,66	17	Molasses Tank	21,2	29,5	28,14
2	Test of Kaino	3,82	3,8	0,53	18	Messeturm Tower	107,4	130	17,40
3	Frame-type 2	37,92	13	191,69	19	New Law Court II	20,96	31,5	33,46
4	Frame-type 3	23,39	5	367,80	20	New Law Court I	26,5	28,1	5,69
5	9-Pile group	1	0,9	11,11	21	New Law Court III	19,1	25,1	23,90
6	Frame-type 7	18,65	4	366,25	22	Congress C. Office	32,63	45	27,49
7	Five-storey	10,42	12,65	17,63	23	Congress C. Hotel	44,80	50	10,40
8	Eurotheum	44,44	32	38,88	24	Commerz Bank	13,91	17	18,18
9	Japan Centre	28,08	50	43,84	25	Main Tower	20,98	20	4,90
10	Forum Kastor	69,26	75	7,65	26	Cambridge Road	33,45	27,5	21,64
11	Forum Pollux	78,99	80	1,26	27	19-Storey Building	64,24	64	0,37
12	American Express	82,74	55	50,44	28	Hotel Japan	19,69	17,5	12,51
13	Westend I Tower	100,9	110	8,32	29	İzmir Hilton Complex	77,91	69,6	11,94
14	Messe-Torhaus	41,16	45	8,53	30	Frame-type 6	90,03	19	373,84
15	Gratham Road	20,09	30	33,03	31	Stonebridge Park Flats	23,71	25	5,16
16	Treptowers	72,98	63	15,84	32	Dashwood House	28,23	33	14,45
					33	Ghent Grain Terminal	111,8	185	39,55

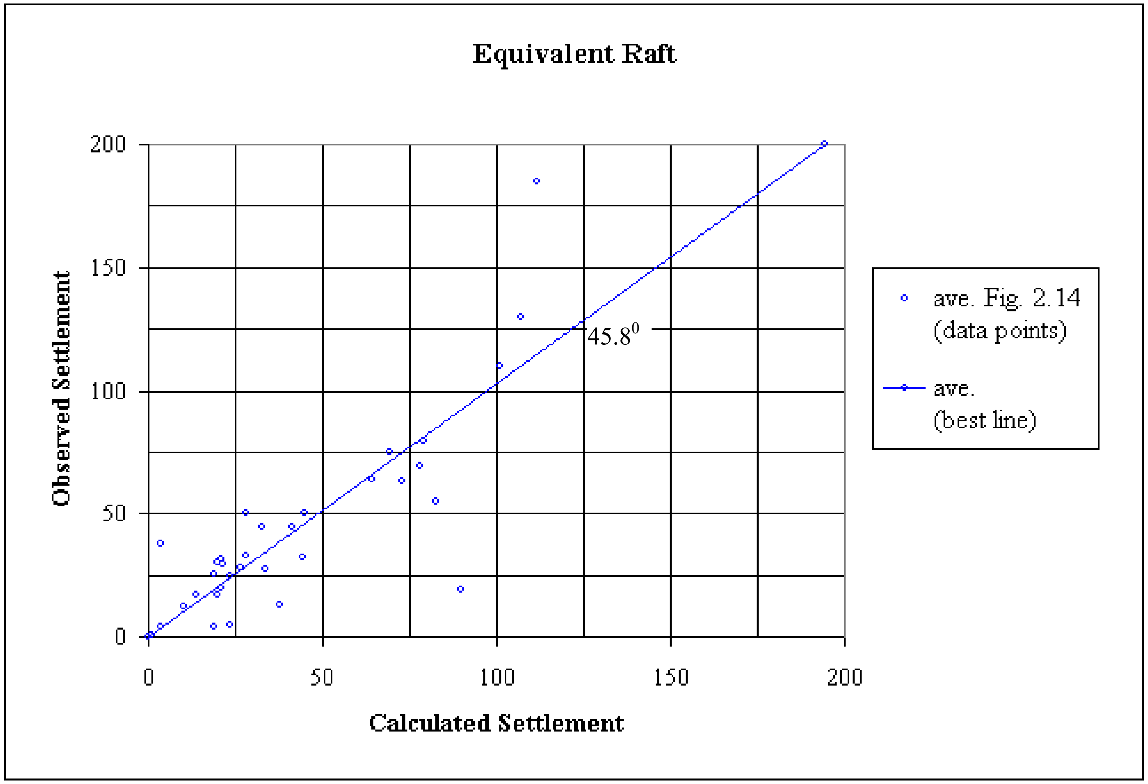

Figure 4.14: Calculated and observed values for equivalent raft method (mm)

Table 4.9: Calculated and observed settlement values for friction and end-bearing piles - equivalent raft method (mm)

	Friction piles	**Ave.**	**Mea.**			**Ave.**	**Mea.**
1	Field Test on five-Pile	3,56	38,1	14	Messeturm Tower	107,38	130
2	Test of Kaino	3,82	3,8	15	New Law Court II	20,96	31,5
3	Frame-type 2	37,92	13	16	New Law Court I	26,5	28,1
4	Frame-type 3	23,39	5	17	New Law Court III	19,1	25,1
5	9-Pile group	1	0,9	18	Congress C. Office	32,63	45
6	Five-storey Building	10,42	12,65	19	Congress C. Hotel	44,80	50
7	Forum Kastor	69,26	75	20	Cambridge Road	33,45	27,5
8	Forum Pollux	78,99	80	21	19-Storey Building	64,24	64
9	American Express	82,74	55	22	Hotel Japan	19,69	17,5
10	Westend I Tower	100,85	110	23	İzmir Hilton	77,91	69,6
11	Messe-Torhaus	41,16	45	24	Stonebridge Park	23,71	25
12	Gratham Road	20,09	30	25	Dashwood House	28,23	33
13	Molasses Tank	21,2	29,5	26	Ghent Grain	111,83	185
	End-bearing piles						
1	Commerz Bank	**13,91**	17	5	Eurotheum	44,44	32
2	Japan Centre	**28,08**	50	6	Treptowers	72,98	63
3	Main Tower	**20,98**	20	7	Frame-type 6	90,03	19
4	Frame-type 7	18,65	4				

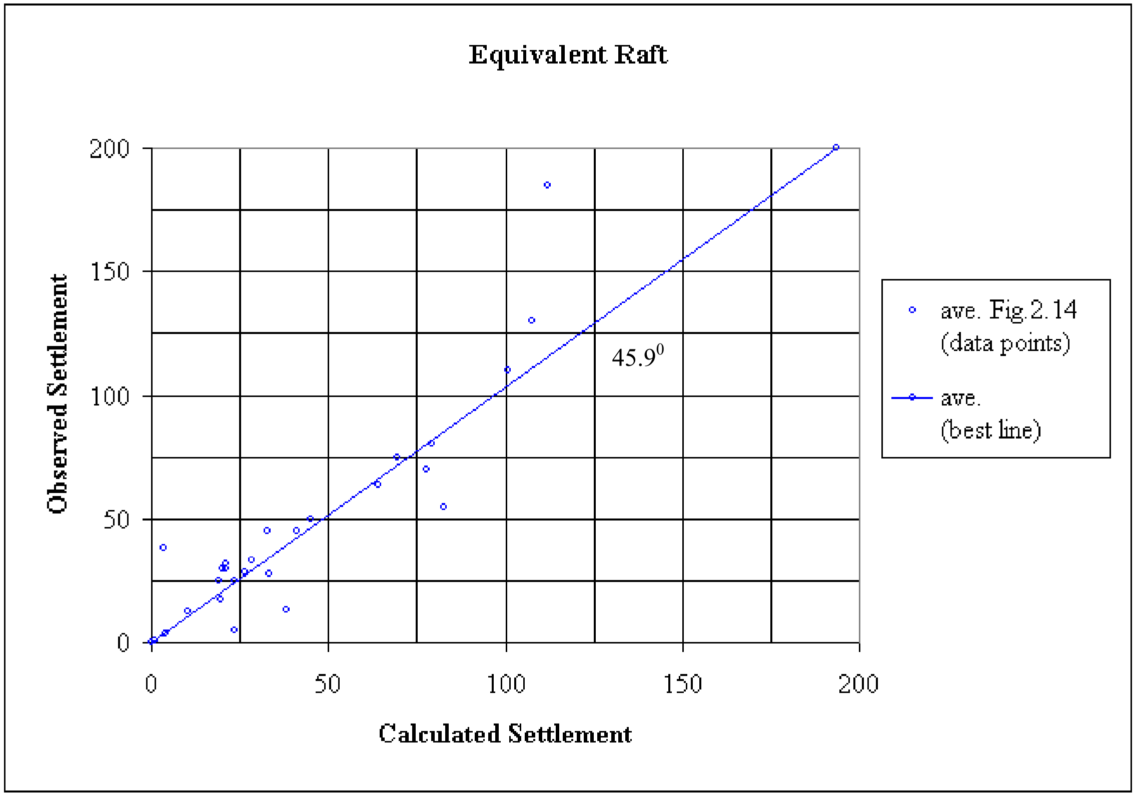

Figure 4.15: Calculated and observed values for friction piles (mm)

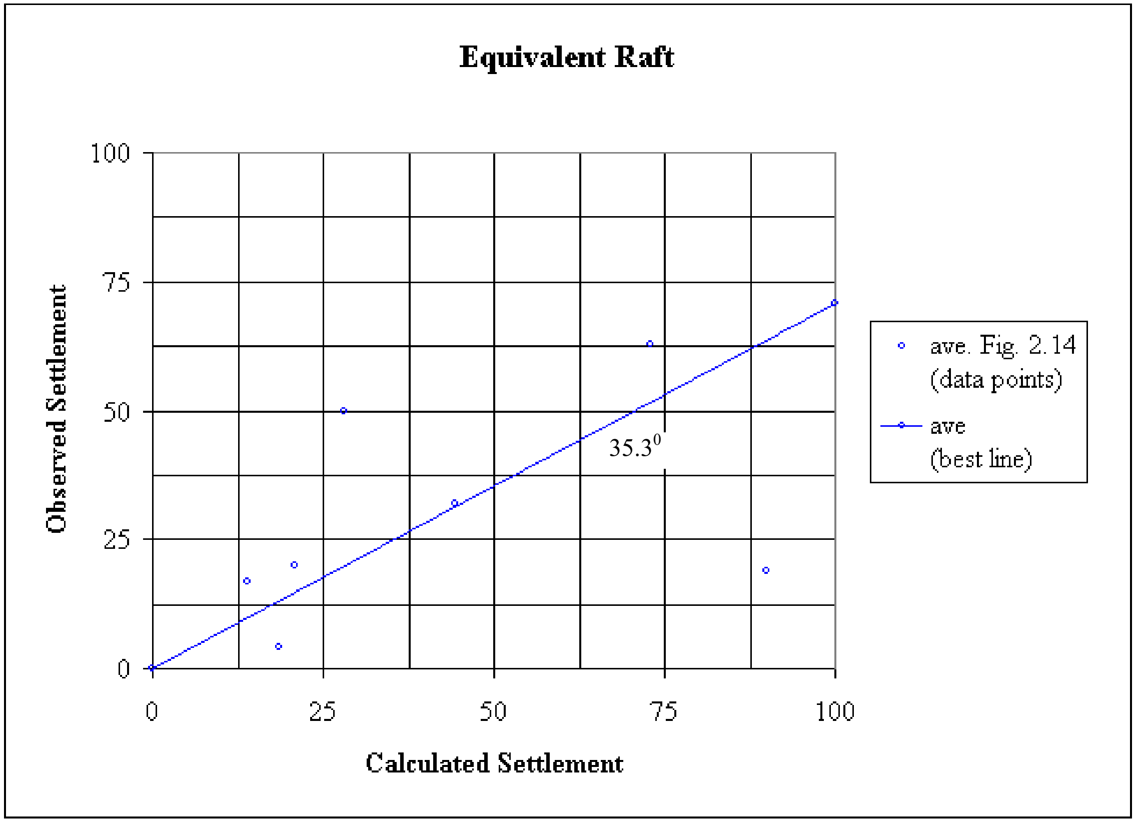

Figure 4. 16: Calculated and observed values for end-bearing piles (mm)

Table 4.10: Calculated and observed settlement values for s/d>4, s/d<4 and s/d<3 - equivalent raft method (for friction piles) (mm)

	Equivalent Raft	**Ave.**	**Mea.**	**s/d**			**Ave.**	**Mea.**	**s/d**
1	Test of Kaino	3,82	3,8	3,5	**1**	Field Test	3,56	38,1	4,05
2	Frame-type 3	23,39	5	3	2	Five-storey	10,42	12,65	7
3	9-Pile group	1	0,9	3	**3**	Forum Kastor	69,26	75	5
4	American Express	82,74	55	3,15	**4**	Forum Pollux	78,99	80	5
5	Messe-Torhaus	41,16	45	3,25	**5**	Westend I Tower	100,85	110	5
6	Stonebridge	23,71	25	3,58	**6**	Gratham Road	20,09	30	4
7	Dashwood House	28,23	33	3,093	**7**	Molasses Tank	21,2	29,5	7
					8	Messeturm Tower	107,38	130	4,75
					9	Congress Office	32,63	45	4,5
					10	Congress Hotel	44,80	50	4,5
1	Frame-type 2	37,92	13	1,8	**11**	Cambridge Road	33,45	27,5	4,5
2	Hotel Japan	19,69	17,5	2	12	19-Storey	64,24	64	6
3	İzmir Hilton	77,91	69,6	2,6	13	Ghent Grain	111,83	185	4

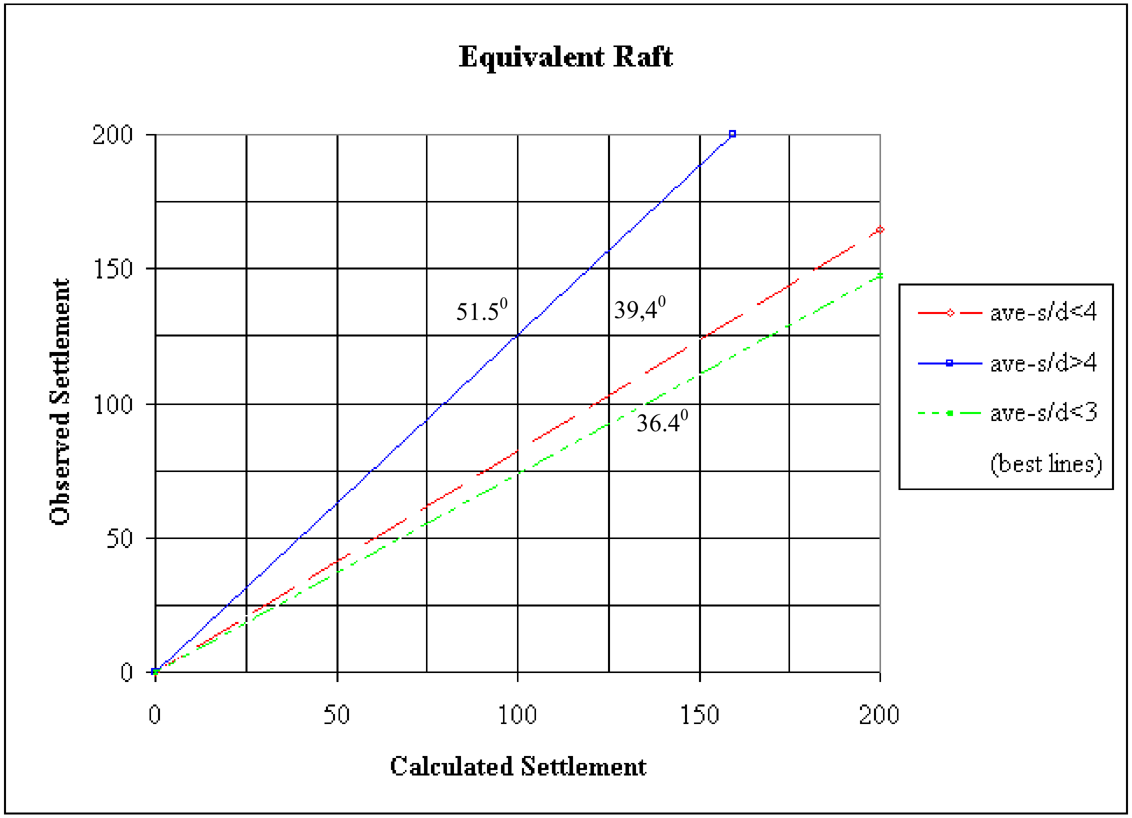

Figure 4.17: Calculated and observed values for friction piles for different s/d (mm)

Table 4.11: Calculated and observed settlement values for different pressure distribution and different raft location - equivalent raft method (for friction piles) (mm)

		Pressure is distributed		If pile raft is located at the tip of the piles				
		4V:1H	8V:1H					
		Ave.	**Ave.**	**Ave.**	**Mea.**	**s/d**	**L_{pile}**	**B_{raft}/L_{pile}**
1	Field Test	3,56	8,47	17,69	38,1	4,05	9,15	0,11
2	Forum Kastor	69,26	75,51	71,74	75	5	25	0,8
3	Forum Pollux	78,99	89,77	84,18	80	5	30	0,66
4	Westend I Tower	100,85	112,58	106,51	110	5	30	1,07
5	Gratham Road	20,09	22,82	23,55	30	4	17,45	1,1
6	Molasses Tank	21,2	27,89	27,71	29,5	7	27	0,3
7	Messeturm Tower	107,38	120,88	113,75	130	4,75	30,9	1,68
8	Congress Office	32,63	35,38	33,52	45	4,5	28	1,26
9	Congress Hotel	44,80	48,85	43,93	50	4,5	28	2
10	Cambridge Road	33,45	37,3	30,43	27,5	4,5	15,3	1,04
	Angle	**50,21°**	**46,71°**	**48,03°**				

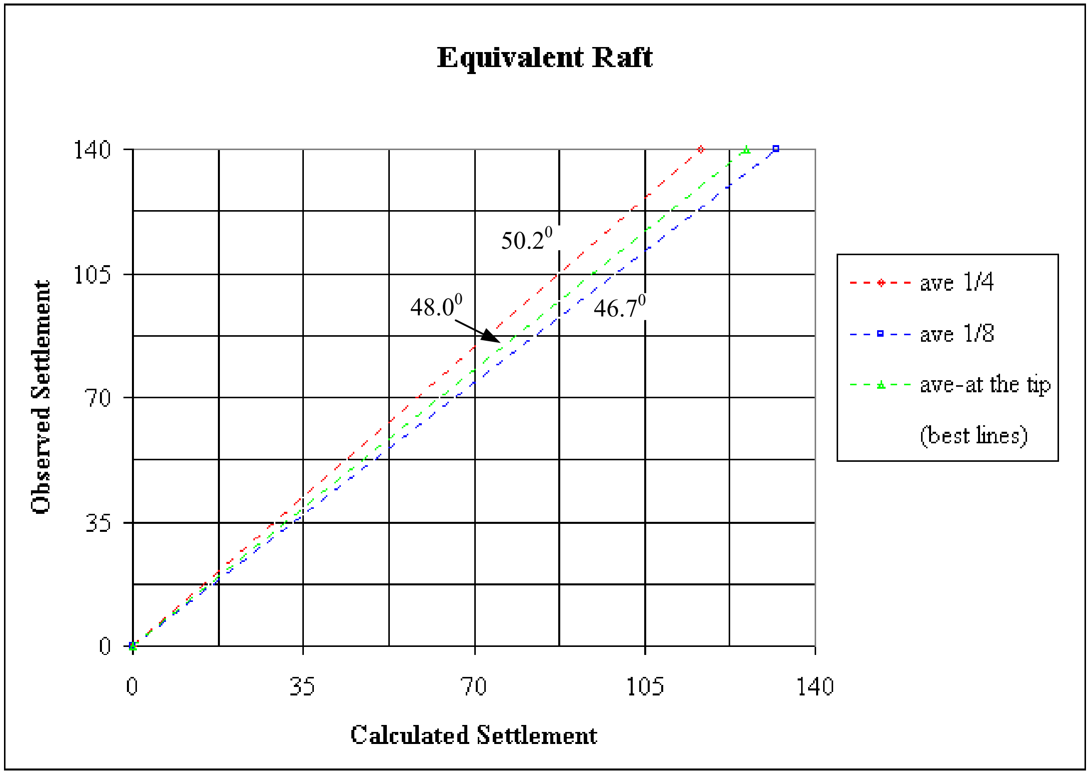

Figure 4.18: Calculated and observed values for friction piles for different pressure distribution and different raft location (mm)

Table 4.12: Calculated and observed settlement values for all methods (mm)

		Settlement Ratio	Equivalent Pier						Equivalent Raft	Mea.
			d_{e1} (Equation 2.13-2.14)			d_{e2} (Fig. 2.9)				
	Friction Piles		A=0	A=5	I (fig 2.10)	A=0	A=5	I (fig 2.10)	Ave	
1	Field Test	7,59	6,01	6,53	4,28	8,94	9,36	6,71	3,56	38,1
2	Test of Kaino	9,48	8,01	9,01	6,83	9,71	10,59	8,26	3,82	3,8
3	Frame-type 2	29,61	27,02	29,38	28,99	31,91	33,9	34,05	37,92	13
4	Frame-type 3	21,58	17,56	18,71	17,58	22,4	23,3	24,86	23,39	5
5	9-Pile group	2,3	1,44	1,55	0,68	2,58	2,71	1,04	1	0,9
6	Five-storey	13,7	4,94	10,84	7,78	10,55	15,89	11,4	10,42	12,7
7	Forum Kastor	156,40	64,64	121,15	84,17	126,00	176,22	124,95	69,26	75
8	Forum Pollux	139,59	77,67	123,17	88,09	141,03	180,96	125,88	78,99	80
9	American	291,43	59,2	163,72	107,05	148,1	240,16	169,95	82,74	55
10	Westend	165,73	76,21	145,36	103,58	148,95	213,09	148,42	100,85	110
11	Messe-Torhaus	47,48	30,7	45,61	36,71	47,68	61,61	45,91	41,16	45
12	Gratham Road	32,84	22,5	36,99	30,5	35,19	48,74	42,42	20,09	30
13	Molasses Tank	25,34	19,27	24,24	14,91	57,82	61,68	27,39	21,2	29,5
14	Messeturm	161,11	60,08	147,43	100,55	141,7	219,2	155,9	107,38	130
15	Congress Office	71,46	21,60	56,58	38,13	55,65	86,22	61,98	32,63	45

16	Congress Hotel	105,45		93,46	58,89	68,42	143,59	99,72	44,80	50
17	Cambridge	31,42	24,89	42,42	34,24	42,7	58,88	47,95	33,45	27,5
18	19-Storey	77,12		87,69	52,03		133,98	82,59	64,24	64
19	Hotel Japan	17,14		17,05	11,51		23,08	16,61	19,69	17,5
20	İzmir Hilton	83,3	8,71	76,29	52,39	57,31	109,1	78,19	77,91	69,6
21	Stonebridge	29,34	9,47	35,18	24,44	28,21	49,52	38,8	23,71	25
22	Dashwood	35,29	14,79	42,47	30,6	35,59	59,39	48,58	28,23	33
23	Ghent Grain	119,14		232,57	205,22	176,49	271,02	235,37	111,83	185
	End-Bearing Piles									
1	Frame-type 7	13,26	11,01	11,98	9,53	15,33	16,13	13,11	18,65	4
2	Eurotheum	35,05		10,42	3,88		20,46	6,75	44,44	32
3	Japan Centre	74,89		22,24	5,52		47,51	10,39	28,08	50
4	Treptowers	98,4		57,27	49,22		80,95	66,02	72,98	63
5	New Court II	30,56	7,91	21,29	20,55	16,17	27,91	26,43	20,96	31,5
6	New Court I	36,67	8,99	26,02	25,39	19,24	33,95	32,66	26,5	28,1
7	New Court III	29,09	2,84	19,61	18,3	12,76	25,87	24,1	19,1	25,1
8	Commerz Bank	36,41		7,49	5,29		14,44	9,33	13,91	17
9	Main Tower	51,1		10,37	4,53		20,08	7,3	20,98	20
10	Frame-type 6	79,69	63,31	86,17	76,17	97,59	118,45	97,77	90,03	19

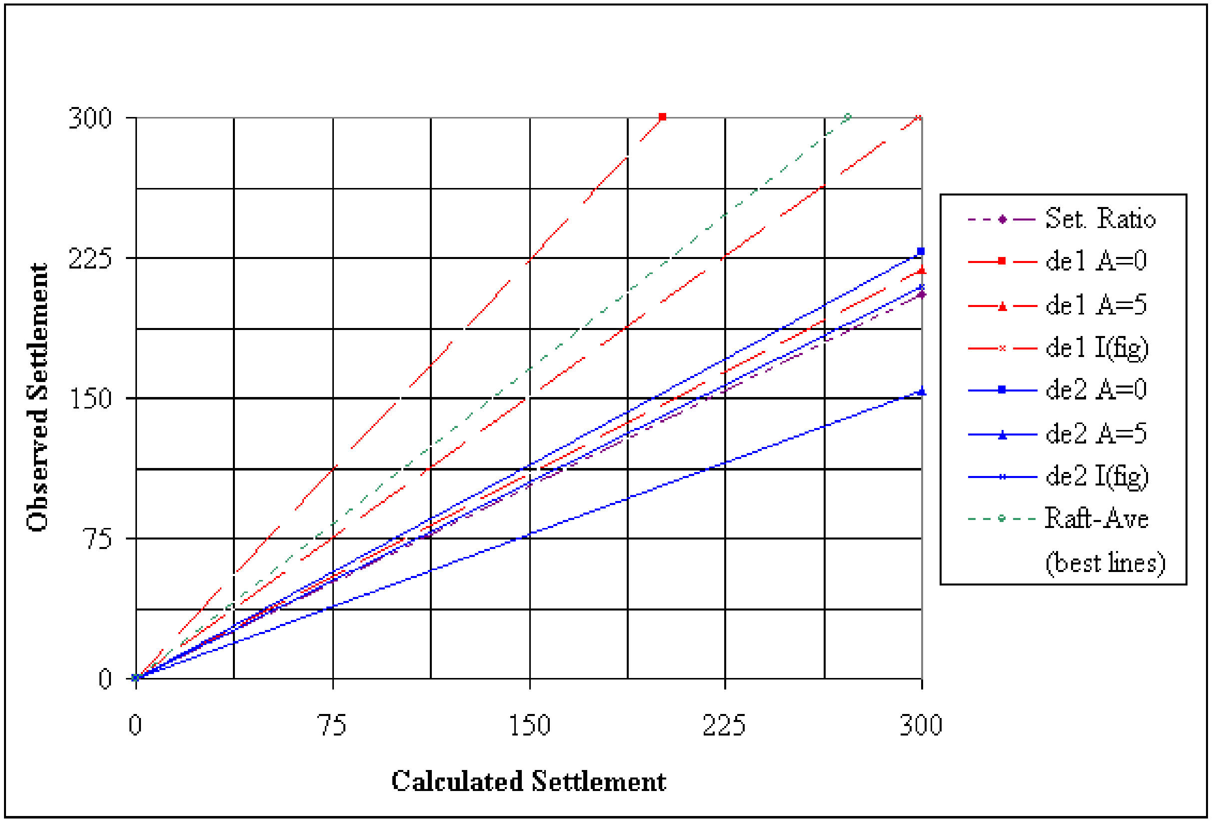

Figure 4.19: Calculated and observed values for friction piles - All methods (mm)

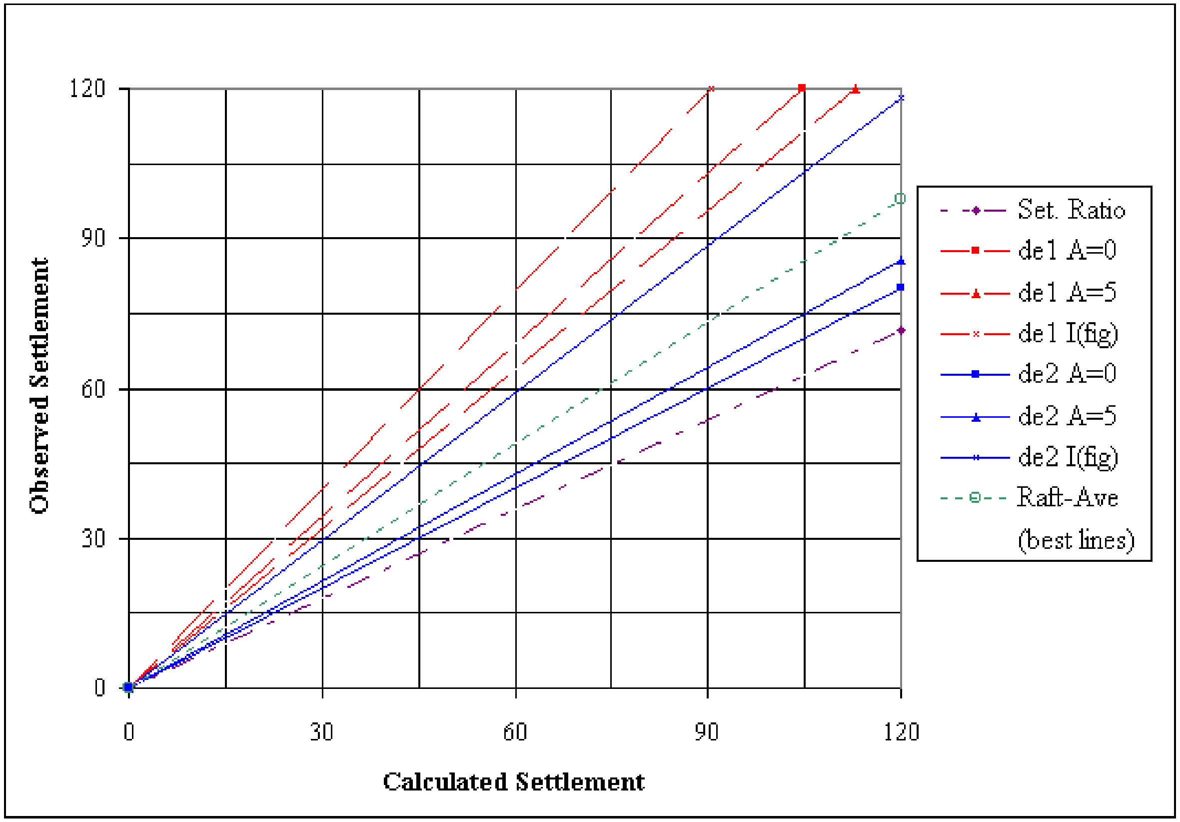

Figure 4.20: Calculated and observed values for end-bearing piles - All methods (mm)

REFERENCES

Bartolomey, A.A., et al, (1981), "Pile foundation Settlement" Proc. Tenth Int. Conf. Soil Mech. Found. Engng., Stockholm, Sweden, Roterdam; A.A Balkema, Vol. 2, pp 611-614.

Birand, A., (2001), "Kazıklı Temeller", Teknik Yayınevi, Ankara, pp 118-164.

Blanchet, R., Tavenas, F., and Garneau, R., (1980), "Behaviour of friction piles in soft sensitive clays" Can. Geotech. J., 17, pp 203-224.

Butler, F.G., (1974), "General report and state-of-the-art review, Session 3, Proceedings of the Conference on Settlement of Structures, Cambridge, Pentech Press, London, 1975, pp 531-578.

Chow, Y.K., (1986), "Analysis of vertically-loaded pile group.", Int. J. Num. and Anal. Methods in Geomechanics, 10(1), pp 59-72.

Christian, J.T. and Carrier, W.D., (1978), "Janbu, Bjerrum and Kjaernsli's chart reinterpreted.", Canadian Geotechnical Journal, Vol.15, pp 123-128.

Ergun, M.U., (1995), "Prediction and observed settlement of a piled raft foundation" Eleventh African Regional Conference on Soil Mechanics and Foundation Engineering, Cairo, Egypt, Vol.3, pp 49-61.

Fleming, W.G.K. et al, (1992), " Piling Engineering", 2nd Ed., Blackie-John Wiley and Sons. Inc., pp 92-214.

Fox, E.N., (1948), "The mean elastic settlement of a uniformly loaded area at a depth below the ground surface", Proceedings of the 2nd International Conference, ISSMFE, Rotterdam, Vol.1, pp 129-132.

Guo, W.D., and Randolph, M.F., (1999), "An efficient approach for settlement prediction of pile groups." Geotechnic 49, No.2, pp 161-179.

Horikoshi, K., and Randolph, M.F., (1999), "Estimation of overall settlement of piled rafts" Soils and Foundations, Japanese Geotechnical Society, Vol 39, No. 2, pp 59-68.

Polo, M. and Clemente, M., (1998) "Pile-group settlement using independent shaft and point loads." Journal of Geotechnical Engineering ASCE, Vol 114, No.4, pp 469-487.

Katzenbach, R., Arslan, U., and Moormann, C., (2000), “Piled raft foundation projects in Germany” Design Applications of Raft Foundations, Hemsley, J.A.; Thomas Telford, London, pp 323-391.

Lee, C.Y., (1993), “Pile group settlement analysis by hybrid layer approach” Journal of Geotechnical Engineering ASCE, Vol 119, No.6, pp 984-997.

Majima, M., and Nagao, T., (2000), “Behaviour of piled raft foundation for tall building in Japan” Design Applications of Raft Foundations, Hemsley, J.A.; Thomas Telford, London, pp 393-411.

Mandolini, A., and Viggiani, C., (1997), “Settlement of piled foundations” Geotechnique 47, No.4, pp 791-816.

Morton, K., and Au, E., (1974), “Settlemet observations on eight structures in London” Con. ‘Settlement of Structures’, Brit. Geotechn. Soc., Cambridge, UK, London: Pentech Pres, pp 183-203.

Ordemir, İ., (1984), “Pile Foundations, Metu Department of Civil Engineering”, Ankara, pp 83-85.

Ottoviani, M., (1975) “Three-dimensional finite element analysis of vertically loaded pile groups.” Geotechnique 25, No. 2, pp 159-174.

Poulos, H.G., (1968), “Analysis of settlement of pile groups.” Geotechnique 18, No.4, p 449-471.

Poulos, H.G., (1972), “Load-settlement prediction for piles and piers.” J. Soil Mech. Found. Div., Proc. ASCE, 98 (SM9), p 879-897.

Poulos, H.G., (1989), “Pile behaviour-theory and application” Geotechnic 39, No.3 pp 365-415.

Poulos, H.G., (1993), “Settlement prediction for bored pile groups.” Deep Foun. On Bored and Auger piles, A.A. Balkema, Rotterdam, The Netherlands, pp 103-117.

Poulos, H.G., (2000), “Practical design procedures for piled raft foundations” Design Applications of Raft Foundations, Hemsley, J.A.; Thomas Telford, London, pp 425-467.

Poulos, H.G., (2001), “Piled raft foundations: design and applications” Geotechnique 51, No.2, pp 95-113.

Poulos, H.G., and Davis, E.H., (1980), “Pile Foundation Analysis and Design”, John Wiley and Sons, New York, New York, pp 71-142.

Randolph, M.F., (1994), "Design methods for pile groups and piled rafts." Thirteenth International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India, Volume 5, pp 61-82.

Randolph, M.F., and Guo, W.D.. (1999), "An efficient approach for settlement prediction of pile groups." Geotechnique 49, No. 2, pp 161-179.

Randolph, M.F., and Wroth, C.P. (1978), "Analysis of deformation of vertically loaded piles." J. Geotech. Engrg. Div. ASCE. 104,(12), pp 1465-1488.

Randolph, M.F., and Wroth, C.P. (1979), "An analysis of vertical deformation of pilegroups." Geotechnique 29, No. 4, pp 423-439.

Shen, W.Y., Chow, Y.K., and Yong, K.Y., (2000), "Practical metjod for settlement analysis of pile groups" Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No.10, pp 890-897.

Skempton, M.A. and Bjerrum, L., (1957), "A contribution to the settlement analysis of foundation on clay" Geotechnique 7, No. 4, pp 168-178.

Tomlinson, M.J., (1986), "Foundation Design and Construction", 5th Ed., Longman Scientific and Technical, Harlow, Essex, England.

Yamashita, K. et al, (1993), "Settlement behaviour of a five-story building on a piled raft foundation." Deep Foun. On Bored and Auger piles, A.A. Balkema, Rotterdam, The Netherlands, pp 351-356.

APPENDIX

CASE HISTORIES

1. Field Test on Five-Pile Group, San Francisco (n=5)

In the framework of an investigation on the behaviour of piles in sand, load tests to failure were performed on a single pile and on a five-pile group. The piles were closed-end steel pipes, 273 mm in diameter, driven to a depth of 9.15 m below ground surface through a 300 mm diameter predrilled to a depth of 1.37 m. The piles of the group were connected by a rigid reinforced concrete cap, clear of the ground.

At the test site, the subsoil consists of a hydraulic fill made of clean sand, about 11 m thick, overlain by 1.4 m of sandy gravel and underlain by sand with interbedded layers of stiff clay down to the bedrock found at a depth of around 14.3 m below ground surface. The average settlement is calculated as 36,8 mm by using non-linear analysis by the program GRUPPALO. Using linear elastic analysis average settlement is estimated as 2,6 mm (Randolph (1994)). (Mandolini, A., and Viggiani, C., 1997)

a) Settlement Ratio Method

n= 5 d= 0,273 m r_0= 0,1365 m

L= 9,15 s/d_{ave}= 4,05 m

E_1=92,2 MN/m^2 E_p= 20000 MN/m^2

υ_s= 0,3 υ_s=0,4 Clean Sand

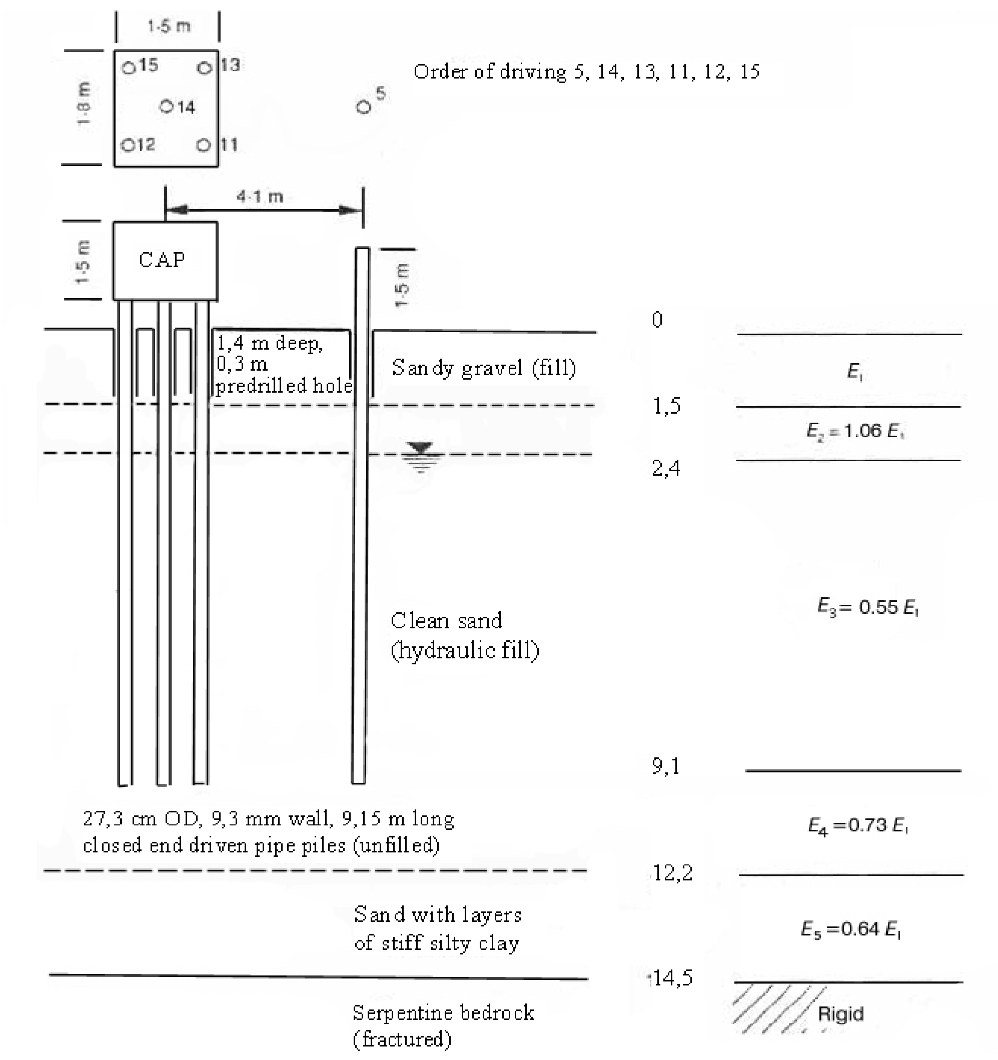

Figure A.1: Layout of the test and subsoil profile (Mandolini and Viggiani, 1997)

$P=2450$ KN

$\lambda=E_p/G_l=20000/22{,}13 \approx 903{,}75$

$\rho=G_{l/2}/G_l=0{,}763\rightarrow1$

$\log\lambda=2{,}956\rightarrow0{,}98$

$s/d=4{,}05\rightarrow0{,}93$

$L/d=33{,}52\rightarrow0{,}55$

$\upsilon_s=0{,}3\rightarrow1$

$\upsilon_s=0{,}4\rightarrow0{,}97$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln\{[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

$P_{single}=2450/5=490$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_tG_lr_0)$
$\upsilon_s=0{,}3$	0,501	0,446	2,240	4,495	1,487	40,690	45,427
$\upsilon_s=0{,}4$	0,486	0,457	2,187	4,341	1,513	40,195	46,689

	P_t/w_t	$K=n\eta_wk$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_sR_s$
$\upsilon_s=0{,}3$	137,225	306,219	8,00	3,57	8,00
$\upsilon_s=0{,}4$	141,037	322,435	7,59	3,47	7,59

$\delta_{measured}=38{,}1$mm

b) Equivalent Pier Method

$B=A_G^{0.5}=1{,}643$

$A_P=\Pi d^2 n/4=0{,}2926\ m^2$

$E_p=20000$ Mpa

$E_s'=57{,}55$ MPa $E_u=66{,}41$ MPa

$d_e=1{,}27\ A_G^{0{,}5}=2{,}087$ m (for friction piles)

$\rho=0{,}763$ L=9,15 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

Method 1

	E_e	λ	ζ	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}3$	2219,277	100,253	2,460	0,789	3,654	0,295	6,02
			2,816	0,738	3,730	0,320	6,53
$\upsilon_s=0{,}4$	2223,225	100,431	2,306	0,814	3,617	0,297	5,63
			2,710	0,751	3,710	0,327	6,19

Method 2

$L/d_e=9{,}15/2{,}086=4{,}38\rightarrow I_\delta=0{,}21$ (Fig. 2.10)

	$\upsilon_s=0{,}3$	$\upsilon_s=0{,}4$
δ (mm)	4,28	3,97

K ≈ 300 (pile stiffness factor) s/d ≈ 4,05 L/d ≈ 33,51 B=1,643 m

$d_e/B\approx 0{,}81$ assumed, then $d_e\approx 1{,}33$ m (Fig. 2.9)

Method 1

	E_e	λ	ζ	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,3	2219,27	100,25	2,910	1,138	4,915	0,279	8,94
			3,151	1,094	5,016	0,292	9,36
υ_s=0,4	2223,225	100,431	2,756	1,168	4,846	0,286	8,514
			3,032	1,114	4,970	0,302	8,987

Method 2

L/d_e=9,15/1,33=6,87 → I_δ=0,21 (Fig. 2.10)

	υ_s=0,3	υ_s=0,4
δ (mm)	6,71	6,23

$\delta_{measured}$=38,1 mm

c) Equivalent Raft Method

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E$

B	L	q_n	L/B	H	D/B	H/B	μ_0	μ_1	δ_i
4,07	4,277	140,745	1,05	3	1,49	0,73	0,905	0,26	2,65
7,534	7,691	42,282	1,02	3,1	1,20	0,41	0,91	0,14	0,56
11,11	11,27	19,567	1,01	2,3	1,09	0,20	0,91	0,05	0,16
									3,22 mm

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$D/(LB)^{0,5}=1,46 \rightarrow \mu_d=0,665$

For sand with layers of stiff silty clay

z/B	σ_z/q	σ_z	H	m_υ	μ_g	μ_d	δ_c (mm)
1,781	0,11	15,48	2,3	0,0145	1	0,665	0,344

$\delta_T=\delta_i+\delta_c=3,22+0,344=$ **3,56 mm**

$\delta_{measured}=38,1$ mm

Table A.1: Measured and computed settlements for Field Test on Five Pile Group (mm)

	Settlement (mm)									
	Set. Ratio	**Equivalent Pier**				**Equivalent Raft**				**Mea.**
		d_{e1}		d_{e2}		H=8,4 m	H=5,3 m (at the pile tip)	H=8,4 m (1/6)	H=8,4 m (1/8)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	Ave.	Ave.	
υs=0,3	8,00	6,02 6,53	4,28	8,94 9,36	6,71	3,56	17,69	5,92	8,47	38,1
υs=0,4	7,59	5,63 6,19	3,97	8,51 8,98	6,23	3,42	17,13	5,77	8,27	

Field Test on Five Pile Group

Rs 8 Mea. 38,1

Pier 6,02
4,28

Raft 3,56

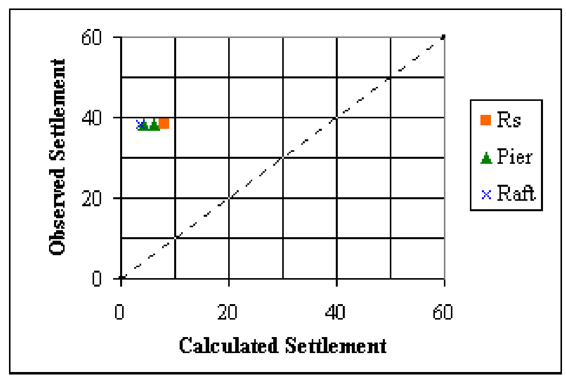

Figure A.2: Measured and computed settlements for Field Test on Five Pile Group (mm)

2. Test of Kaino and Aoki (n=5)

The soil profile consisted of layers of alluvial clay underlain by interbedded sand and clay layers. The modulus values varied from about 12 Mpa near the surface to about 74 Mpa along the lower parts of the pile, while the value at the pile tip was taken as 38 Mpa. The piles were 24 m long and 1 m diameter and were constructed using the reverse circulation method. The interaction factor method was used to analyse the settlement using the program DEFPIG (non-linear), and the group settlement under a load of 6.66 MN was computed to be 3.9 mm. (H.G. Poulos, 1993)

a) Settlement Ratio Method

n= 5 d= 1 m r_0= 0,5 m

L=24 m s/d_{ave}= 3,5 s=3,5 m

E_p= 30000 MN/m^2

P=6,66 MN υ_s= 0,3 υ_s=0,4

$\lambda=E_p/G_l=30000/14,6 \approx 2054,79$

$\rho=G_{l/2}/G_l=0,657 \rightarrow 0,975$

$\log\lambda=3,31 \rightarrow 1,05$

$s/d=3,5 \rightarrow 0,975$

$L/d=24 \rightarrow 0,542$

$\upsilon_s=0,3 \rightarrow 1$

$\upsilon_s=0,4 \rightarrow 0,97$

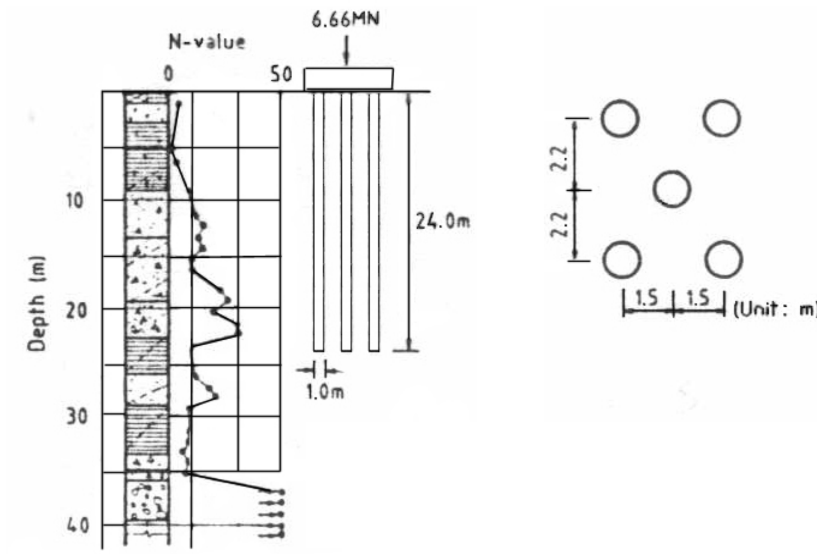

Figure A.3: The soil profile and the pile group configuration (Poulos, 1993)

$\eta_w=n^{-e}$ $R_s=n^e$ $\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et.al, 1992)

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

$P_{single}=6660/5=1332$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_tG_lr_0)$
$\upsilon_s=0,3$	0,541	0,418	2,388	4,012	0,747	40,68	45,96
$\upsilon_s==0,4$	0,524	0,429	2,327	3,857	0,762	40,449	47,984

	P_t/w_t	$K=n\eta_wk$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_sR_s$
$\upsilon_s=0,3$	335,51	702,32	9,48	3,97	9,48
$\upsilon_s=0,4$	350,28	752,65	8,84	3,80	8,84

$\delta_{measured}$=3,8 mm

b) Equivalent Pier Method

$B=A_G^{0.5}$ =4,647 m $A_P=\Pi d^2 n/4$=3,926 m^2

E_p=30000 MPa E_s'=37,96 MPa E_u=43,8 MPa

$d_e=1,27\ A_G^{0,5}$ =5,902 m (for friction piles)

ρ=0,657 L=24 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et.al, 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,3	5485,213	375,699	2,236	0,396	3,865	0,269	8,01
			2,664	0,363	3,896	0,303	9,01
υ_s=0,4	5487,602	375,863	2,082	0,411	3,851	0,266	7,34
			2,566	0,370	3,889	0,304	8,40

Method 2

L/d_e=24/5,9 =4,066→ I_δ=0,23 (Fig. 2.10)

	υ_s=0,3	υ_s=0,4
δ (mm)	6,83	6,34

$K \approx 700$ (pile stiffness factor) $s/d \approx 3,5$ $L/d \approx 24$ $B=4,647$ m

$d_e/B \approx 0,96$ assumed, then $d_e \approx 4,46$ m

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0,3$	5485,213	375,699	2,516	0,495	4,980	0,246	9,71
			2,855	0,464	5,024	0,269	10,59
$\upsilon_s=0,4$	5487,602	375,863	2,362	0,510	4,957	0,246	8,99
			2,748	0,473	5,011	0,272	9,94

Method 2

$L/d_e=24/4,46=5,38 \rightarrow I_\delta=0,21$ (Fig. 2.10)

	$\upsilon_s=0,3$	$\upsilon_s=0,4$
δ (mm)	8,26	7,67

$\delta_{measured}=3,8$ mm

c) Equivalent Raft Method

P=6660 KN

$q_n=P/(BL)$

$\delta_i=\mu_1\mu_0 q_n B/E_u$

B	L	H	H/B	L/B	D/B	E_u	q_n	μ_0	μ_1	δ_i (mm)
12	13,4	7	0,58	1,11	1,33	38,60	41,41	0,91	0,21	2,461
20	21,4	3	0,15	1,07	1,15	45,55	15,56	0,91	0,04	0,249
23,4	24,8	3	0,12	1,06	1,11	48,45	11,47	0,92	0,03	0,153
26,8	28,2	5	0,18	1,05	1,08	67,50	8,81	0,92	0,05	0,161
										3,023

$(LB)^{0,5}/D= 0,79 \rightarrow m_d=68$

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

for sand $q_c/N=5$ $M_0=2q_c+20$

$\delta_c=m_\upsilon\, \sigma_z\, H\, m_d\, m_g$

z/B	σ_z/q	σ_z	m_υ	m_g	m_d	δ_c
0,29	0,68	28,16	0,02	1	0,68	2,68 (sand)
0,70	0,39	16,15	0,019	0,85	0,68	0,52 (clay)
0,95	0,28	11,59	0,033	1	0,68	0,78 (sand)
1,29	0,18	7,45	0,013	0,85	0,68	0,27 (clay)
						$\Sigma\delta_c$ =4,26
						$\Sigma\delta_c$ =0,798

$\delta_T=\delta_i+\delta_c$

δ_T= 0,798+3,051= 3,82 mm

$\delta_{measured}$=3,8 mm

Table A.2: Measured and computed settlements for Test of Kaino (mm)

	Settlement (mm)						
	Set. Ratio	Equivalent Pier				Eq. Raft	Mea.
		d_{e1}		d_{e2}			
		Met1	Met2	Met1	Met2	Ave.	
υs=0,3	9,48	8,02	6,83	9,71	8,26	3,82	3,80
		9,01		10,59			
υs=0,4	8,84	7,34	6,34	8,99	7,67		
		8,40		9,94			

Test of Kaino

Rs	9,48	Mea.	3,8
Pier	8,02		
	6,83		
Raft	3,82		

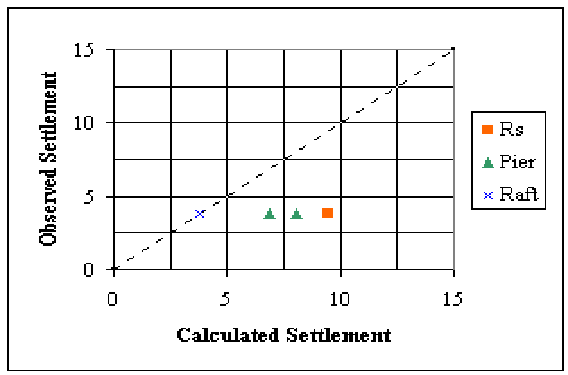

Figure A.4: Measured and computed settlements for Test of Kaino (mm)

3. Frame-Type Building 2 (n=6)

The total structural load of 15 MN is supported on a piled foundation which has 6 filling piles with a diameter of 1000 mm and length of 15,5 m. The distance between piles is 1,8 m. The size of groups in plane is 4,8*3 m. At the site of the building tough plastic clay, E_u=35 Mpa, is located. Different formulations are used to obtain settlement value. Settlement predictions and methods are given below:

USSR standarts	Poulos	Vesic	Skempton	Bartolomey:
48 mm	38 mm	5 mm	14 mm	11 mm

(Bartolomey, A.A., Yushkov, B.S., Leshin, G.M., Khanin, R.E., Kolesnik, G.S., Mulyukov, E.I., Doroshkevitch, N.M., 1981)

a) Settlement Ratio Method

n= 6 d= 1 m r_0= 0,5 m

L= 15,5 m s= 1,8 m

G=11,7 (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,15 υ_s =0,3 Tough Plastic Clays

P=15 MN

$\lambda=E_p/G_l=30000/11{,}7 \approx 2564{,}103$

$\rho=G_{l/2}/G_l=1 \rightarrow 1{,}06$

$\log\lambda=3{,}408 \rightarrow 1{,}065$

$s/d=1{,}8 \rightarrow 1{,}09$

$L/d=15,5 \rightarrow 0,525$

$\upsilon_s=0,15 \rightarrow 1,035$

$\upsilon_s=0,3 \rightarrow 1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al.,1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

$P_{single}=15000/6=2500$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0,15$	0,668	0,301	3,313	4,188	0,423	29,273	47,809
$\upsilon_s=0,3$	0,646	0,314	3,181	3,994	0,433	29,195	50,600

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,15$	279,687	506,447	29,61	8,938	29,61
$\upsilon_s=0,3$	296,012	558,168	26,87	8,445	26,87

$\delta_{measured}=13$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=3,794$

$A_P=\Pi d^2 n/4=4,712\ m^2$

$E_p=30000$ MPa

E_s'=26,833 MPa E_u=35 MPa

d_e=1,27 $A_G^{0,5}$ =4,819 (for friction piles)

ρ=1 L=15,5 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,15	9835,530	843,045	2,615	0,193	3,176	0,232	27,02
			2,926	0,183	3,180	0,253	29,38
υ_s=0,3	9837,88	843,247	2,421	0,201	3,173	0,237	24,37
			2,788	0,187	3,179	0,263	26,98

Method 2

L/d_e=15,5/4,819=3,216 → I_δ=0,25 (Fig. 2.10)

	υ_s=0,15	υ_s=0,3
δ (mm)	28,99	25,65

K ≈ 850 (pile stiffness factor) s/d ≈ 1,8 L/d ≈ 15,5 B=3,794 m

d_e/B ≈ 0,93 assumed, then d_e ≈ 3,529 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,15	9835,530	843,045	2,926	0,250	4,302	0,201	31,91
			3,164	0,240	4,309	0,214	33,90
υ_s=0,3	9837,88	843,247	2,732	0,258	4,296	0,208	29,13
			3,014	0,246	4,305	0,223	31,36

Method 2

L/d_e=15,5/3,529=4,392 → I_δ=0,215 (Fig. 2.10)

	υ_s=0,15	υ_s =0,3
δ (mm)	34,05	30,12

$\delta_{measured}$=13 mm

c) Equivalent Raft Method

L=9,76 B=7,96 L/B=1,22

H=15,92 D=10,3 D/B=1,293 H/B=2

μ_0=0,91 μ_1=0,55

q_n=15000/(BL) = 193,076 KPa

$\delta_{i\ ave} = q_n\ \mu_0\ \mu_1 B/E_u$= 193,076 . 0,91 . 0,55 . 7,96 / 35 = 21,97 mm

$D/(LB)^{0,5}$=1,16→ μ_d=0,7

Tough Clay→ μ_g=0,7

$z/B=1$ $\sigma_z/q=0,3$ $\sigma_z=57,92$ KPa

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0,0352$

$\delta_c=m_\upsilon\, \sigma_z\, H\, \mu_d\, \mu_g$

$=0,0352\ .\ 57,92\ .\ 15,92\ .\ 0,7\ .\ 0,7 \approx 15,94$ mm

$\delta_{T\,ave}=\delta_i+\delta_c$ = 37,92 mm

$\delta_{measured}=13$ mm

Table A.3: Measured and computed settlements for Frame Type Building 2 (mm)

	Settlement (mm)							
	Set. Ratio	**Equivalent Pier**				**Equi. Raft**		**Mea.**
		d_{e1}		d_{e2}		H=5,6 m	H=15,92 m	
		Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,15	29,61	27,02 29,38	28,99	31,91 33,90	34,05	22,28	37,92	13,00
υs=0,3	26,87	24,37 26,99	25,65	29,13 31,36	30,12	18,64	33,04	

Frame 2 (Though Clay)

Rs	29,61	Mea.	13
Pier	27,02 28,99		
Raft	37,92		

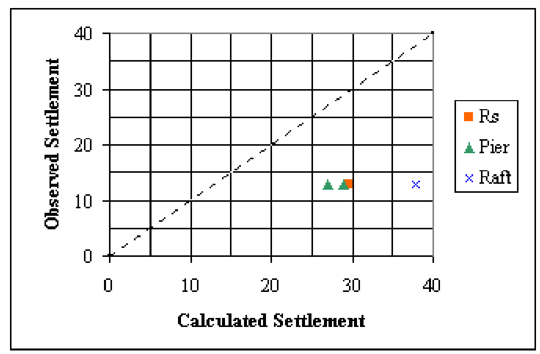

Figure A.5: Measured and computed settlements for Frame Type Building 2 (mm)

4. Frame-Type Building 3 (n=9)

The total structural load of 8,1 MN is supported on a group of 9 driven piles 35*35 cm in section. Penetration depth is 15,5 m. The group has a plan area of 3*3 m. At the site of the building tough plastic clay, E_u=35 Mpa, is located. Different formulations are used to obtain settlement value. Settlement predictions and methods are given below:

USSR standarts	Poulos	Vesic	Skempton	Bartolomey:
40 mm	32 mm	9 mm	14 mm	6 mm

(Bartolomey, A.A., et al.., 1981)

a) Settlement Ratio Method

n= 9 d= 0,394 m r_0= 0,197 m

L= 15,5 m s= 1,182 m

E_u= 35 (MN/m^2) Tough Plastic Clays E_p= 25000 MN/m^2

P=8,1 MN υ_s= 0,15 υ_s =0,3

$\lambda=E_p/G_l$=25000/11,7≈ 2136,752

$\rho=G_{l/2}/G_l$=1→1,06

logλ=3,329→1,05

s/d=3→1

L/d=39,34→0,552

υ_s=0,15→1,035

υ_s =0,3→1

$\eta_w = n^{-e}$ $R_s = n^e$ $\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 1$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single} = 8100/9 = 900$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,15	0,635	0,247	4,043	5,119	1,064	58,213	73,169
υ_s=0,3	0,614	0,259	3,857	4,925	1,084	57,662	75,569

	P_t/w_t	$K = n\eta_w k$	$\delta = P/K$(mm)	P_{single}/k	$\delta = \delta_s R_s$
υ_s=0,15	168,649	375,356	21,58	5,33	21,57
υ_s=0,3	174,17	406,419	19,93	5,16	19,93

$\delta_{measured}$= 5 mm

b) Equivalent Pier Method

$B = A_G^{\ 0.5} = 3$

$A_P = \Pi d^2 n/4 = 1{,}0973\ m^2$

E_p=25000 Mpa E_s'=26,833 MPa E_u=35 MPa

$d_e = 1{,}27\ A_G^{0,5} = 3{,}81$ (for friction piles)

$\rho = 1$ L=15,5 m

$E_e = E_p A_p/A_G + E_s(1-A_p/A_G)$

$\zeta_1 = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

ζ_2=ln/{5+[0,25+(2,5 ρ (1-υ)-0,25)ξ] L/r_0} (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
$υ_s$=0,15	3071,61	262,531	2,850	0,420	3,844	0,221	17,56
			3,104	0,403	3,861	0,236	18,71
$υ_s$=0,3	3074,69	262,793	2,655	0,435	3,829	0,229	16,10
			2,957	0,412	3,852	0,248	17,38

Method 2

L/d_e=15,5/3,81=4,068→ I_δ=0,222 (Fig. 2.10)

	$υ_s$=0,15	$υ_s$=0,3
δ (mm)	17,58	15,56

K ≈ 700 (pile stiffness factor) s/d ≈ 3 L/d ≈ 39,34 B=3 m

d_e/B ≈ 0,85 assumed, then d_e ≈ 2,55 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
$υ_s$=0,15	3071,61	262,531	3,251	0,588	5,462	0,189	22,41
			3,428	0,573	5,490	0,196	23,30
$υ_s$=0,3	3074,69	262,793	3,057	0,606	5,428	0,199	20,89
			3,268	0,586	5,465	0,209	21,91

Method 2

L/d_e=15,5/2,55=6,078 → I_δ=0,21 (Fig. 2.10)

	υ_s=0,15	υ_s=0,3
δ (mm)	24,86	21,99

$\delta_{measured}$=5 mm

c) Equivalent Raft Method

L=7,96 B=7,96 L/B=1

H=15,92 D=10,3 D/B=1,29 H/B=2

μ_0=0,91 μ_1=0,53

q_n=8100/(BL) = 127,83 KPa

$\delta_{i\ ave}=q_n\ \mu_0\ \mu_1 B/E_u$= 127,83 . 0,91 . 0,53 . 7,96 / 35 = 14,02 mm

$D/(LB)^{0,5}$=1,293→ μ_d=0,685

Tough Clay→ μ_g=0,7

z/B=1 σ_z/q=0,272 σ_z=34,771 KPa

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0,0353$

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

=0,0353 . 34,771 . 15,92 . 0,685 . 0,7 ≈ 9,37 mm

$\delta_{T\ ave}=\delta_i+\delta_c$ = 23,39 mm

$\delta_{measured}$=5 mm

Table A.4: Measured and computed settlements for Frame Type Building 3 (mm)

	Settlement (mm)							
	Set. Ratio	Equivalent Pier				Equi. Raft		Mea.
		d_{e1}		d_{e2}		H=5,588 m	H=7,96 m	
		Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,15	21,58	17,56	17,58	22,40	24,86	14,41	23,39	5,00
		18,71		23,30				
υs=0,3	19,93	16,10	15,56	20,89	21,99	12,11	20,52	
		17,38		21,91				

Frame 3 (Though Clay)

Rs 21,58 Mea. 5

Pier 17,56
17,58

Raft 23,39

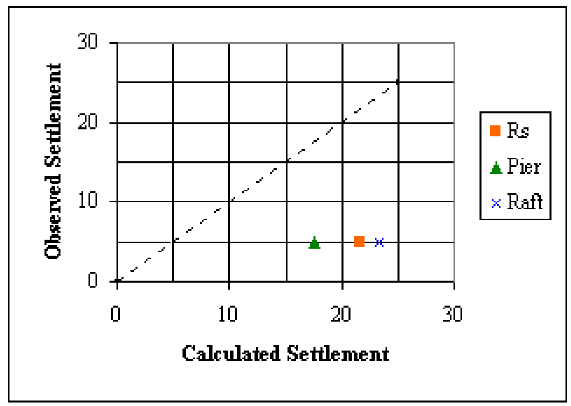

Figure A.6: Measured and computed settlements for Frame Type Building 3 (mm)

5. 9-Pile Group (n=9)

The site consists of various layers of stiff to very stiff clay, and geotechnical data is available from standart penetration tests, cone penetration tests, pressuremeter tests, unconsolidated undrained triaxial tests, laboratory consolidation tests, and seismic cross-hole tests. The piles were about 13 m long, 0.273 m diameter steel tubes with a 9.3 mm wall thickness. The pile spacing in the group was three times the pile diameters, or 0,819 m. The programs DEFPIG (non-linear), PIGLET (simplified continiuum analysis) and GAPFIX (non-linear) were employed and approximately 1,43, 1,43, 1,21 mm settlement predictions were obtianed relatively. (H.G. Poulos ,(1989), M.Polo and M. Clemente, (1998))

a) Settlement Ratio Method

n= 9 piles d= 0,273 m r_0 =0,1365 m

L= 13 m s/d= 3

P= 1.8 MN

υ_s= 0,15 υ_s= 0,33

1. E=40+5,38z E_u=110 MPa Gl=36,67 MPa

2. From Pressuremeter test E_u=147 MPa Gl=49 MPa

3. E_u=190 MPa Gl=63,3 Mpa

E_p= 21000, 25000 MN/m^2

$\lambda=E_p/G_l$

$\rho=G_{l/2}/G_l$

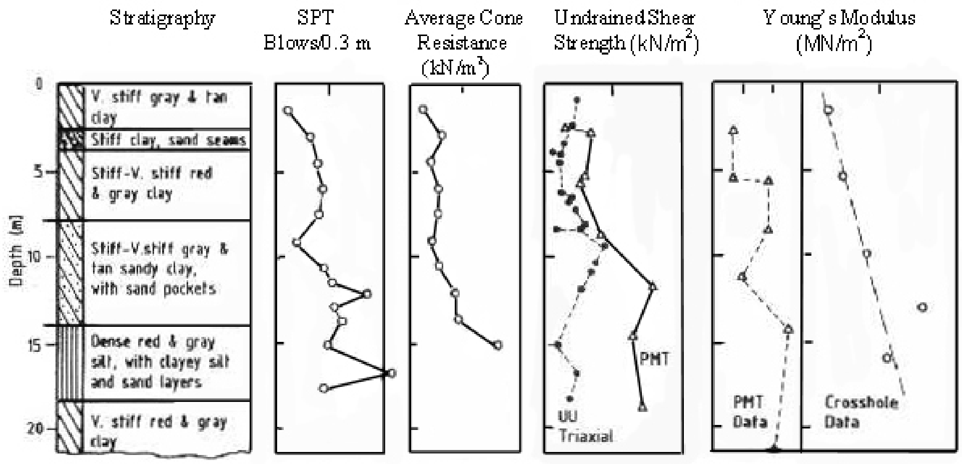

Figure A.7: Summary of geotechnical data at test site(Poulos, 1989, .Polo and Clemente, 1998)

s/d=3⟶1

L/d=13/0,273=47,62⟶0,549

υ_s=0,15⟶1,04

υ_s==0,33⟶1

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al.1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

P_{single}=1800/9=200 KN

	E'=145,6	E_u=168,4	E'=112,7	E_u=130,3	E'=84,3	E_u=97,5
e	0,467	0,449	0,515	0,495	0,520	0,500
η_w	0,358	0,372	0,322	0,336	0,318	0,332
R_s	2,791	2,683	3,100	2,968	3,139	3,004

ζ	4,923	4,685	4,974	4,736	4,926	4,688
μL	3,333	3,417	2,673	2,740	2,325	2,383
$\tanh\mu L$ $L/(\mu L\, r_0)$	28,497	27,810	35,282	34,469	40,186	39,286
$P_t/(w_t G_l r_0)$	26,047	26,992	33,113	34,240	36,404	37,703
P_t/w_t	225,17	233,33	221,47	229,01	182,36	188,87
$K=n\eta_w k$	725,87	782,49	642,79	694,24	522,74	565,76
$\delta=P/K$(mm)	2,48	2,30	2,80	2,59	3,44	3,18
P_{single}/k	0,88	0,85	0,90	0,87	1,09	1,05
$\delta=\delta_s R_s$	2,48	2,30	2,80	2,59	3,44	3,18

$\delta_{measured}$=0,9 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =2,73 m

$A_P=\Pi d^2 n/4$=0,5268 m^2

E_p=25000 MPa

E=40+5,38z E_s'=84,33 MPa E_u=110 MPa

Fom Pressuremeter test E_s'=112,7 MPa E_u=147 MPa

E_u=190 MPa

$d_e=1,27\ A_G^{0,5}$ =3,46 m (for friction piles)

ρ=0,71-0,68-0,67   L=13 m

$E_e=E_pA_p/A_G + E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et.al 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

	E’=145,6	E_u=168,4	E’=112,7	E_u=130,3	E’=84,3	E_u=97,5
E_e	1652,71	1673,86	1911,19	1927,55	1884,87	1897,12
λ	26,09	26,43	39,00	39,33	51,40	51,79
$\zeta_{(1-2)}$	2,39	2,15	2,44	2,11	2,39	2,06
	2,65	2,48	2,69	2,45	2,66	2,42
μL	1,35	1,42	1,09	1,17	0,96	1,03
	1,28	1,32	1,04	1,09	0,91	0,95
$\tanh\mu L$	2,44	2,37	2,76	2,66	2,93	2,84
$L/(\mu L\ d_e)$	2,52	2,48	2,82	2,77	2,99	2,94
I_δ	0,43	0,46	0,37	0,38	0,35	0,35
	0,46	0,49	0,39	0,41	0,37	0,38
δ	1,58	1,44	1,74	1,53	2,20	1,91
	1,66	1,55	1,84	1,65	2,34	2,07

Method 2

$L/d_e=13/3{,}46=3{,}74 \longrightarrow I_\delta=0{,}22$ (Fig. 2.10)

	E’=145,6	E_u=168,4	E’=112,7	E_u=130,3	E’=84,3	E_u=97,5
δ	0,79	0,68	1,02	0,88	1,37	1,18

$K \approx 200$ (pile stiffness factor) $s/d \approx 3$ $L/d \approx 47{,}6$ B=2,73 m

$d_e/B \approx 0,75$ assumed, then $d_e \approx 2,05$ m (Fig. 2.9)

	E'=145,6	E_u=168,4	E'=112,7	E_u=130,3	E'=84,3	E_u=97,5
E_e	1652,71	1673,86	1911,19	1927,55	1884,87	1897,12
λ	26,09	26,43	39,00	39,33	51,40	51,79
$\zeta_{(1-2)}$	2,90	2,66	2,95	2,62	2,91	2,58
	3,14	2,96	3,18	2,93	3,15	2,90
μL	2,05	2,13	1,67	1,76	1,47	1,55
	1,97	2,02	1,60	1,67	1,41	1,46
tanhμL	2,98	2,88	3,53	3,38	3,88	3,73
$L/(\mu L\ d_e)$	3,08	3,02	3,63	3,53	3,99	3,89
I_δ	0,45	0,49	0,37	0,39	0,35	0,36
	0,47	0,52	0,39	0,42	0,36	0,38
δ	2,76	2,58	2,95	2,68	3,63	3,27
	2,87	2,71	3,06	2,82	3,79	3,47

Method 2

$L/d_e=13/2,05=6,34 \longrightarrow I_\delta=0,2$ (Fig. 2.10)

	E'=145,6	E_u=168,4	E'=112,7	E_u=130,3	E'=84,3	E_u=97,5
δ	1,20	1,04	1,55	1,34	2,08	1,80

$\delta_{measured}$=0,9 mm

c) Equivalent Raft Method

L=7,07 m B=7,07 m H=14,06 m

L/B=1 D/B=1,23 H/B=2

P=1800 kN q_n=1800/(BL) = 36,01 Kpa υ_s= 0,15

E=40+5,38z $E_s'_{mid}$=77,56 MPa $E_{u\,mid}$ =101,17 MPa

Fom Pressuremeter test $E_s'_{mid}$ =104 MPa $E_{u\,mid}$ =136 MPa

$E_s'_{mid}$ =133,64 MPa $E_{u\,mid}$ =174,32 MPa

μ_1 = 0,536 μ_0 = 0,91

$\delta_{i\,ave}=\mu_1\mu_0 q_n B/E_u$ =0,91 . 0,536 . 36,42 . 7,03 / 101,17 =1,23 mm

=0,91 . 0,536 . 36,42 . 7,03 / 136 =0,91 mm

=0,91 . 0,536 . 36,42 . 7,03 / 147,32 =0,71 mm

$D/(LB)^{0,5}=1,23\rightarrow\mu_d=0,68$

stiff clay$\rightarrow\mu_g$=0,7

z/B=1 σ_z/q=0,27 σ_z=9,83 KPa

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E'(1-\upsilon)] \approx 0,0122 – 0,0091 – 0,0070$

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

=0,0122 . 9,83 . 14,6 . 0,68 . 0,7 ≈0,803 mm

=0,0091 . 9,83 . 14,6 . 0,68 . 0,7 ≈ 0,599 mm

=0,0070 . 9,83 . 14,6 . 0,68 . 0,7 ≈ 0,466 mm

$\delta_T=\delta_{i\,ave}+\delta_c$ = 2,03 – 1,51 – 1,18 mm

$\delta_{measured}$=0,9 mm

Table A.5: Measured and computed settlements for 9-Pile Group (mm)

	Settlement (mm)														
	Settlement Ratio			Equivalent Pier											
				de1						de2					
				Met1			Met2			Met1			Met2		
	E_p=25000		E_p=21000	E_p=25000		E_p=21000				E_p=25000		E_p=21000			
E_s=	147	110	190	147	110	190	147	110	190	147	110	190	147	110	190
υs=0,15	2,80	3,44	2,48	1,74	2,20	1,58	1,02	1,37	0,79	2,95	3,63	2,76	1,55	2,08	1,20
				1,84	2,34	1,66				3,06	3,79	2,87			
υs=0,33	2,59	3,18	2,30	1,53	1,91	1,44	0,88	1,18	0,68	2,68	3,27	2,58	1,34	1,80	1,04
				1,65	2,07	1,55				2,82	3,47	2,71			

	Equivalent Raft						Mea.
	Eu=110		Eu=147		Eu=190		
H (m)	5,4	14,1	5,4	14,1	5,4	14,1	
	Ave.	Ave.	Ave.	Ave.	Ave.	Ave.	
υs=0,15	1,28	2,03	0,95	1,51	0,74	1,09	0,90
υs=0,33	1,02	1,72	0,76	1,28	0,59	1,00	

9-Pile Group

Rs	2,59	Mea.	0,9
	2,30		
Pier	1,53		
	0,88		
	1,44		
	0,68		
Raft	1,00		
	1,28		

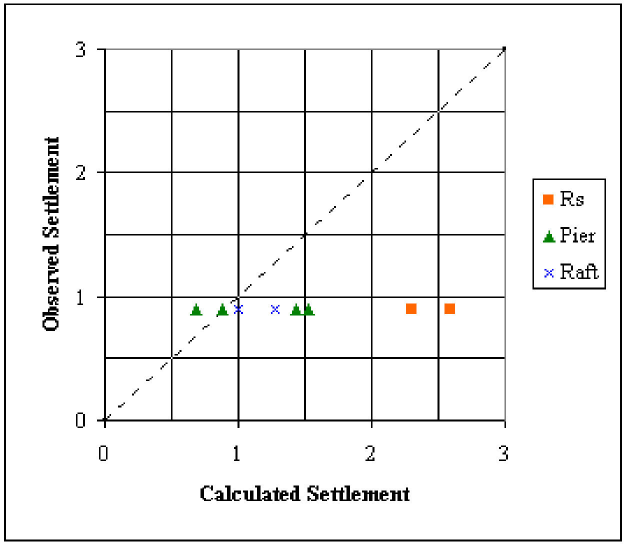

Figure A.8: Measured and computed settlements for 9-Pile Group (mm)

6. Frame-Type Building 7 (n=16)

The total structural load of 16 MN is supported on a group of 16 piles 40*40 cm in section. Distance between piles varies from 1,2 to1,6 and penetration depth is 20 m. The group has a plan area of 4,5*5 m. At the site of the building shingle, E_u=80 Mpa is located. Different formulations are used to obtain settlement value. Settlement predictions and methods are given below:

USSR standarts	Poulos	Vesic	Skempton	Bartolomey:
40 mm	32 mm	9 mm	14 mm	6 mm

(Bartolomey, A.A., et al.., 1981)

a) Settlement Ratio Method

n= 16 d= 0,4512 m r_0= 0,2256 m

L= 20 m s= 1,4 m

E_u=80 MN/m^2 Shingle E_p= 25000 MN/m^2

P=16 MN υ_s= 0,35 υ_s =0,4

$\lambda=E_p/G_l$=25000/26,7 ≈ 936,329

$\rho=G_{l/2}/G_l$=1→1,06

logλ=2,971→0,99

s/d=3,1→1

L/d=44,326→0,55

υ_s=0,35→0,98

υ_s=0,4→0,97

$\eta_w = n^{-e}$ $R_s = n^e$ $\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 1$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single} = 16000/16 = 1000$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s = 0{,}35$	0,565	0,208	4,798	4,970	1,837	45,854	58,508
$\upsilon_s = 0{,}4$	0,559	0,211	4,722	4,890	1,852	45,551	59,093

	P_t/w_t	$K = n\eta_w k$	$\delta = P/K$(mm)	P_{single}/k	$\delta = \delta_s R_s$
$\upsilon_s = 0{,}35$	352,426	1175,185	13,61	2,83	13,61
$\upsilon_s = 0{,}4$	355,949	1206,079	13,26	2,81	13,26

$\delta_{measured} = 4$ mm

b) Equivalent Pier Method

$B = A_G^{\ 0.5} = 4{,}763$

$A_P = \Pi d^2 n/4 = 2{,}558 m^2$

$E_p = 25000$ MPa

$E_s' = 72$ MPa $E_u = 80$ MPa

$d_e = 1{,}13\ A_G^{0,5} = 5{,}36$ (for end-bearing piles)

$\rho = 1$ $L = 20$ m

$E_e = E_p A_p/A_G + E_s(1 - A_p/A_G)$

ζ_1=ln(2,5 ρ (1-υ) L/r_0) (W. Fleming, et al, 1992)

ζ_2=ln/{5+[0,25+(2,5 ρ (1-υ)-0,25)ξ] L/r_0} (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,35	2906,347	108,852	2,495	0,640	3,293	0,265	11,01
			2,840	0,600	3,339	0,289	11,98
υ_s=0,4	2908,71	108,94	2,415	0,650	3,281	0,266	10,63
			2,784	0,605	3,333	0,291	11,65

Method 2

L/d_e=20/5,36=3,731 → I_δ=0,23 (Fig. 2.10)

	υ_s=0,35	υ_s=0,4
δ (mm)	9,53	9,198

K ≈ 300 (pile stiffness factor) s/d ≈ 3,1 L/d ≈ 44,32 B=4,74 m

d_e/B ≈ 0,75 assumed, then d_e ≈ 3,55 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,35	2906,347	108,852	2,905	0,894	4,485	0,245	15,33
			3,147	0,859	4,553	0,258	16,13
υ_s=0,4	2908,71	108,94	2,825	0,906	4,462	0,247	14,93
			3,084	0,867	4,537	0,262	15,78

Method 2

L/d_e=20/3,557=5,621→ I_δ=0,21   (Fig. 2.10)

	υ_s=0,35	υ_s=0,4
δ (mm)	13,11	12,65

$\delta_{measured}$=4 mm

c) Equivalent Raft Method

L=5   B=4,5   L/B=1,11

H=9   D=20   D/B=4,444   H/B=2

μ_0=0,88 μ_1=0,53

q_n=16000/(BL) = 711,11 KPa

$\delta_{i\ ave}=q_n\ \mu_0\ \mu_1 B/E_u$= 71,11 0,88 0,53 4,5 / 80 = 18,656

$\delta_{measured}$=4 mm

Table A.6: Measured and computed settlements for Frame Type Building 7 (mm)

	Settlement (mm)						
	Set. Ratio	Equivalent Pier				Eq. Raft	Mea.
		d_{e1}		d_{e2}		Ave.	
		Met1	Met2	Met1	Met2		
υs=0,35	13,61	11,01	9,53	15,33	13,11	18,65	4,00
		11,98		16,13			
υs=0,4	13,26	10,63	9,19	14,93	12,65		
		11,65		15,78			

Frame 7 (Shingle)

Rs 13,61 Mea. 4

Pier 11,01
9,53

Raft 18,65

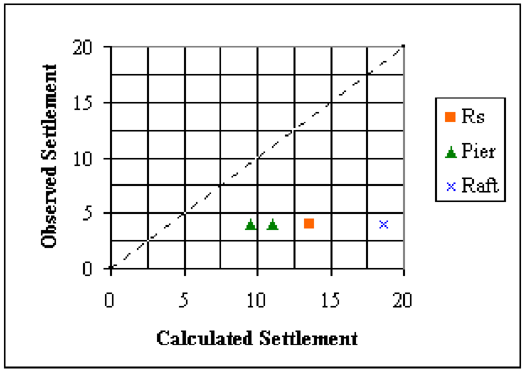

Figure A.9: Measured and computed settlements for Frame Type Building 7 (mm)

7. Five-storey Building in Urawa-Japan (n=20)

A piled raft foundation has been adopted in Japan for a five-storey building with plan area measuring 24 m by 23 m. The foundation consisted of a raft (0.3 m thick) with 20 piles, one under each column. The piles were bored concrete piles, either 0.8 or 0.7 m in diameter, with a central steel H-pile inserted. The pile diameter and steel pile size depended on the column load, which ranged between 1.02 MN and 3.95 MN. The GASGROUP (using superposition principle, with interaction factors) analysis yielded a settlement ratio, R_s, of 2,516, and predicted settlement of the pile group is calculated as 12,6 mm. The average settlement computed by program GARP (plate on springs approach) is 13,5 mm. (Poulos, H.G., 2001, Yamashita, K. et al, 1993, Randolph, M. and Guo, W., 1999)

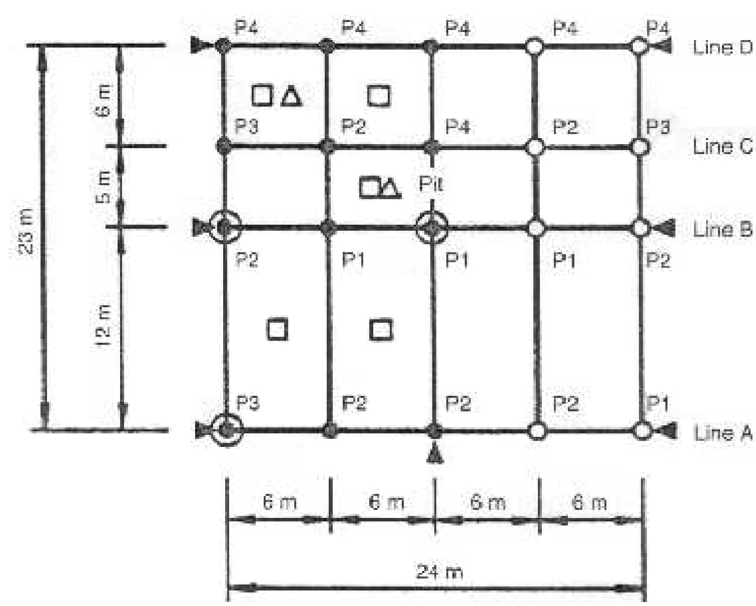

Figure A.10: Five-storey building in Japan, foundation plan (Yamashita et al, 1993)

a) Settlement Ratio Method

n= 20 piles   d= 0,75 m r_0 =0,375 m

L= 15,8 m   s ≈ 5,25 m E_p= 25000 MN/m^2

$E_{u(ave)}$=42,5 Mpa (loose to medium sand)

$E_{u(ave)}$=60 Mpa (stiff cohesive soil)

P= 23 MN

υ_s= 0,2 υ_s= 0,3

$\lambda=E_p/G_l=25000/20 \approx 1250$

$\rho=G_{l/2}/G_l=0{,}71 \rightarrow 0{,}98$

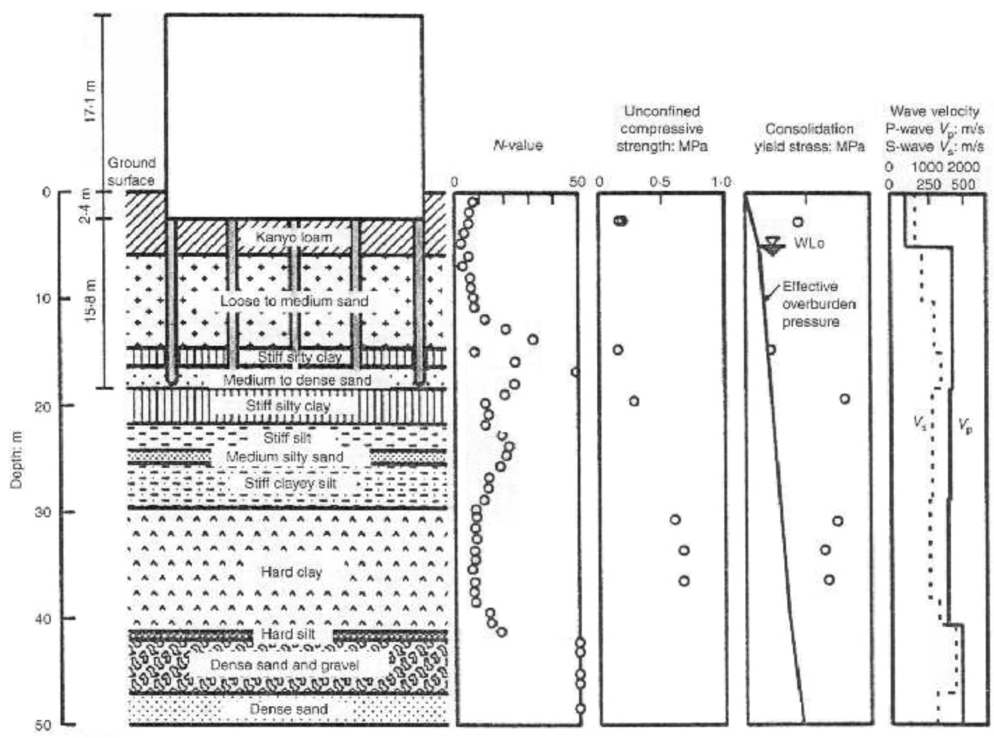

Figure A.11: Elevation of building and summary of soil investigation (Yamashita et al, 1993)

$\log\lambda=3{,}09 \rightarrow 1{,}02$

$s/d=7 \rightarrow 0{,}78$

$L/d=21{,}06 \rightarrow 0{,}538$

$\upsilon_s=0{,}2 \rightarrow 1{,}03$

$\upsilon_s=0{,}3 \rightarrow 1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$ $\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

$P_{single}=23000/20=1150$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0{,}2$	0,432	0,274	3,648	4,091	0,833	34,497	40,820
$\upsilon_s=0{,}3$	0,538	0,284	3,513	3,958	0,847	34,296	42,261

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0{,}2$	306,15	1678,21	13,70	3,75	13,70
$\upsilon_s=0{,}3$	316,96	1804,21	12,74	3,62	12,74

$\delta_{measured}=12{,}65$ mm

Equivalent Pier Method

$B=A_G^{\ 0.5}=24{,}24$ m

$A_P=\Pi d^2 n/4=8{,}836\ m^2$

E_p=25000 MPa

E_s'=48 MPa E_u=60 MPa

d_e=1,27 $A_G^{0,5}$ =30,79 m (for friction piles)

ρ=0,71   L=15,8 m

$E_e=E_pA_p/A_G + E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,2	423,067	21,153	0,376	0,514	0,472	0,317	4,94
			1,864	0,231	0,504	0,696	10,84
υ_s=0,3	427,007	21,350	0,243	0,637	0,453	0,250	3,59
			1,836	0,231	0,504	0,691	9,93

Method 2

L/d_e=15,8/30,79 =0,513 ⟶ I_δ=0,5 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	7,78	7,18

K ≈ 420 (pile stiffness factor)   s/d ≈ 7   L/d ≈ 21,06   B=24,24

d_e/B ≈ 0,78 assumed, then d_e ≈ 18,91 m   (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	423,067	21,153	0,864	0,552	0,759	0,416	10,55
			1,997	0,363	0,800	0,627	15,89
υ_s=0,3	427,007	21,350	0,730	0,598	0,748	0,394	9,232
			1,956	0,365	0,800	0,631	14,76

Method 2

L/d_e=15,8/18,91=0.835 $\rightarrow$ I_δ=0,45 (Fig.2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	11,40	10,52

$\delta_{measured}$=12,65 mm

Equivalent Raft Method

P=23000KN

q_n=P/(BL)

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E$

q_n	L	B	L/B	D	H	D/B	H/B	μ_0	μ_1	E_u	δ_i
30,45	30	29	1,034	12,93	5,3	0,445	0,182	0,93	0,045	42,5	0,869
21,03	36	35	1,028	18,2	11,8	0,52	0,337	0,93	0,1	60	1,141
											Σ2,01

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

for sand $q_c/N=5$ $q_c \approx 5MPa$ $M_0=4q_c$

$\delta_c=m_\upsilon\ \sigma_z\ H\ m_d\ m_g$

$D/(LB)^{0,5}=0{,}438 \rightarrow \mu_d=0{,}87$

Silt and sand $\rightarrow$ $\mu_g=1$

z/B	σ_z/q	σ_z	$E_{mid\text{-}dr}$	ν	H	m_υ	μ_g	μ_d	δ_c
0,091	0,88	23,26	36,8	0,3	5,3	0,05	1	0,87	5,36
0,386	0,6	15,86	48	0,2	11,8	0,0187	1	0,87	3,05
									Σ8,41

$\delta_T=\delta_i+\delta_c$ = 2,01+8,41 = **10,42 mm**

$\delta_{measured}$=12,5 mm

Table A.7: Measured and computed settlements for Five-Storey Building in Urawa Japan (mm)

	Settlement (mm)						
	Set. Ratio	Equivalent Pier				Eq. Raft	Mea.
		d_{e1}		d_{e2}			
		Met1	Met2	Met1	Met2	Ave.	
υs=0,2	13,70	4,94	7,78	10,55	11,40	10,43	12,65
		10,84		15,89			
υs=0,3	12,74	3,60	7,18	9,23	10,52		
		9,93		14,76			

Urawa - Japan

Rs 13,7 Mea. 12,65

Pier 10,84
7,78
10,55

Raft 10,43

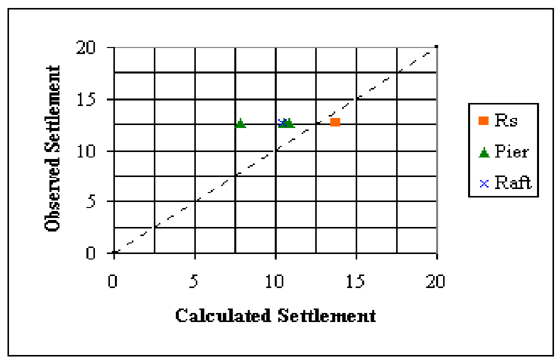

Figure A.12: Measured and computed settlements for Five-Storey Building in Urawa Japan (mm)

8. Eurotheum Building (n=25)

The basement and raft of the new high–rise Eurotheum building are loaded eccentrically by a 110 m high office tower, 28.1 m square in plan, which is surrounded by a six-storey apartment building. The foundation level is 11,5-13 m below street level and 6 m below the groundwater level. Frankfurt limestone exixts 55 m below the ground surface. The building is founded on a piled raft with a thickness of 1.0-2.5 m and a plan area of 2000 m^2, together with 25 bored piles which are concentrated beneath the eccentrically placed core of the skyscraper. The length of the 1.5 m diameter bored piles depends on their location, varying from 25 m for the corner piles, to 27.5 m for the edge piles and 30 m for the inner piles. (Katzenbach, R., Arslan,U., and Moormann, C., 2000)

a) Settlement Ratio Method

n= 25   d= 1,5 m r_0= 0,75 m

L= 27,5 m   s= 4 m

G= 20+ 1,0z MN/m^2 Frankfurt clay E_p= 35000 MN/m^2

E_u=20000 MN/m^2 Frankfurt Limestone

υ_s= 0,1 υ_s =0,3

P=570 MN

$\lambda=E_p/G_l=35000/52,5 \approx 666,667$

$\rho=G_{l/2}/G_l=0,742 \rightarrow 1$

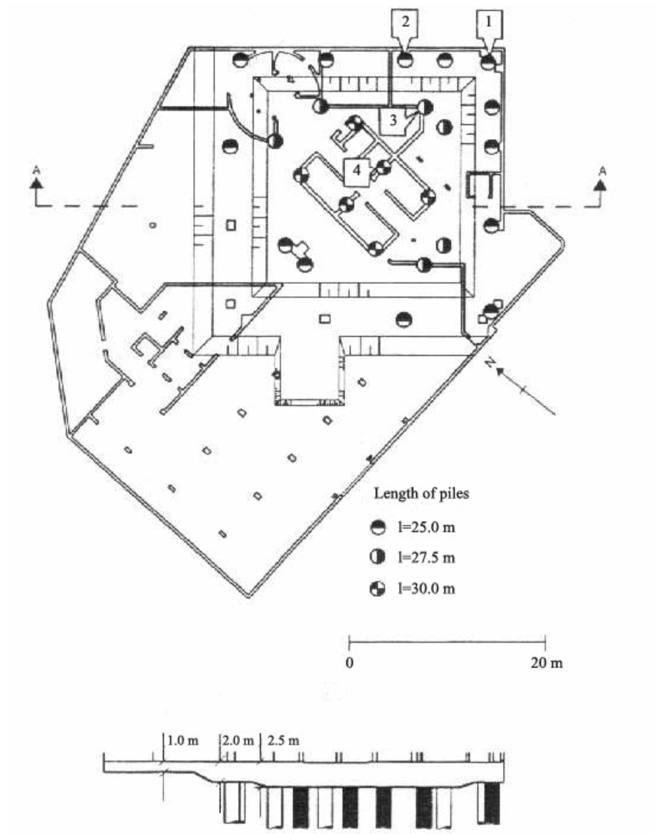

Figure A.13: Piled raft foundation for Eurotheum building, plan and section A-A (Katzenbach et al, 2000)

$\log\lambda=2{,}823\rightarrow0{,}96$

$s/d=4\rightarrow0{,}93$

$L/d=18{,}33\rightarrow0{,}533$

$\upsilon_s=0{,}1\rightarrow1{,}05$

$\upsilon_s=0{,}3\rightarrow1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=0{,}0078$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=570000/25=22800 KN

	e	η_w	R_s	ζ	μL	tanhμL L/(μL r_0)	$P_t/(w_tG_lr_0)$
υ_s=0,1	0,499	0,200	4,994	2,259	1,336	23,895	82,501
υ_s=0,3	0,475	0,216	4,626	2,248	1,339	23,858	83,657

	P_t/w_t	$K=n\eta_wk$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_sR_s$
υ_s=0,1	3248,507	16260,56	35,05	7,018	35,05
υ_s=0,3	3294,021	17800,79	32,02	6,921	32,02

$\delta_{measured}$=32 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =28,1 m

$A_P=\Pi d^2n/4$=44,178 m^2

E_p=35000 MPa E_s'=1155 MPa E_u=157,5 MPa

$d_e=1{,}13\ A_G^{0,5}$=31,753 m (for end-bearing piles)

ρ=0,742 L=27,5 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	2067,29	39,376	-0,793	———			
			1,696	0,299	0,841	0,067	10,42
υ_s=0,3	2087,11	39,754	-0,804	———			
			1,695	0,298	0,841	0,076	10,09

Method 2

L/d_e=27,5/31,753=0,866→ I_δ=0,025 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	3,88	3,28

K ≈ 200 (pile stiffness factor) s/d ≈ 4 L/d ≈ 18,33 B=28,1 m

d_e/B ≈ 0,78 assumed, then d_e ≈ 21,918 m (Fig.2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	2067,29	39,376	-0,422	———			
			1,732	0,429	1,182	0,090	20,46
υ_s=0,3	2087,11	39,754	-0,433	———			
			1,731	0,427	1,183	0,104	19,96

$\delta_{measured}$=32 mm

Method 2

L/d_e=27,5/21,918=1,25 → I_δ=0,03 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	6,75	5,71

$\delta_{measured}$=32 mm

c) Equivalent Raft Method

L=28,1 B=28,1 L/B=1

H=14,5 D=40,5 H/B=0,516 D/B=1,441

E_{uave}=176,25 E_s'=131,45

μ_0→ 0,905 μ_1→0,168

$\delta_{iave}=q_n B \mu_0 \mu_1 / E_u$ = 17,20 mm

$D/(LB)^{0,5}$=1,441→ μ_d=0,68

Frankfurt clay→ μ_g=0,7

z/B=0,258 σ_z/q=0,727 σ_z=524,803 Kpa

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0,00743$

$\delta_c = m_\upsilon \sigma_z H \mu_d \mu_g$

=0,00743. 524,803. 14,5 . 0,68 . 0,7 ≈ 26,94 mm

$\delta_{Taverage}$ = 17,20+26,94=44,15 mm

$\delta_{measured}$=32 mm

Table A.8: Measured and computed settlements for Eurotheum Building (mm)

	Settlement (mm)							
	Set. Ratio	Equivalent Pier				Equivalent Raft		Mea.
		d_{e1}		d_{e2}		H=23,66 m (end-bearing piles)	H=14,5 m (friction piles)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,1	35,05	10,42	3,88	20,47	6,75	46,71	44,15	32,00
υs=0,3	32,02	10,09	3,28	19,96	5,71	36,71	34,52	

Eurotheum

Rs	35,05	Mea.	32
Pier	20,47		
Raft	44,15		

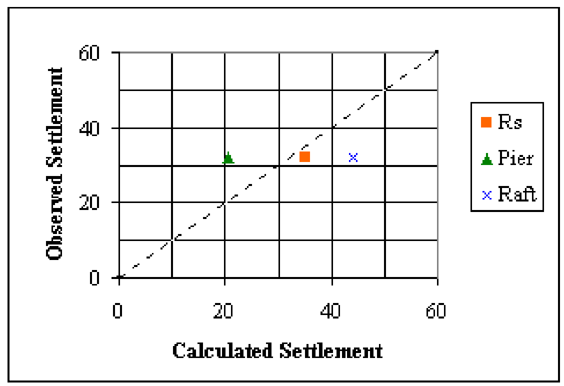

Figure A.14: Measured and computed settlements for Eurotheum Building (mm)

9. Japan Centre (n=25)

The 115.3 m high Taunustor-Japan-Centre office tower is located in the centre of the financial district of Frankfurt am Main. The building comprises four basement floors and eccentrically placed tower with 29 floors above grade having dimensions of 36,6*36,6 m in plan. The total structural load of 1050 MN is supported on a piled raft 15,8 m below the ground surface, which is about 9,5 m below the groundwater table. The raft has a thickness of 3,0 m at the centre, reducing 1,0 m at the edges. The raft is loaded with a remarkable eccentricity in the building load of 7.5. Therefore the positions of the 25 bored piles (diameter 1.3 m, length 22 m) under the raft were optimised during the design to guarantee fairly constant settlements over the entire foundation. At the site of the Japan-Centre building, the boundary between the Frankfurt Clay and the rocky Frankfurt Limestone is located approximately 43 m below the ground surface, which is only about 5 m below the base level of the piles. (Katzenbach, R., Arslan,U., and Moormann, C., 2000)

a) Settlement Ratio Method

n= 25 d= 1,3 m r_0= 0,65 m

L= 22 m s= 4,5 m

G= 20+1,0z (MN/m^2) Frankfurt Clay E_p= 30000 MN/m^2

E_u=20000 MN/m^2 Frankfurt Limestone

P=1050 MN υ_s= 0,1 υ_s =0,3

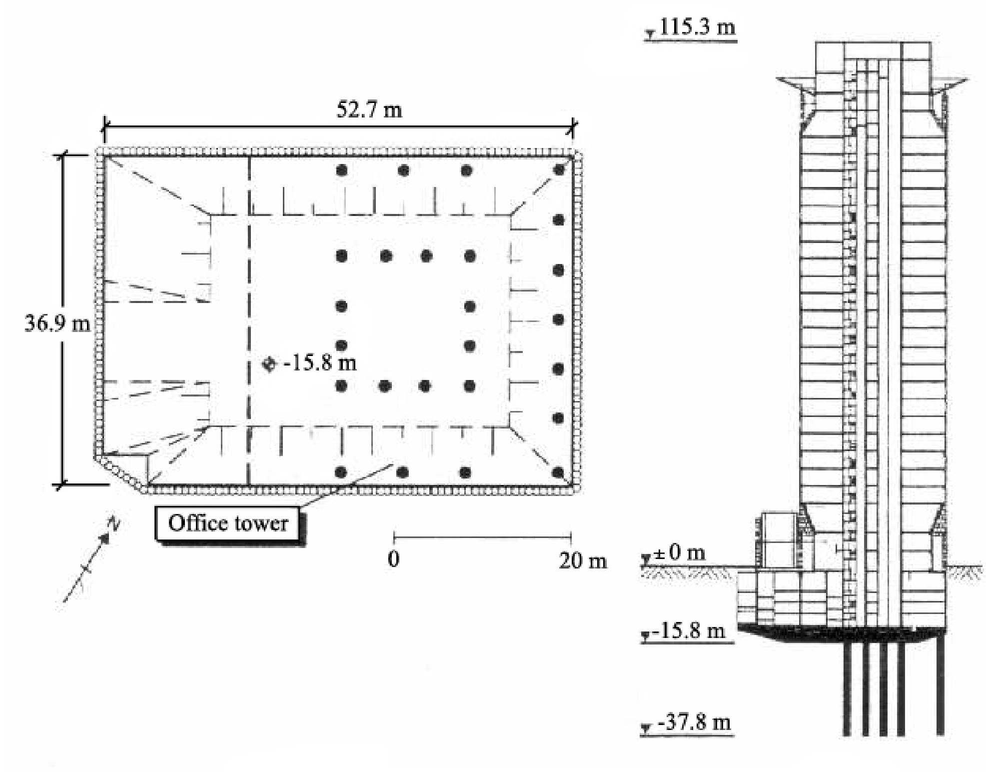

Figure A.15: Japan Centre building, ground plan and sectional elevation (Katzenbach et al, 2000)

$\lambda = E_p/G_l = 30000/47{,}8 \approx 627{,}615$

$\rho = G_{l/2}/G_l = 0{,}769 \rightarrow 1$

$\log\lambda = 2{,}797 \rightarrow 0{,}95$

$s/d = 4{,}5 \rightarrow 0{,}9$

$L/d = 16{,}92 \rightarrow 0{,}528$

$\upsilon_s = 0{,}1 \rightarrow 1{,}05$

$\upsilon_s = 0{,}3 \rightarrow 1$

$\eta_w = n^{-e}$ $R_s = n^e$ $\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 0{,}00717$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=1050000/25=42000 KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,474	0,217	4,598	2,177	1,294	22,489	83,000
υ_s=0,3	0,451	0,233	4,276	2,167	1,298	22,455	84,066

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	2578,838	14019,22	74,89	16,28	74,89
υ_s=0,3	2611,93	15269,22	68,76	16,08	68,76

$\delta_{measured}$=50 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =32,012 m

$A_P=\Pi d^2 n/4$=33,183 m^2

E_p=30000 MPa

E_s'=105,16 MPa E_u=143,4 MPa

$d_e=1,13\ A_G^{0,5}$=36,174 (for end-bearing piles)

ρ=0,769   L=22 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	1073,16	22,450	-1,148				
			1,670	0,280	0,592	0,080	22,24
υ_s=0,3	1091,66	22,838	-1,27				
			1,670	0,278	0,593	0,092	21,50

Method 2

L/d_e=22/36,174 =0,541 → I_δ=0,02 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	5,52	4,67

K ≈ 210 (pile stiffness factor) s/d ≈ 4,5 L/d ≈ 16,9 B=32,012 m

d_e/B ≈ 0,75 assumed, then d_e ≈ 24,009 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	1073,160	22,450	-0,738				
			1,700	0,419	0,866	0,114	47,51
υ_s=0,3	1091,66	22,838	-0,749				
			1,699	0,415	0,866	0,131	46,22

Method 2

L/d_e=22/24,001=0,916 → I_δ=0,025 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	10,39	8,79

$\delta_{measured}$=50 mm

c) Equivalent Raft Method

L=36,6 B=27 L/B=1,355

H=5 D=37,8 D/B=1,4 H/B=0,185

E_{uave}=156,9 E_s'=115,06

μ_0→ 0,92 μ_1→0,04

q_n=1050000/(BL) = 1062,53 KPa

$\delta_{iave}=q_n B \mu_0 \mu_1 / E_u$ = 8,31 mm

$D/(LB)^{0,5}$=1,202→ μ_d=0,695

Frankfurt Clay→ μ_g=0,7

z/B=2,5/27=0,092 σ_z/q=0,9 σ_z=956,28 KPa

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0,00849$

$\delta_c=m_\upsilon \sigma_z H \mu_d \mu_g$=0,00849 . 956,28 . 5 . 0,695 . 0,7 ≈ 19,76 mm

$\delta_{Taverage}$ = 28,08 mm

$\delta_{measured}$=50 mm

Table A.9: Measured and computed settlements for Japan-Centre Building (mm)

	Settlement (mm)										
	Set. Ratio	Equivalent Pier								Eq. Raft	Mea.
		B*L(36,6*36,6)				B*L(28*36,6)				B*L (28*36,6)	
		d_{e1}		d_{e2}		d_{e1}		d_{e2}			
		Met1	Met2	Met1	Met2	Met1	Met2	Met1	Met2	Ave.	
υs=0,1	74,89	21,43	4,82	45,94	9,09	22,24	5,52	47,51	10,39	28,08	50
υs=0,3	68,76	20,66	4,08	44,55	7,69	21,50	4,67	46,22	8,79	21,02	(40 - 60)

Japan Centre

Rs	74,89	Mea.	50
Pier	47,51		
Raft	28,08		

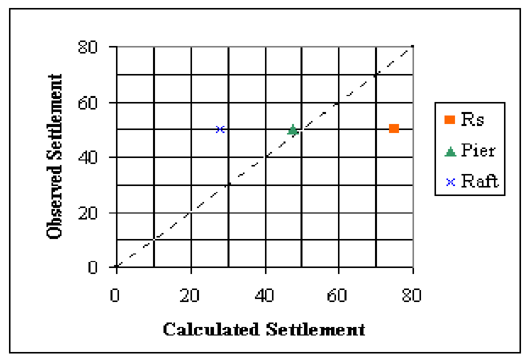

Figure A.16: Measured and computed settlements for Japan-Centre Building (mm)

10. Forum (Pollux and Kastor) Building Complex (n=26,22)

The Forum building complex is located 150 m south-east of the Messeturm tower. There two office towers; the so-called Kastor with height of 94 m, and Pollux with height of 130 m. These towers are located at opposite ends of a 120.5 m wide parking basement with three underground floors. Although the raft loading is extremely eccentric, the raft is designed as a single structure (14000 m^2 plan area) with bored piles having a diameter of 1.3 m and lengths of 20 m and 30 m concentrated under the Kastor building (26 piles) and under the Pollux building (22 piles). The thickness of the raft is 3.0 m beneath the towers and 1.0 m in the area of the parking basement. (Katzenbach, R., Arslan,U., and Moormann, C., 2000)

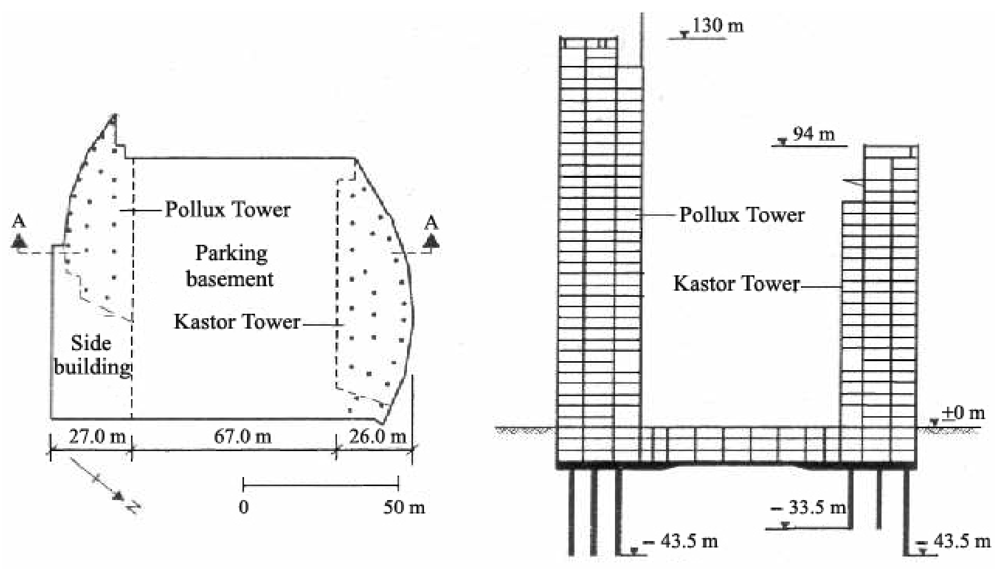

Figure A.17: Forum building complex, ground plan and section A-A (Katzenbach et al, 2000)

Solution for Forum Pollux

a) Settlement Ratio Method

n= 26   d= 1,3 m r_0= 0,65 m

L= 30 m   s= 5 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,1 υ_s =0,3   Frankfurt Clay

P=990 MN

$\lambda=E_p/G_l=30000/55,5 \approx 540,540$

$\rho=G_{l/2}/G_l=0,729 \rightarrow 0,99$

$\log\lambda=2,732 \rightarrow 0,93$

$s/d=5 \rightarrow 0,875$

$L/d=23,077 \rightarrow 0,54$

$\upsilon_s=0,1 \rightarrow 1,05$

$\upsilon_s=0,3 \rightarrow 1$

$\eta_w=n^{-e}$ $R_s=n^e$ $\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

P_{single}=990000/26=38076,923 KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,456	0,225	4,429	4,328	1,349	29,889	33,490
υ_s=0,3	0,435	0,242	4,126	4,077	1,390	29,318	35,215

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	1208,154	7091,875	139,59	31,51	139,59
υ_s=0,3	1270,405	8004,951	123,67	29,97	123,67

$\delta_{measured}$=80 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =34,35 m

$A_P=\Pi d^2 n/4$=34,50 m^2

E_p=30000 MPa E_s'=122,1MPa E_u=166,5 MPa

$d_e=1,27\ A_G^{0,5}$=38,81 (for friction piles)

ρ=0,729   L=30 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al.,1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	995,912	17,944	0,814	0,508	0,633	0,418	77,67
			2,111	0,316	0,665	0,662	123,17
υ_s=0,3	1017,46	18,332	0,563	0,605	0,614	0,371	58,37
			2,048	0,317	0,665	0,677	106,49

Method 2

L/d_e=30/43,62 =0,687 → I_δ=0,474 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	88,09	74,54

K ≈ 200 (pile stiffness factor) s/d ≈ 5 L/d ≈ 23,07 B=34,35 m

d_e/B ≈ 0,75 assumed, then d_e ≈ 25,763 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	995,912	17,944	1,341	0,671	0,016	0,448	141,03
			2,177	0,526	1,067	0,575	180,96
υ_s=0,3	1017,46	18,332	1,089	0,736	0,991	0,442	117,91
			2,076	0,533	1,065	0,604	161,00

Method 2

L/d_e=30/25,76 =1,164→ I_δ=0,4 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	125,88	106,51

$\delta_{measured}$=80 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
69	30	30	2,3	1	1,11
84	45	30	1,86	0,66	1,41

P=990000 KN υ_s= 0,1

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,91	0,36	181,5	478,26	25,89
0,905	0,24	271,5	261,90	9,42

$\delta_{i\ ave}$ = 35,31 mm

$m_\upsilon = [(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,736 → μ_d=0,786

Frankfurt Clay → μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

E_{mid-dr}	m_v	σ_z	δ_c
133,1	0,0073	294,13	35,39
199,1	0,0049	102,82	8,27

δ_c= 43,66 mm

$\delta_T = \delta_{i\ ave} + \delta_c$ = 78,99 mm

$\delta_{measured}$=80 mm

Table A.10: Measured and computed settlements for Forum Pollux (mm)

	Settlement (mm)										
	Set. Ratio	**Equivalent Pier**				**Equivalent Raft**					**Mea.**
		d_{e1}		d_{e2}		H=40 m	H=60 m	H=40 m (at the pile tip)	H=53,2 m (1/6)	H=50 m (1/8)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	Ave.	Ave.	Ave.	
υs=0,1	139,59	77,67	88,09	141,0	125,9	69,62	78,99	84,01	86,52	89,77	80
		123,2		181,0							
υs=0,3	123,67	58,37	74,54	117,9	106,5	54,7	63,39	68,2	69,41	72,14	
		106,5		161,0							

Forum Pollux

Rs	139,6	Mea.	80
Pier	77,67 88,09		
Raft	78,99		

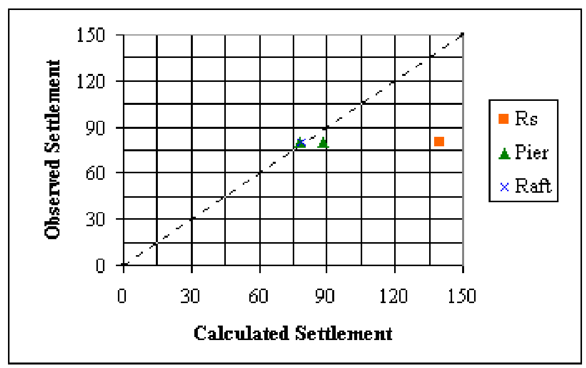

Figure A.18: Measured and computed settlements for Forum Pollux (mm)

Solution for Forum Kastor

a) Settlement Ratio Method

n= 22 d= 1,3 m r_0= 0,65 m

L= 25 m s= 5 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,1 υ_s =0,3 Frankfurt Clay

P=920 MN

$\lambda=E_p/G_l=30000/50,5 \approx 594,059$

$\rho=G_{l/2}/G_l=0,752\rightarrow 1$

$\log\lambda=2,773\rightarrow0,94$

s/d=5→0,875

L/d=19,23→0,535

υ_s=0,1→1,05

υ_s=0,3→1

$\eta_w=n^{-e}$ $R_s=n^e$ $\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al.an, 1992)

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=920000/22=41818,18 KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_tG_lr_0)$
υ_s=0,1	0,462	0,239	4,171	4,176	1,092	28,092	33,975
υ_s=0,3	0,440	0,256	3,896	3,925	1,126	27,649	35,975

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0{,}1$	1115,234	5882,159	156,40	37,49	156,40
$\upsilon_s=0{,}3$	1180,903	6666,851	137,99	35,41	137,99

$\delta_{measured}=75$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=38{,}73$ m

$A_P=\Pi d^2 n/4=29{,}201$ m^2

$E_p=30000$ MPa $E_s'=111{,}1$ MPa $E_u=151{,}5$ MPa

$d_e=1{,}27\ A_G^{0{,}5}=49{,}18$ (for friction piles)

$\rho=0{,}752$ $L=25$ m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}1$	692,959	13,722	0,543	0,526	0,465	0,384	64,64
			2,044	0,271	0,496	0,719	121,15
$\upsilon_s=0{,}3$	712,766	14,114	0,291	0,708	0,437	0,290	41,42
			1,993	0,271	0,496	0,727	103,55

Method 2

L/d_e=25/49,18 =0,508 → I_δ=0,5   (Fig. 2.10)

	υ_s=0,1	υ_u=0,5
δ (mm)	84,17	71,22

K ≈ 200 (pile stiffness factor)   s/d ≈ 5   L/d ≈ 19,23   B=41,53 m

d_e/B ≈ 0,77 assumed, then d_e ≈ 31,98 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,1	692,959	13,722	1,043	0,626	0,743	0,453	126,00
			2,059	0,446	0,786	0,634	176,22
υ_s=0,3	712,766	14,114	0,792	0,709	0,721	0,430	101,14
			1,975	0,449	0,786	0,659	155,02

Method 2

L/d_e=25/29,82 =0,838 → I_δ=0,45   (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	124,95	105,73

$\delta_{measured}$=75 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
83,33	28,33	28,33	2,94	0,99	1,06
97,49	42,49	28,33	2,29	0,66	1,37

P=900000 KN υ_s= 0,1

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B / E_u$

μ_0	μ_1	E_{uave}	q	δi
0,915	0,36	168,97	389,64	21,52
0,905	0,24	253,96	222,03	8,07

$\delta_{i\ ave}$ = 29,59 mm

$m_\upsilon = [(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,620 ⟶ μ_d=0,812

Frankfurt Clay ⟶ μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
123,91	0,0079	247,42	31,43
186,24	0,0052	97,41	8,23

δ_c= 39,67 mm

$\delta_T = \delta_{i\ ave} + \delta_c$ = 69,26 mm

$\delta_{measured}$=75 mm

Figure A.11: Measured and computed settlements for Forum Kastor (mm)

	Settlement (mm)										
	Set. Ratio	**Equivalent Pier**				**Equivalent Raft**					**Mea.**
		d_{e1}		d_{e2}		H=40 m	H=56,66 m	H=40 m (at the pile tip)	H=51 m (1/6)	H=48,32 m (1/8)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	Ave.	Ave.	Ave.	
υs=0,1	156,4	64,64 121,2	84,17	126 176,2	125	59,42	70,17	73,98	74,46	76,33	75
υs=0,3	138	41,42 103,6	71,22	101,1 155	105,7	45,81	56	60,07	59,56	61,04	

Forum Kastor

Rs	156,4	Mea.	75
Pier	121,2 84,17 126		
Raft	70,17		

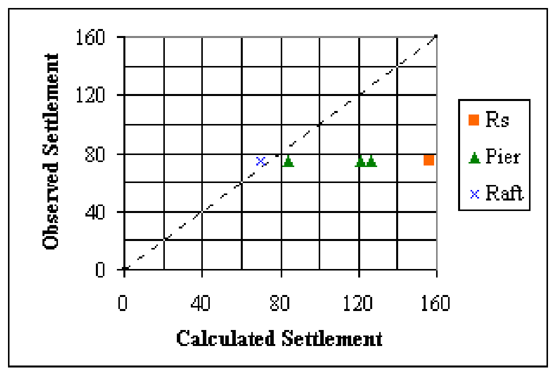

Figure A.19: Measured and computed settlements for Forum Kastor (mm)

11. American Express (n=35)

The American Express office building constructed in 1991-92 is situated about one km west of Messeturm tower. The raft of the 74 m high American Express building is loaded eccentricaly by the 16-storey office tower. To minimise tilting and differential settlement of the raft, 35 bored piles with a diameter of 0.9 m and a length of 20 m were located under the tower. (Katzenbach, R., Arslan,U., and Moormann, C., 2000)

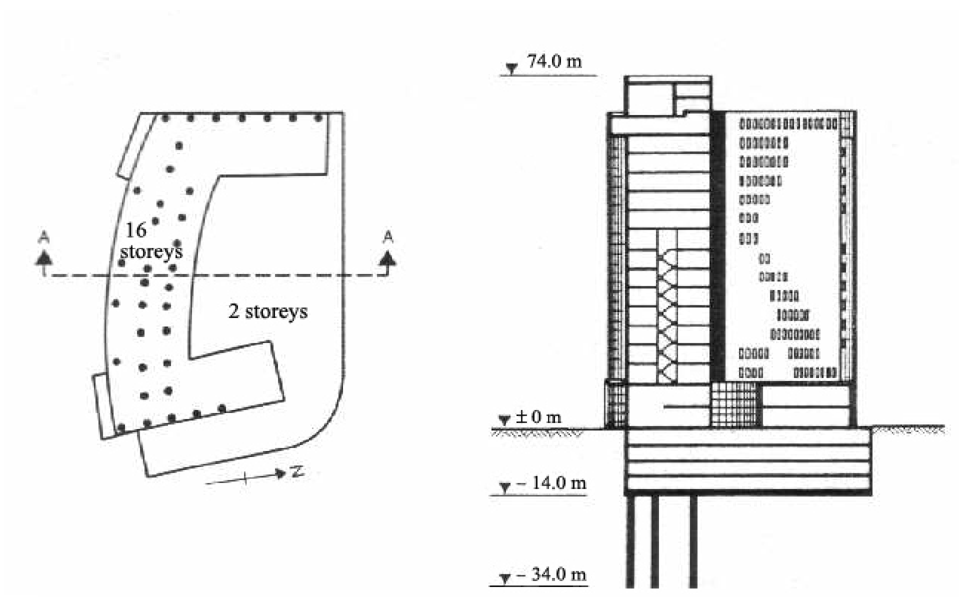

Figure A.20: American Express building, ground plan and section A-A (Katzenbach et al, 2000)

a) Settlement Ratio Method

n= 35 d= 0,9 m r_0= 0,45 m

L= 20 m s= 3,15 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,1 υ_s =0,3 Frankfurt Clay

P=1200 MN

$\lambda=E_p/G_l=30000/46 \approx 652{,}174$

$\rho=G_{l/2}/G_l=0{,}78\rightarrow1{,}01$

$\log\lambda=2{,}81\rightarrow0{,}95$

$s/d=3{,}5\rightarrow0{,}97$

$L/d=22{,}22\rightarrow0{,}54$

$\upsilon_s=0{,}1\rightarrow1{,}05$

$\upsilon_s=0{,}3\rightarrow1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=1200000/35=34285,7 KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,527	0,153	6,528	4,360	1,178	31,184	37,104
υ_s=0,3	0,502	0,167	5,970	4,109	1,214	30,671	39,078

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,1$	768,064	4117,534	291,43	44,64	291,41
$\upsilon_s=0,3$	808,925	4741,869	253,06	42,38	253,06

$\delta_{measured}=55$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=43,60$ m

$A_P=\Pi d^2 n/4=22,266\ m^2$

$E_p=30000$ MPa $E_s'=101,2$ MPa $E_u=138$ MPa

$d_e=1,27\ A_G^{0,5}=55,38$ (for friction piles)

$\rho=0,78$ $L=20$ m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0,1$	451,303	9,811	0,240	0,665	0,315	0,276	59,20
			1,836	0,240	0,354	0,764	163,72
$\upsilon_s=0,3$	469,48	10,206	-0,01	—	—	—	—
			1,789	0,238	0,354	0,764	138,49

Method 2

L/d_e=20/55,38=0,361 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	107,05	90,58

K ≈ 200 (pile stiffness factor) s/d ≈ 3,5 L/d ≈ 22,22 B=43,60 m

d_e/B ≈ 0,8 assumed, then d_e ≈ 34,85 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μLd_e)	I_δ	δ
υ_s=0,1	451,303	9,811	0,702	0,617	0,510	0,435	148,10
			1,948	0,370	0,548	0,706	240,16
υ_s=0,3	469,48	10,206	0,451	0,755	0,484	0,374	107,79
			1,882	0,369	0,548	0,724	208,39

Method 2

L/d_e=20/34,88=0,57 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	169,95	143,80

$\delta_{measured}$=55 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
92,16	28,9	28,9	3,18	1	0,94
106,61	43,35	28,9	2,46	0,66	1,29

P=1200000 KN υ_s= 0,1

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B / E_u$

μ_0	μ_1	E_{uave}	q	δi
0,92	0,36	161,25	450,54	26,74
0,91	0,24	247,95	259,65	9,91

$\delta_{i\ ave}$ = 36,65 mm

$m_\upsilon = [(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,529 → μ_d=0,838

Frankfurt Clay → μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
118,25	0,00827	286,54	40,16
181,8	0,00537	64,91	5,91

δ_c= 46,07 mm

$\delta_T = \delta_{i\ ave} + \delta_c$ = 82,72 mm

$\delta_{measured}$=55 mm

Table A.12: Measured and computed settlements for American Express (mm)

	Settlement (mm)											
	Set. Ratio	**Equivalent Pier**								**Equivalent Raft**		**Mea.**
		B'=32,36				**B'=43,60**						
		d_{e1}		d_{e2}		d_{e1}		d_{e2}		H=44,48 m	H= 57,8 m	
		Met1	Met2	Met1	Met2	Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,1	291,43	107,70 199,26	144,25	199,41 285,02	210,67	59,20 163,72	107,05	148,10 240,16	169,95	78,14	82,74	55
υs=0,3	253,06	68,00 170,04	122,05	157,77 249,24	178,26	___ 138,49	90,58	107,79 208,39	143,80	62,97	66,28	

American Express

Rs	291,43	Mea.	55
Pier	163,72 107,05 148,10		
Raft	82,74		

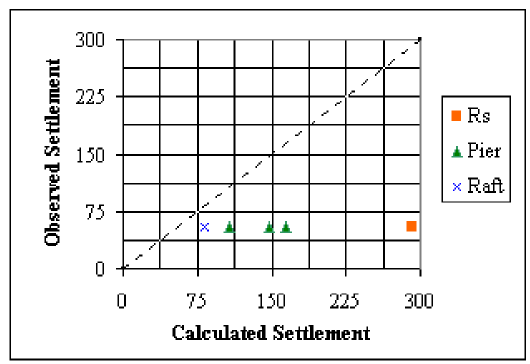

Figure A.21: Measured and computed settlements for American Express (mm)

12. Westend I Tower, Frankfurt (n=40)

The Westend 1 Tower is a 51-storey, 208 m high building in Frankfurt, Germany. The foundation for the tower consists of piled raft with 40 bored piles, each about 30 m long and 1.3 m in diameter. The central part of the raft is 4.5 m thick, decreasing to 3 m at the edges. The raft is founded at a depth of 14.5 m below the ground surface and about 9.5 m below groundwater level. The piles concentrated beneath the heavy columns of the superstructure. The 2940 m^2 raft of the skyscraper is separated by a settlement joint from the adjacent raft of the side building, which has a plan area of 3000 m^2. Hence the office tower is founded on its own centrically loaded piled raft. Using programs GARP (plate on springs approach) and GASP (a piled strip analysis), approximately 106 and 141 mm settlement predictions are obtained relatively. (Poulos, H.G., 2001, Katzenbach, R., Arslan,U., Moormann, C., 2000)

a) Settlement Ratio Method

$n= 40$ $d= 1,3$ m $r_0= 0,65$ m

$L= 30$ m $s/d= 5$

$G= 20+1,0z$ (MN/m^2) Stiff Frankfurt Clay $E_p= 30000$ MN/m^2

$P=1420$ MN $\upsilon_s= 0,1$ $\upsilon_s =0,3$

$\lambda=E_p/G_l=30000/53 \approx 566,03$

$\rho=G_{l/2}/G_l=0,717 \rightarrow 0,98$

$\log\lambda=2,752 \rightarrow 0,94$

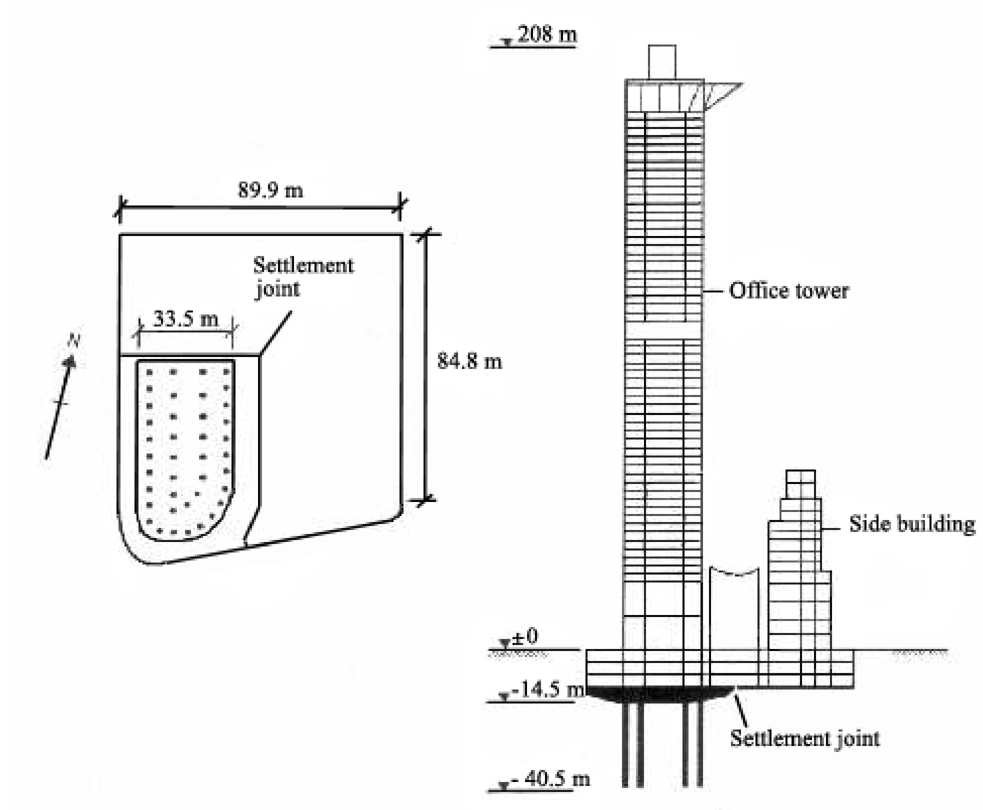

Figure A.22: Westend 1 Tower, Frankfurt; foundation plan and cross section (Poulos, 2001, Katzenbach et al, 2000)

s/d=5→0,875

L/d=23,07→0,54

υ_s=0,1→1,05

υ_s=0,3→1

$\eta_w=n^{-e}$ $R_s=n^e$ $\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, F. Randolph, K. Elson, J. Weltman, 1992)

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=1420000/40=35500 KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,457	0,185	5,397	4,310	1,321	30,286	33,558
υ_s=0,3	0,435	0,200	4,981	4,059	1,361	29,717	35,324

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	1156,103	8567,74	165,73	30,70	165,72
υ_s=0,3	1216,915	9772,29	145,30	29,17	145,30

$\delta_{measured}$=110 mm

b) Equivalent Pier Method

$B=A_G^{\,0.5}$ =46,28 m

$A_P=\Pi d^2 n/4$=53,09 m^2

E_p=30000 MPa

E_s'=116,6 MPa E_u=159 MPa

$d_e=1{,}27\ A_G^{0,5}$ =58,78 m (for friction piles)

ρ=0,716   L=30 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, F. Randolph, K. Elson, J. Weltman, 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L$ $L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	857,195	16,173	0,498	0,508	0,470	0,367	76,21
			1,894	0,260	0,499	0,701	145,36
υ_s=0,3	877,87	16,563	0,247	0,713	0,438	0,263	46,09
			1,837	0,261	0,499	0,706	123,93

Method 2

L/d_e=30/58,78 =0,51→ I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	103,58	87,65

K ≈ 200 (pile stiffness factor) s/d ≈ 5 L/d ≈ 23,07 B=46,28 m

d_e/B ≈ 0,78 assumed, then d_e ≈ 36,1 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L$ $L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	857,195	16,173	0,986	0,588	0,746	0,441	148,95
			2,038	0,409	0,787	0,631	213,09
υ_s=0,3	877,87	16,563	0,734	0,673	0,724	0,413	117,86
			1,958	0,412	0,786	0,653	186,47

Method 2

L/d_e=30/36,1=0,83 → I_δ=0,44 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	148,42	108,84

$\delta_{measured}$=110 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
69,82	42,2	42,2	1,65	1	0,81
90,92	63,3	42,2	1,43	0,66	1,21

P=1420000 KN υ_s= 0,1

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,92	0,36	195,3	481,94	34,49
0,91	0,24	321,9	246,73	10,59

$\delta_{i\ ave}$= 45,08 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,635 → μ_d=0,81

Frankfurt Clay → μ_g=0,7

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

E_{mid-dr}	m_v	σ_z	δ_c
143,22	0,0068	282,90	46,21
236,06	0,0041	96,38	9,55

δ_c= 55,76 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 100,85 mm

$\delta_{measured}$=110 mm

Table A.13: Measured and computed settlements for Westend I Tower (mm)

	Settlement (mm)										
	Set. Ratio	**Equivalent Pier**				**Equivalent Raft**					**Mea.**
		d_{e1}		d_{e2}		H=64,4 m	H=84,4 m	H=64,4 m (at the pile tip)	H=77,72 m (1/6)	H=74,4 m (1/8)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	Ave.	Ave.	Ave.	
υs=0,1	165,7	76,21 145,4	103,6	149 213,1	148,4	86,08	100,85	106,51	108,78	112,58	110
υs=0,3	145,3	46,1 123,9	87,65	117,9 186,5	125,6	69,95	80,93	86,4	87,41	90,61	

Westend I Tower (DG-Bank)

Rs	165,7	Mea.	110
Pier	145,4 103,6 149		
Raft	100,9		

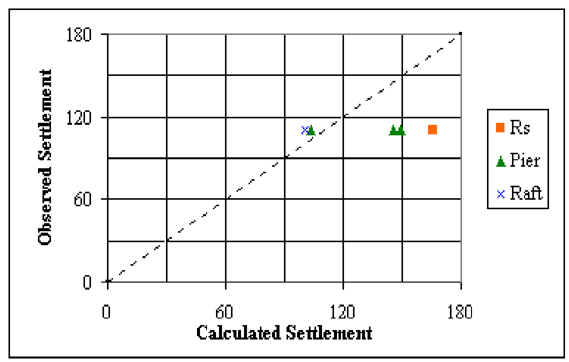

Figure A.23: Measured and computed settlements for Westend I Tower (mm)

13. Messe-Torhaus Building (n=42)

The 30-storey tall building is a 130-m-high structure supported by two identical separate piled rafts. The pile group beneath each raft comprised 42 bored piles having a diameter of 90 cm and length of 20 m. The pile spacing varied from $6r_0$ to $7r_0$. The subsoil below the raft was underlain by layers of Frankfurt clay extending to great depth. Within the clay, thin calciferous sand, silt incluions, and isolated floating limestone layers were embeded. Based on the computed settlement of single pile and group settlement ratio, the settlement of the pile group is obtained as 50 mm. (W.Shen, Y.Chow and K.Yong 2000)

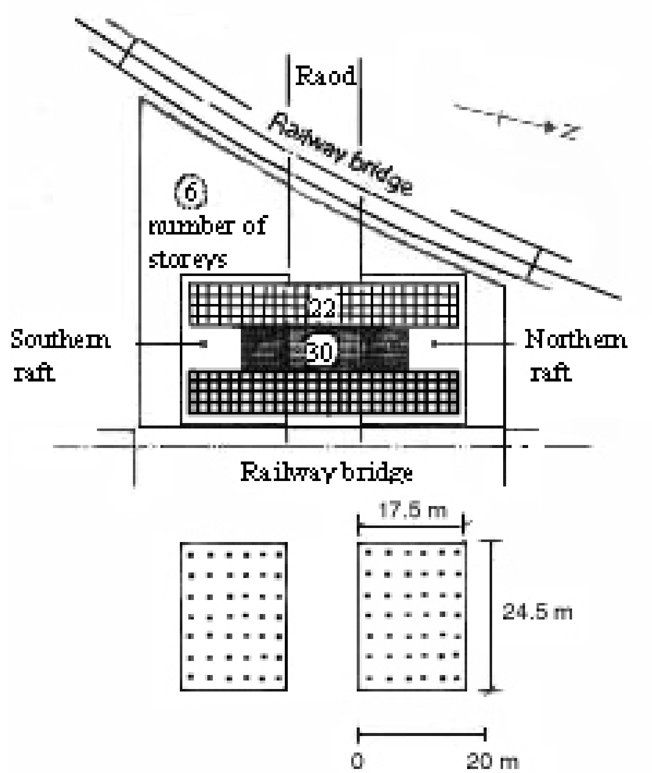

Figure A.24: Messe-Torhaus building, site plan (Shen et al, 2000)

a) Settlement Ratio Method

$G = 20+1{,}0z$ (MN/m^2)   Frankfurt clay $E_p = 30000$ MN/m^2

n= 42 piles (under each raft)

L= 20 m $s = 6r_0 - 7r_0$ d= 0,9 m $r_0 = 0{,}45$ m

P= 180,75 MN $\upsilon_s = 0{,}1$ $\upsilon_s = 0{,}3$

$\lambda = E_p/G_l = 30000/40 \approx 750$

$\rho = G_{l/2}/G_l = 0{,}75 \longrightarrow 1$

$\log\lambda = 2{,}875 \longrightarrow 0{,}97$

$s/d_{ave} = 3{,}25 \longrightarrow 0{,}98$

$L/d = 22{,}22 \longrightarrow 0{,}537$

$\upsilon_s = 0{,}1 \longrightarrow 1{,}05$

$\upsilon_s = 0{,}3 \longrightarrow 1$

$\eta_w = n^{-e}$ $R_s = n^e$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et.al 1992)

$\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 1$

$\mu L = (2/(\lambda\zeta))^{0{,}5} L/r_0$

$P_{single} = 180750/42 = 4303{,}57$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0{,}1$	0,536	0,134	7,427	4,317	1,104	32,275	37,395
$\upsilon_s=0{,}3$	0,511	0,148	6,751	4,066	1,138	31,778	39,498

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,1$	673,119	3806,068	47,48	6,39	47,48
$\upsilon_s=0,3$	710,971	4422,892	40,86	6,053	40,86

$\delta_{measured}$=45 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =20,706 m

$A_P=\Pi d^2 n/4$=26,719 m^2

E_p=30000 KPa

E_s'=88 MPa E_u=120 MPa

$d_e=1,27\ A_G^{0,5}$=26,296 m (for friction piles)

ρ=0,75 L=20 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al. 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu l\ d_e)$	I_δ	δ
$\upsilon_s=0,1$	1952,084	48,802	0,942	0,317	0,736	0,388	30,36
			2,023	0,216	0,748	0,578	45,20
$\upsilon_s=0,3$	1967,087	49,177	0,691	0,369	0,727	0,350	23,16
			1,945	0,219	0,748	0,587	38,83

Method 2

L/d_e=20/26,29=0,76 ⟶ I_s=0,47 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	36,71	31,06

K ≈ 250 (pile stiffness factor) s/d ≈ 3,25 L/d ≈ 22,22 B=20,7 m

d_e/B ≈ 0,82 assumed, then d_e ≈ 17 (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,1	1952,084	48,802	1,378	0,405	1,115	0,38	46,88
			2,193	0,321	1,137	0,502	60,73
υ_s=0,3	1967,087	49,177	1,127	0,446	1,104	0,376	38,48
			2,090	0,328	1,136	0,520	53,18

Method 2

L/d_e=20/17=1,17 ⟶ I_δ=0,38 (Fig 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	45,91	38,85

$\delta_{measured}$=45 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
30,26	23,26	15,5	1,300	0,666	0,571
38,01	31,01	15,5	1,225	0,499	0,928
45,76	38,76	15,5	1,180	0,399	1,142

P=180750 KN υ_s= 0,1

q_n=P/(B*L) = 240,272 KN

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,93	0,23	123,15	256,802	10,374
0,92	0,18	169,65	153,348	4,641
0,91	0,13	216,15	101,908	2,161

$\delta_{i\ ave}$ = 17,17 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,501 ⟶ μ_d=0,86

Frankfurt Clay ⟶ μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
90,31	0,0108	172,057	17,38
124,41	0,0078	74,472	5,46
158,51	0,0061	19,935	1,14

δ_c= 23,99 mm

$\delta_T = \delta_{i\ ave\ 1} + \delta_c$ = 41,17 mm

$\delta_{measured}$=45 mm

Table A.14: Measured and computed settlements for Messe Torhaus (mm)

<table>
<tr><th rowspan="4"></th><th colspan="8">Settlement (mm)</th></tr>
<tr><th rowspan="3">Set. Ratio</th><th colspan="4">Equivalent Pier</th><th colspan="2">Equivalent Raft</th><th rowspan="3">Mea.</th></tr>
<tr><th colspan="2">d_{e1}</th><th colspan="2">d_{e2}</th><th>H=35 m</th><th>H=46,46 m</th></tr>
<tr><th>Met1</th><th>Met2</th><th>Met1</th><th>Met2</th><th>Ave.</th><th>Ave.</th></tr>
<tr><td rowspan="2">υs=0,1</td><td rowspan="2">47,49</td><td>30,36</td><td rowspan="2">36,71</td><td>46,88</td><td rowspan="2">45,91</td><td rowspan="2">35,42</td><td rowspan="2">41,17</td><td rowspan="4">45</td></tr>
<tr><td>45,21</td><td>60,73</td></tr>
<tr><td rowspan="2">υs=0,3</td><td rowspan="2">40,86</td><td>23,16</td><td rowspan="2">31,08</td><td>38,48</td><td rowspan="2">38,84</td><td rowspan="2">29,04</td><td rowspan="2">32,60</td></tr>
<tr><td>38,83</td><td>53,18</td></tr>
</table>

Messe Torhaus

Rs	47,49	Mea.	45
Pier	30,36 36,71		
Raft	41,17		

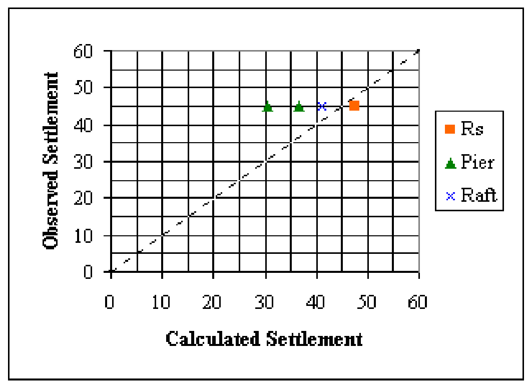

Figure A.25: Measured and computed settlements for Messe Torhaus (mm)

14. Gratham Road (n=48)

This is a 22-storey structure constructed at a site in London Borough of Lambeth. A site investigation showed 2 m Flood Plain Gravels overlying London Clay (c_u=60-180 kN/m^2). It was decided that large diameter underreamed (1.52 to 2.74 m) piles taken to depths of 19 m should be constructed to support the building. Pile lenghts are between 16.2 and 18.7m and shafts are 0.76 and 0.91 m diameter. (Morton, K., and Au, E., 1974)

a) Settlement Ratio Method

n= 48 r_0= 0,455 m r_b= 1,065 m

L= 17,45 m s= 3,64 m

c_u= 60-180 (kN/m^2) London Clay N=62-136 Flood Plain Gravel

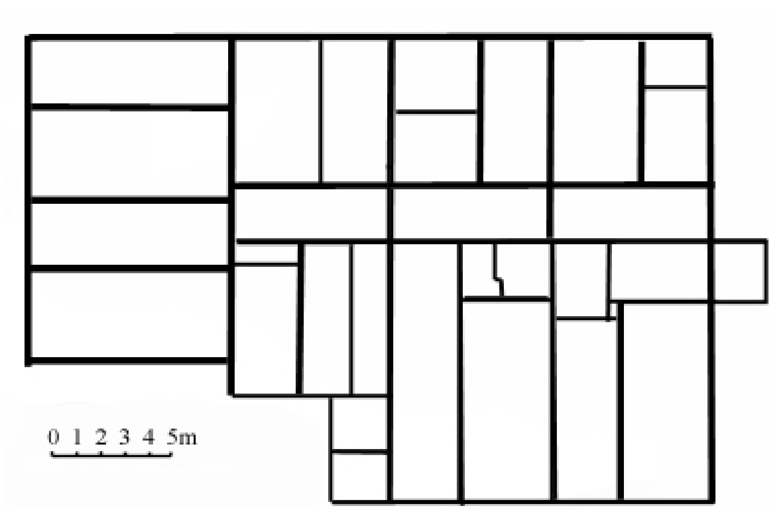

Figure A.26: Gratham Road foundation plan (Morton and Au, 1974)

E_p= 30000 MN/m^2

υ_s= 0,1 υ_s =0,3   P=100 MN

$\lambda=E_p/G_l=30000/25 \approx 1200$

$\rho=G_{l/2}/G_l=20/25=0,8\rightarrow1,02$

$\log\lambda=3,079\rightarrow1,01$

$s/d=4\rightarrow0,93$

$L/d=19,175\rightarrow0,535$

$\upsilon_s=0,1\rightarrow1,05$

$\upsilon_s=0,3\rightarrow1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=2,34$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=100000/48=2083 KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_tG_lr_0)$
υ_s=0,1	0,538	0,124	8,032	4,235	0,760	32,339	44,793
υ_s=0,3	0,512	0,137	7,273	3,983	0,784	32,034	48,309

	P_t/w_t	$K=n\eta_wk$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_sR_s$
υ_s=0,1	509,523	3044,762	32,84	4,088	32,84
υ_s=0,3	549,522	3626,289	27,57	3,790	27,57

$\delta_{measured}$=30 mm

b) Equivalent Pier Method

$B = A_G^{\ 0.5} = 23{,}47$ m

$A_P = \Pi d^2 n/4 = 31{,}218$ m^2

$E_p = 30000$ MPa

$E_s' = 55$ MPa $E_u = 75$ MPa

$d_e = 1{,}27\ A_G^{0{,}5} = 29{,}80$ (for friction piles)

$\rho = 0{,}8$ $L = 17{,}45$ m

$E_e = E_p A_p/A_G + E_s(1 - A_p/A_G)$

$\zeta_1 = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al. 1992)

$\zeta_2 = \ln/\{5 + [0{,}25 + (2{,}5\ \rho\ (1-\upsilon) - 0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}1$	17,52,15	70,086	0,745	0,229	0,575	0,369	22,50
			1,961	0,141	0,581	0,606	36,99
$\upsilon_s=0{,}3$	1761,59	70,463	0,494	0,280	0,570	0,309	15,95
			1,893	0,143	0,581	0,608	31,040

Method 2

$L/d_e = 17{,}45/29{,}80 = 0{,}585 \rightarrow I_\delta = 0{,}5$ (Fig. 2.10)

	$\upsilon_s=0{,}1$	$\upsilon_s=0{,}3$
δ (mm)	30,49	25,80

$K \approx 400$ (pile stiffness factor) $s/d \approx 4$ $L/d \approx 19{,}175$ $B=23{,}47$ m

$d_e/B \approx 0{,}84$ assumed, then $d_e \approx 19{,}71$ m   (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}1$	17,52,15	70,086	1,158	0,277	0,863	0,381	35,19
			2,102	2,206	0,872	0,528	48,74
$\upsilon_s=0{,}3$	1761,59	70,463	0,907	0,313	0,857	0,357	27,86
			2,010	0,210	0,872	0,539	42,10

Method 2

$L/d_e=17{,}45/29{,}065=0{,}6 \rightarrow I_\delta=0{,}5$ (Fig. 2.10)

	$\upsilon_s=0{,}1$	$\upsilon_s=0{,}3$
δ (mm)	42,42	35,89

$\delta_{measured}=30$ mm

c) Equivalent Raft Method

$L=34{,}38$ $B=25{,}09$ $H=50{,}18$ $D=12{,}63$

$L/B=1{,}37$ $H/B=2$

$E_{uave}=146{,}4$ $E_s'=107{,}36$

$q_n=100000/(BL) = 115{,}93$ Kpa

μ_0=0,93 μ_1=0,56

$\delta_{i\ ave}$=q_n B μ_0 μ_1/E_u = 115,93 . 25,09 . 0,93 . 0,56 / 146,4 = 10,34 mm

D/(LB)^{0,5}=0,430→ μ_d=0,876

London Clay→ μ_g=0,7

z/B=1 σ_z/q=0,3 σ_z=34,77 KPa

m_υ=[(1+υ)(1-2υ)]/[E_s'(1-υ)] ≈ 0,0091

δ_c=m_υ σ_z H μ_d μ_g

=0,0091 . 34,77 . 50,18 . 0,876 . 0,7 ≈ 9,74 mm

δ_T =δ_i+δ_c = **20,09 mm**

$\delta_{measured}$=30 mm

Table A.15: Measured and computed settlements for Gratham Road (mm)

	Settlement (mm)										
	Set. Ratio	Equivalent Pier				Equivalent Raft					Mea.
		d_{e1}		d_{e2}		H=38,56 m	H=50,18 m	H=38,56 m (at the tip)	H=46,3 m (1/6)	H=44,36 m (1/8)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	Ave.	Ave.	Ave.	
υs=0,1	32,84	22,50 36,99	30,49	35,19 48,74	42,42	20,29	20,09	23,94	22,01	22,82	30
υs=0,3	27,57	15,95 31,40	25,80	27,86 42,01	35,89	16,56	16,61	19,86	18,09	18,78	

Gratham

Rs 32,84 Mea. 30

Pier 22,5
30,49

Raft 20,09

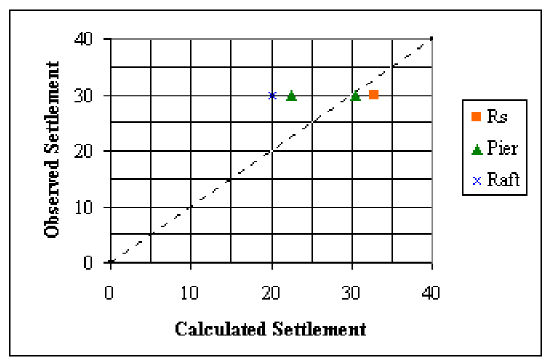

Figure A.27: Measured and computed settlements for Gratham Road (mm)

15. Treptowers Building (n=54)

The 121 m high Treptowers office building in Berlin is 37.1 m square in plan, and is located close to the river Spree. The raft of thickness 2-3 m is founded 8 m below ground level in the remainin area of he elevator pit, and 5.5 below ground level in the remaining area. To transmit part of the total building load of 670 MN through the loose sands just below raft level to medium dense to dense sand at greater depth, 54 bored piles with diameter of 0.9 m have been arranged under the raft. Due to the different founding levels of the raft, the length of piles varies between 12.5 m and 16.0 m. At the end of the construction, the mean settlement of the building was 63 mm.(Katzenbach, R., Arslan,U., and Moormann, C., 2000)

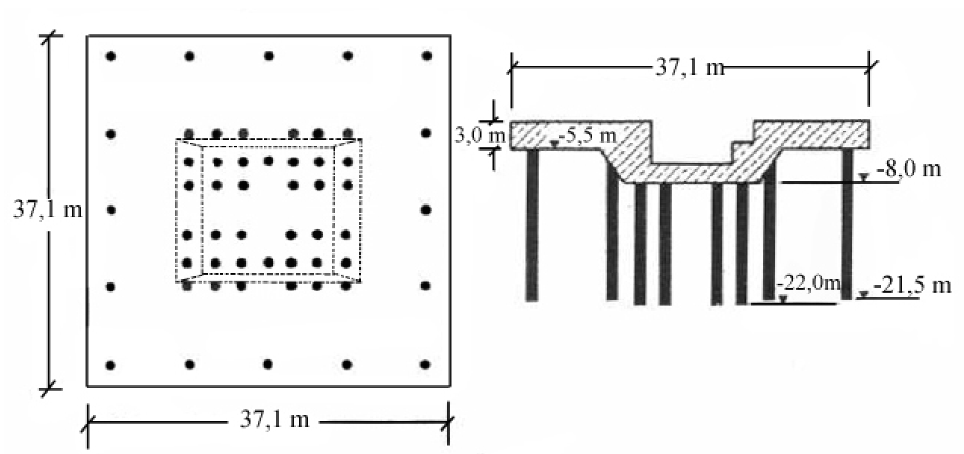

Figure A.28: Treptowers building, Berlin; plan and cros-section of piled raft foundation (Katzenbach et al, 2000)

a) Settlement Ratio Method

$n = 54$ $d = 0{,}9$ m $r_0 = 0{,}45$ m

$L = 14{,}25$ m $s = 5$ m

$E_u \approx 20000 \sqrt{z}$ kPa   for $0<z<20$ m

$E_u \approx 60000 \sqrt{z}$ kPa   for $z>20$ m   (z=depth below surface)

Berlin Sand $\upsilon_s = 0{,}25$ $\upsilon_s = 0{,}35$ $\upsilon_u = 0{,}5$

$E_p = 30000$ MN/m^2

P=670 MN

$\lambda = E_p/G_l = 30000/31{,}27 \approx 959{,}386$

$\rho = G_{l/2}/G_l = 0{,}825 \rightarrow 1{,}02$

$\log\lambda = 2{,}981 \rightarrow 0{,}99$

$s/d = 5 \rightarrow 0{,}87$

$L/d = 15{,}83 \rightarrow 0{,}528$

$\upsilon_s = 0{,}25 \rightarrow 1{,}02$

$\upsilon_s = 0{,}35 \rightarrow 0{,}98$

$\upsilon_u = 0{,}5 \rightarrow 0{,}93$

$\eta_w = n^{-e}$ $R_s = n^e$ $\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 0{,}333$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single} = 670000/54 = 12407{,}4$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,25	0,473	0,151	6,601	3,074	0,824	26,020	52,613
υ_s=0,35	0,454	0,163	6,130	2,968	0,839	25,862	54,934
υ_u=0,5	0,431	0,179	5,589	2,784	0,866	25,566	59,498

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,25	740,352	6055,767	110,63	16,75	110,63
υ_s=0,35	773,011	6808,638	98,40	16,05	98,40
υ_u=0,5	837,226	8089,04	82,82	14,81	82,82

$\delta_{measured}$=63 mm

b) Equivalent Pier Method

$B=A_G^{0.5}$ =37,1 m

$A_P=\Pi d^2 n/4$=34,35 m^2

E_p=30000 Mpa

E_s'=78,167 MPa E_u=93,8 MPa

$d_e=1{,}13\ A_G^{0,5}$=41,923 m (for friction piles)

ρ=0,825   L=14,25 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,25	824,975	26,385	-0,766				
			1,698	0,143	0,337	0,313	64,044
υ_s=0,35	831,072	26,58	-0,873				
			1,689	0,143	0,337	0,302	57,27
υ_u=0,5	840,218	26,872	-1,057				
			1,676	0,143	0,337	0,274	46,756

Method 2

L/d_e=14,25/41,923=0,34→ I_δ=0,26

	υ_s=0,25	υ_s=0,35	υ_u=0,5
δ (mm)	53,15	49,22	44,29

K ≈ 300 (pile stiffness factor)   s/d ≈ 5   L/d ≈ 15,833   B=37,1 m

d_e/B ≈ 0,81 assumed, then d_e ≈ 30,051 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,25	824,975	26,385	-0,433				
			1,731	0,198	0,468	0,314	89,63
υ_s=0,35	831,072	26,58	-0,540				
			1,719	0,198	0,468	0,306	80,95
υ_u=0,5	840,218	26,872	-0,724				

			1,701	0,198	0,468	0,283	67,31

Method 2

L/d_e=14,25/30,051=0,474→ I_δ=0,25

	υ_s=0,25	υ_s=0,35	υ_u=0,5
δ (mm)	71,037	66,02	59,422

$\delta_{measured}$=63 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
37,1	37,1	37,1	1	1	0,59
55,65	55,65	37,1	1	0,66	1,06

P=670 KN υ_s= 0,25

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,93	0,36	382,07	486,77	15,82
0,92	0,24	528,71	216,34	5,02

$\delta_{i\ ave}$ = 20,85 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,592 → μ_d=0,82

Berlin Sand→μ_g=1

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
318,33	0,00571	250,68	43,54
588,48	0,004	70,58	8,59

δ_c= 52,10 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 72,98 mm

qc	mv	δc	δT
25	0,008	63,97	87,749
30	0,0067	53,57	77,349
35	0,00571	45,66	69,439
40	0,005	39,98	63,759
45	0,0044	35,18	58,959
	ave	70,808	

$\delta_{measured}$=63 mm

Table A.16: Measured and computed settlements for Treptowers Building (mm)

	Settlement (mm)						
	Set. Ratio	**Equivalent Pier**				**Eq.Raft**	**Mea.**
		d_{e1}		d_{e2}		Ave.	
		Met1	Met2	Met1	Met2		
υs=0,35	110,63	— 64,04	53,15	— 89,63	71,3		
υs=0,35	98,40	— 57,27	49,22	— 80,95	66,02	72,98	63
υs=0,50	82,82	— 46,75	44,29	— 67,31	59,42		

Treptowers

Rs	94,4	Mea.	63
Pier	80,95		
Raft	72,98		

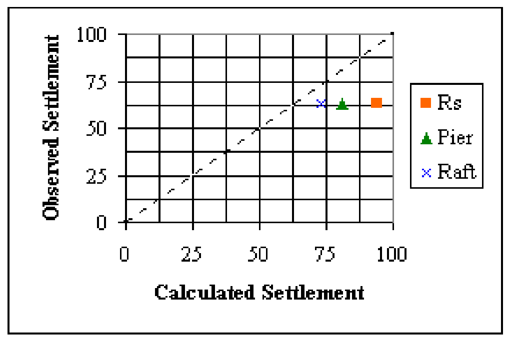

Figure A.29: Measured and computed settlements for Treptowers Building (mm)

16. Molasses Tank (n=55)

The tank under examination was constructed in Scotland in 1978 to store molasses. It is supported by 55 precast concrete piles, each 250*250 mm^2 in cross section and 29 m long, laid out on a triangular grid at a spacing of 2 m. A 2 m thick pad of dense granular material was constructed over the piles and incorporated a 150 mm thick reinforced concrete membrane connecting the pile heads. The effective pile length was then reduced to 27 m.

The foundation soil is a silty clay with interbedded sandy silt and silty sand layers until a maximum depth of 18 m below ground level, overlying a slightly over consolidated silty clay with occasional intercalations of sand and silt. According to Randolph (1994), the subsoil can be modelled as a unique cohesive layer. The average settlement is calculated as 29.4 mm by using non-linear analysis by the program GRUPPALO. Linear elastic analysis slightly underpredicts the settlement for this case. Average settlement may be estimated as 27.8 mm Randolph (1994). (Mandolini, A., and Viggiani, C., 1997, Randolph, M.F., 1994, Randolph, M.F., and Guo, W.D., 1999)

a) Settlement Ratio Method

n= 55 d= 0,28 m r_0= 0,141 m

L= 27 m s= 2 m

E= 4,5+1,35z (MN/m^2) E_p= 20000 MN/m^2

P=24,2 MN υ_s= 0,2 υ_s= 0,3

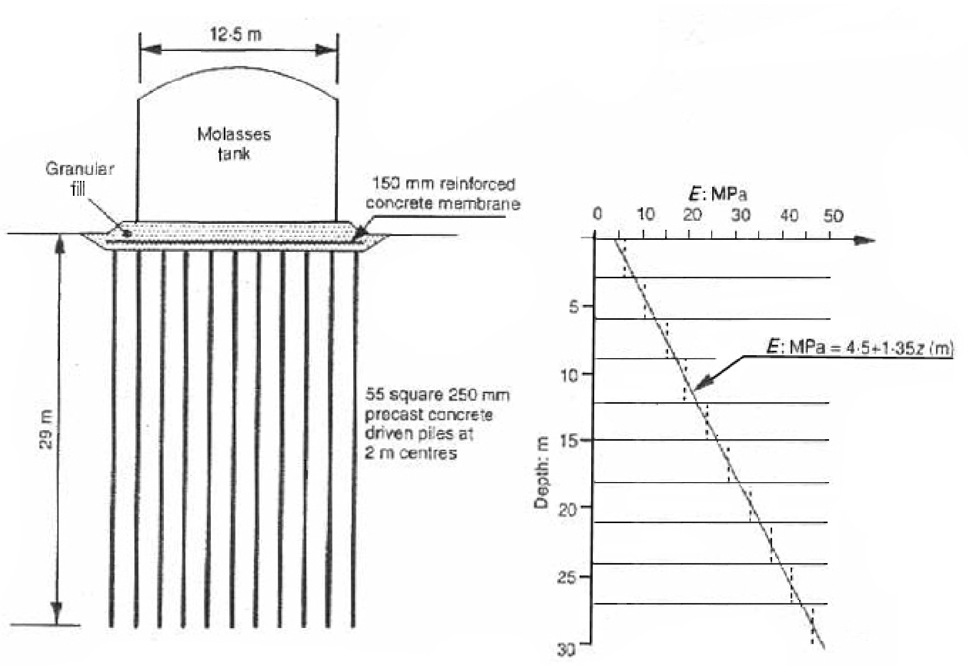

Figure A.30: Schematic of the Molasses tank and subsoil model adopted in the analysis (Mandolini and Viggiani, 1997, Randolph, 1994, Randolph and Guo, 1999)

$\lambda = E_p/G_l = 20000/14{,}55 \approx 1374{,}57$

$\rho = G_{l/2}/G_l = 0{,}582 \rightarrow 0{,}95$

$\log\lambda = 3{,}138 \rightarrow 1{,}03$

$s/d = 7 \rightarrow 0{,}79$

$L/d = 95{,}74 \rightarrow 0{,}51$

$\upsilon_s = 0{,}2 \rightarrow 1{,}03$

$\upsilon_s = 0{,}3 \rightarrow 1$

$\eta_w = n^{-e}$ $R_s = n^e$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1 \quad \xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=24200/55=440 KN

	e	η_w	R_s	ζ	μL	tanhμL L/(μL r_0)	$P_t/(w_tG_lr_0)$
υ_s=0,2	0,406	0,196	5,089	5,408	3,141	60,735	43,076
υ_s=0,3	0,394	0,206	4,854	5,274	3,180	59,998	43,866

	P_t/w_t	$K=n\eta_wk$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_sR_s$
υ_s=0,2	88,373	954,960	25,34	4,97	25,34
υ_s=0,3	89,994	1019,67	23,73	4,88	23,73

$\delta_{measured}$=29,5 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =12,071 m

$A_P=\Pi d^2n/4$=3,386 m^2

E_s'=34,92 MPa $\quad E_u$=43,65 MPa $\quad E_p$=20000 MPa

$d_e=1,27\ A_G^{0,5}$=15,331 m (for friction piles)

ρ=0,582 $\quad$ L=27 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,2	498,897	34,288	1,411	0,716	1,511	0,426	19,27
			2,208	0,572	1,591	0,536	24,24
υ_s=0,3	501,74	34,483	1,278	0,750	1,491	0,422	17,61
			2,150	0,578	1,587	0,546	22,78

Method 2

L/d_e=27/15,33=1,76 → I_δ=0,33 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	14,91	13,77

K ≈ 460 s/d ≈ 7 L/d ≈ 95,74 B=12,07 m

d_e/B ≈ 0,44 assumed, then d_e ≈ 5,311 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,2	498,897	34,288	2,471	1,561	2,980	0,443	57,82
			2,823	1,461	3,123	0,472	61,68
υ_s=0,3	501,74	34,483	2,338	1,601	2,926	0,457	55,09
			2,732	1,481	3,094	0,492	59,31

Method 2

L/d_e=27/5,31=5,08 → I_δ=0,21 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	27,39	25,29

$\delta_{measured}$=29,5 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
26,6	17,28	17,28	1,54	1	1,15
35,24	25,92	17,28	1,36	0,66	1,43

P=24200 KN υ_s= 0,2

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,91	0,36	43,16	52,64	6,90
0,907	0,24	66,49	26,49	2,24

$\delta_{i\ ave}$= 9,15 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,932 ⟶ μ_d=0,74

Cohesive layer ⟶ μ_g=1

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

E_{mid-dr}	m_v	σ_z	δ_c
34,53	0,0260	30,01	10,00
53,19	0,0169	9,47	2,05

δ_c= 12,05 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 21,20 mm

$\delta_{measured}$=29,5 mm

Table A.17: Measured and computed settlements for Molasses Tank (mm)

	Settlement (mm)										
	Set. Ratio	**Equivalent Pier**				**Equivalent Raft**					**Mea.**
		d_{e1}		d_{e2}		H=16,56 m	H=34,56 m	H=16,56 m (at the tip)	H=28,56 m (1/6)	H=25,56 m (1/8)	
		Met1	Met2	Met1	Met2	Ave.	Ave.	Ave.	Ave.	Ave.	
υs=0,2	25,34	19,27 24,24	14,91	57,82 61,68	27,40	16,75	21,2	27,71	25,11	27,89	29,5
υs=0,3	23,73	17,61 22,78	13,77	55,09 59,31	25,29	14,38	18,33	24,16	21,75	24,13	

Molasses Tank

Rs	25,34	Mea.	29,5
Pier	19,27 14,91		
Raft	21,6 21,2		

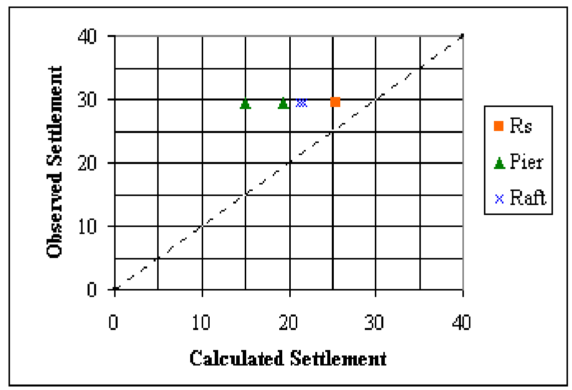

Figure A.31: Measured and computed settlements for Molasses Tank (mm)

17. Messeturm Tower (n=64)

The building has a basement with two underground floors, 58,8 m square in plan, and a 60-storey core shaft (41 m* 41 m in plan) up to height of 210 m. The estimated total load of the building is 1880 MN. At the site of the Messeturm building there are gravels and sands with a thickness of 8 m, followed by Frankfurt Clay to a depth of more than 100 m below the ground surface.

In order to reduce settlements and tilt, the foundation system comprised a base slab or raft supported and stabilised against tilt by 64 large diameter bored piles. The raft is founded at a depth of 14 m below the ground surface on the Frankfurt Clay, and is 9 m below the grounwater table. The thickness of the raft decrease from 6.0 m at the centre to 3.0 m at the edges. The bored piles have a diameter of 1.3 m and are arranged in three concentric circles below the raft. The distance between the piles varies from 3,5 to 6 pile diameters. The pile length varies from 26.9 m for the 28 piles in the outer circle to 30.9 m for the 20 piles in the middle circle, and to 34.9 m for the 16 piles in the inner circle. Calculated range of settlement is 150-200 mm using different methods.(Katzenbach, R. et al., 2000, Poulos, H.G., 2000, Poulos, H.G., 2001)

a) Settlement Ratio Method

n= 64 d= 1,3 m r_0= 0,65 m s= 4,75 m

P=1.880 MN L= 30,9 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

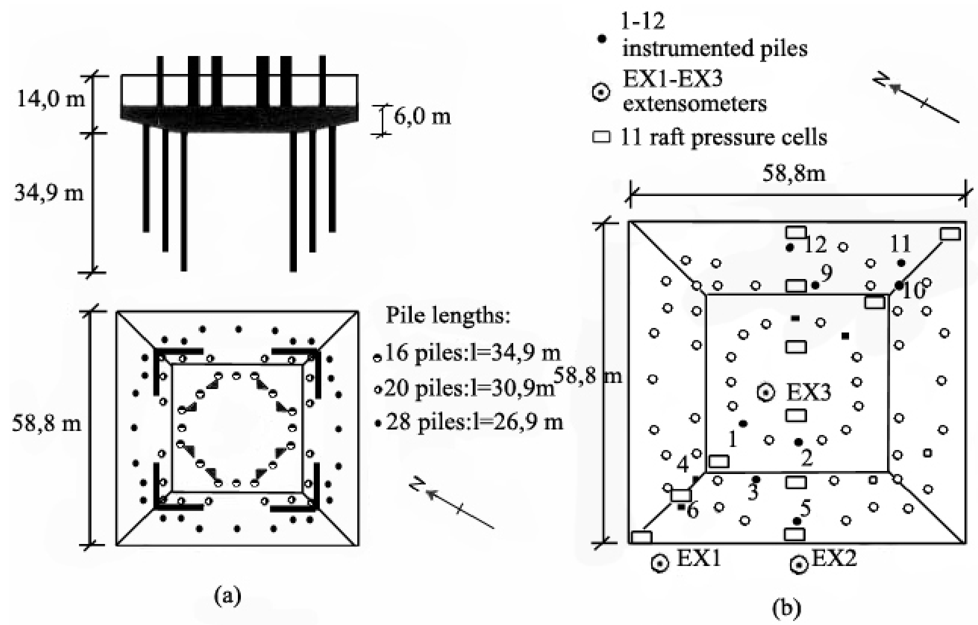

Figure A.32: Piled raft foundation for Messeturm building, **(a)** plan and cross-section **(b)** location of instrumentation. (Katzenbach et al, 2000, Poulos, 2000, Poulos, 2001)

υ_s= 0,1 υ_s=0,3   Frankfurt Clay

$\lambda=E_p/G_l=30000/56,9 \approx 572,24$

$\rho=G_{l/2}/G_l=0,728\rightarrow0,99$

$\log\lambda=2,722\rightarrow0,93$

$s/d=4,75\rightarrow0,88$

$L/d=23,769\rightarrow0,54$

$\upsilon_s=0,1\rightarrow1,05$

$\upsilon_s=0,3\rightarrow1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

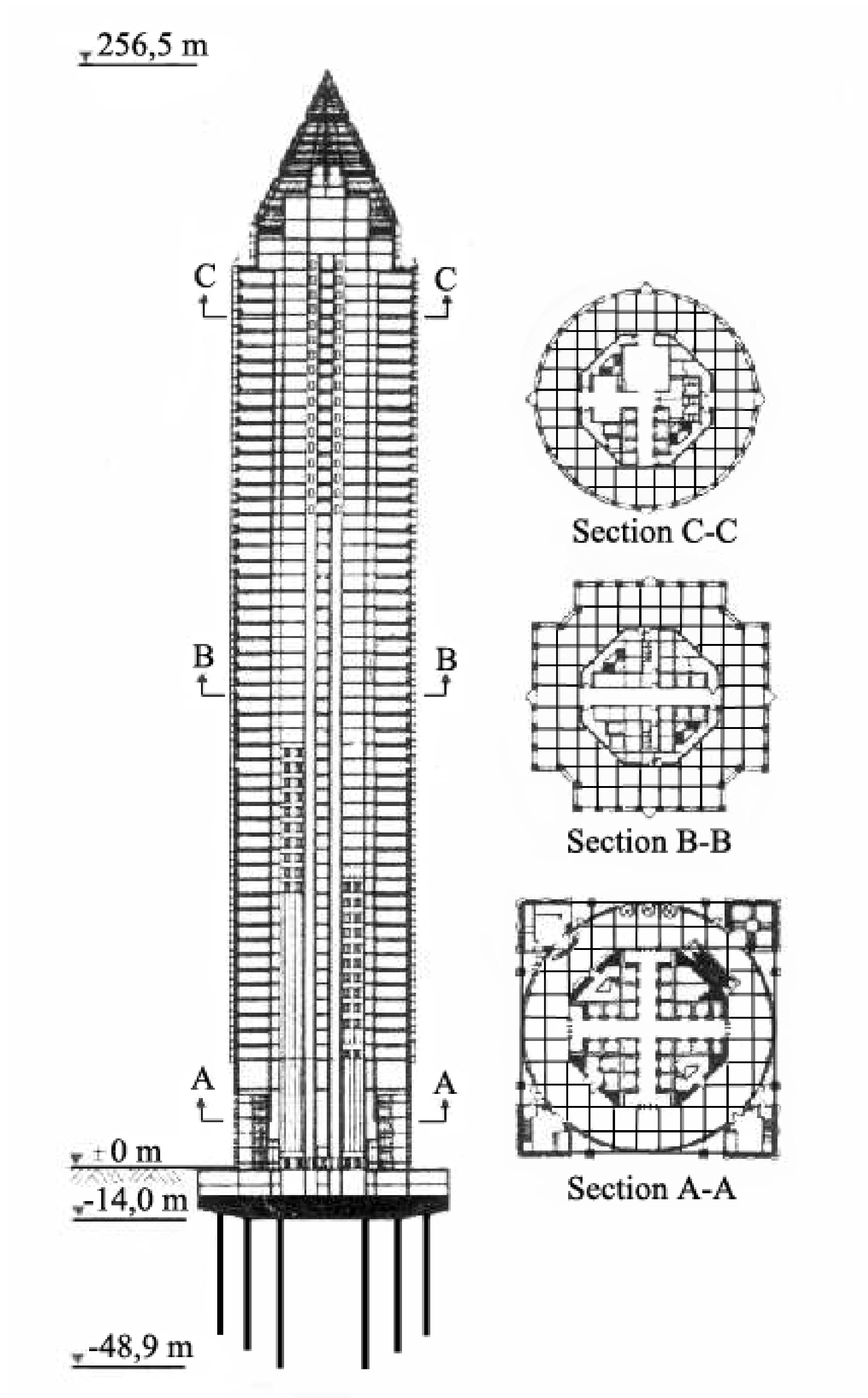

Figure A.33: Messeturm building, cross-sections .(Katzenbach et al, 2000, Poulos, 2000, Poulos, 2001)

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

P_{single}=1880000/64=29375 KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,459	0,147	6,756	4,356	1,403	30,022	33,309
υ_s=0,3	0,437	0,162	6,169	4,104	1,445	29,431	34,983

	P_t/w_t	$K=n\eta_w k$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	1231,954	11668,88	161,11	23,84	161,11
υ_s=0,3	1293,872	13422,63	140,06	22,70	140,06

$\delta_{measured}$=130 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =58,8 m

$A_P=\Pi d^2 n/4$=84,9487 m^2

E_p=30000 MPa

E_s'=125,18 MPa E_u=170,7 MPa

$d_e=1,27\ A_G^{0,5}$=74,676 (for friction piles)

ρ=0,728   L=30,9 m

$E_e=E_p A_p/A_G + E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	859,199	15,100	0,304	0,545	0,377	0,298	60,08
			1,849	0,221	0,407	0,733	147,43
υ_s=0,3	881,4	15,49	0,053	1,285	0,276	0,104	17,80
			1,800	0,221	0,407	0,732	124,55

Method 2

L/d_e=30,9/74,676 =0,413 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	100,55	85,08

K ≈ 200 (pile stiffness factor) s/d ≈ 4,75 L/d ≈ 23,769 B=58,8 m

d_e/B ≈ 0,77 assumed, then d_e ≈ 45,276 m (Fig.2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	859,199	15,100	0,805	0,553	0,620	0,427	141,70
			1,979	0,353	0,655	0,660	219,20
υ_s=0,3	881,4	15,49	0,554	0,659	0,598	0,380	106,69
			1,908	0,355	0,655	0,677	190,12

Method 2

L/d_e=30,9/45,276=0,68 → I_δ=0,47 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	155,90	131,91

$\delta_{measured}$=130 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
62	62	40	1	0,645	0,558
82	82	40	1	0,487	0,909

P=1880000 KN υ_s= 0,1

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,93	0,23	199,8	489,07	32,46
0,92	0,18	320,4	279,59	11,84

$\delta_{i\ ave}$= 44,31 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,558 ⟶ μ_d=0,83

Frankfurt Clay ⟶ μ_g=0,7

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

E_{mid-dr}	m_v	σ_z	δ_c
146,52	0,0066	322,78	50,06
234,96	0,0041	134,49	13,00

δ_c= 63,06 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 107,38 mm

$\delta_{measured}$=130 mm

Table A.18: Measured and computed settlements for Messeturm Building (mm)

	Settlement (mm)									
	Set. Ratio	Equivalent Pier				Equivalent Raft (ave.)				Mea.
		d_{e1}		d_{e2}		H=80 m	H=71 m (at the tip)	H=80 m (1/6)	H=80 m (1/8)	
		Met1	Met2	Met1	Met2					
υs=0,1	161,11	60,08 147,43	100,55	141,7 219,2	155,9	107,38	113,75	115,21	120,88	130
υs=0,3	140,06	17,8 124,55	85,08	106,69 190,12	131,91	84,85	90,75	91,24	95,91	

Messe Turm

Rs	161,11	Mea.	130
Pier	147,43 100,55 141,70		
Raft	107,38		

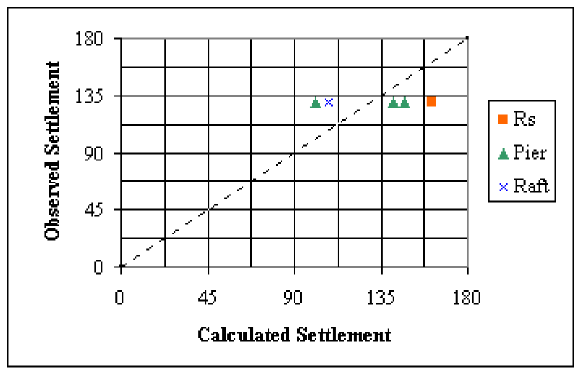

Figure A.34: Measured and computed settlements for Messeturm Building (mm)

18. New Law Court Building I, Naples (n=82-77-82)

The building (cases 18,19,20) belongs to the New Directional Centre of Naples, in the eastern area of the town. It consists of three towers, ranging in the height between 70 m (Tower C) and 110 m (Tower A), and has a steel frame structure with reinforced concrete stiffening cores.

The foundation is a reinforced concrete slab, 1 m thick, stiffned by heavy reinforced concrete frames in lower stages and resting on 241 (Tower A:82, Tower B:77, Tower C:82), bored piles with a length of 42 m and diameters ranging between 1.5 m and 2.2 m. Equipped with a preloading cell at the base.

Starting from the ground surface and moving downwords, the following soils are typically found; made ground; volcanic ashes and organic soils; stratified sands; pozzolana, cohesionless or slightly indurated; volcanic tuff. The groundwater table is found at a shallow depth below the ground surface, located at an average elevation of 5 m above mean sea level. The predicted settlements by GRUPPALO (NL analysis) are 32,7 mm, 32,5 mm, 24,8 mm for Towers A,B,C.(Mandolini, A., and Viggiani, C., 1997)

a) Settlement Ratio Method

$n= 82$ $d= 2$ m $r_0= 1$ m

$L= 42$m $s/d \approx 2,9$

$E_l= 39,3$ MN/m^2 $E_p= 47160$ MN/m^2

$\upsilon_s= 0,2$ $\upsilon_s=0,3$

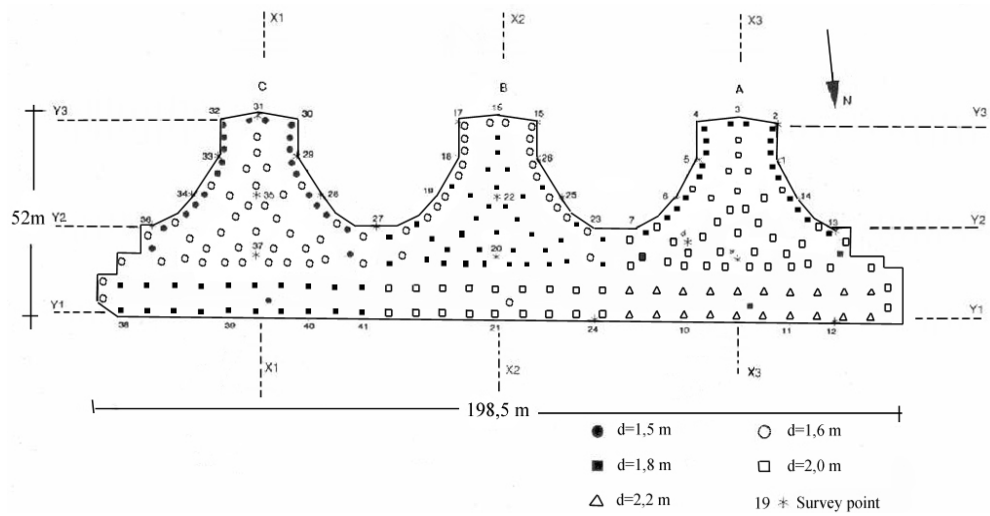

Figure A.35: Layout of the foundation (Mandolini and Viggiani, 1997)

P=567875 KN

$\lambda = E_p/G_l = 47160/39{,}3 \approx 1200$

$\rho = G_{l/2}/G_l = 1 \rightarrow 1{,}06$

$\log\lambda = 3{,}079 \rightarrow 1{,}01$

$s/d = 2{,}9 \rightarrow 1{,}01$

$L/d = 21 \rightarrow 0{,}536$

$\upsilon_s = 0{,}2 \rightarrow 1{,}03$

$\upsilon_s = 0{,}3 \rightarrow 1$

$\eta_w = n^{-e}$ $R_s = n^e$

$\zeta = \ln\{[0{,}25 + (2{,}5\ \rho\ (1-\upsilon) - 0{,}25)\xi]\ L/r_0)\}$ (W. Fleming, et al., 1992)

$\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 0{,}3$

$\mu L = (2/(\lambda\zeta))^{0{,}5} L/r_0$

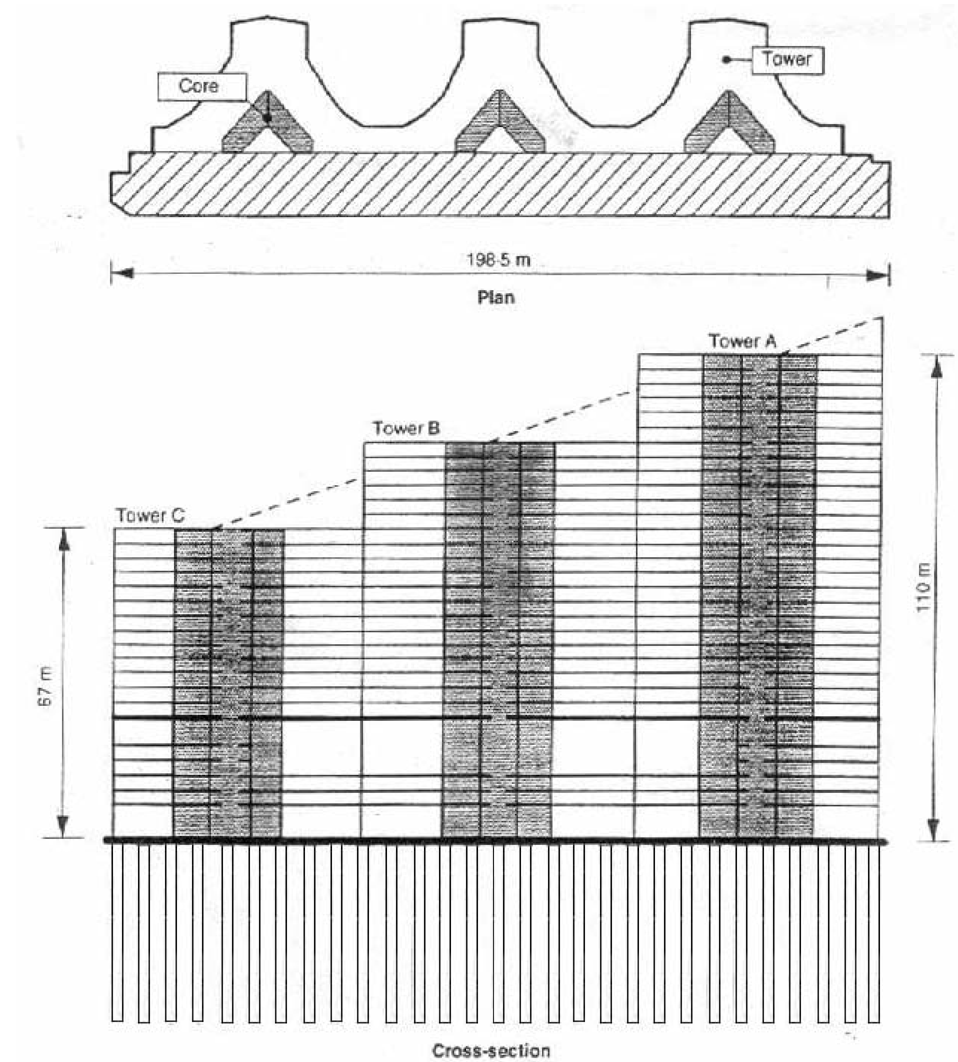

Figure A.36: Schematic plan and section of the structure (Mandolini and Viggiani, 1997)

P_{single}=567875/82=6925,3 KN

	e	η_w	R_s	ζ	μL	tanhμL L/(μL r_0)	$P_t/(w_t G_l r_0)$
υ_s=0,2	0,596	0,072	13,883	3,483	0,918	33,156	66,705
υ_s=0,3	0,579	0,077	12,859	3,381	0,932	32,958	68,834

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0{,}2$	2621,520	15483,980	36,67	2,64	36,66
$\upsilon_s=0{,}3$	2705,194	17250,59	32,92	2,56	32,92

$\delta_{measured}=28{,}1$ mm

b)Equivalent Pier Method

$B=A_G^{\ 0.5}=47{,}21$ m

$A_P=\Pi d^2 n/4=257{,}61\ m^2$

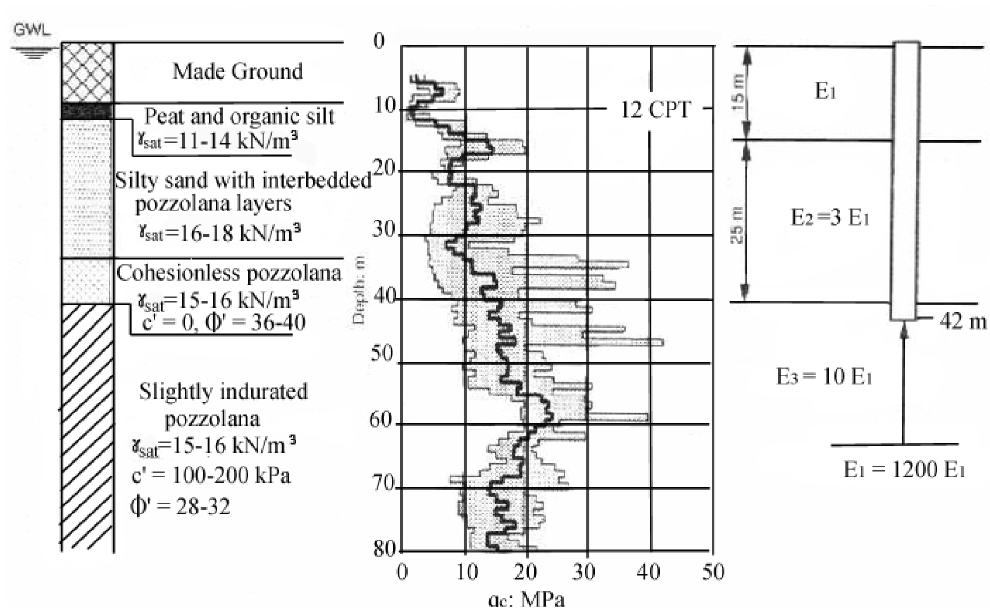

Figure A.37: Subsoil profile and properties, and subsoil model adopted in the analysis (Mandolini and Viggiani, 1997)

E_p=47160 MPa

E_s'=94,32 MPa E_u=117,9 MPa

$d_e=1{,}13\ A_G^{0,5}$=53,34 m (for end bearing piles)

ρ=1   L=42 m $\xi=G_l/G_b$=0,3

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln\{[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0)\}$ (W. Fleming, et al.,1992)

$\zeta_2=\ln\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0)\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,2	5534,335	140,822	0,199	0,420	0,744	0,079	8,99
			1,827	0,138	0,782	0,230	26,02
υ_s=0,3	5541,286	140,999	0,097	0,601	0,704	0,050	5,22
			1,808	0,139	0,782	0,226	23,61

Method 2

L/d_e=42/53,34 =0,78 E_b/E_s=393/117,9=3,33 → I_δ=0,225 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	25,39	23,44

K ≈ 400 (pile stiffness factor)   s/d ≈ 2,9   L/d ≈ 21   B=47,21 m

d_e/B ≈ 0,82 assumed, then d_e ≈ 38,71 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μl	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	5534,335	140,822	0,519	0,358	1,040	0,123	19,24
			1,899	0,187	1,072	0,218	33,95
υ_s=0,3	5541,286	140,999	0,417	0,399	1,030	0,113	16,24
			1,874	0,188	1,072	0,216	31,08

Method 2

L/d_e=42/38,71=1,084 E_b/E_s=393/117,9=3,33 → I_δ=0,21 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	32,66	30,14

$\delta_{measured}$=28,1mm

c) Equivalent Raft Method

L=59,12 m B=51,21 m H=102,4 m

L/B=1,154 D/B=0,806 H/B=2

P=567875 KN

q_n=P/(B*L) = 187,570 KN

E_u=393 Mpa E_s'= 314,4 Mpa

μ_1 = 0,54 μ_0 = 0,92

$\delta_{i\ ave} = \mu_1\mu_0 q_n B/E_u$ =0,92 . 0,54 . 187,570 . 51,21 / 393 =12,14 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0,0029$

$z/B= 1 \quad \sigma_z/q=0,29 \quad \sigma_z=54,395\ kN/m^2$

$D/(LB)^{0,5}=0,750 \rightarrow \mu_d=0,775$

Stiff Clay $\rightarrow \mu_g=0,85$

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g \quad =0,0029 \ .\ 54,395 \ .\ 102,4 \ .\ 0,775 \ .\ 0,85 = 10,51\ mm$

$\delta_T=\delta_i+\delta_c = 22,65\ mm$

$\delta_{measured}=28,1mm$

Table A.19: Measured and computed settlements for New Law Court I (mm)

	Settlement (mm)									
	Set. Ratio	Equivalent Pier				Equivalent Raft (ave.)				Mea.
		d_{e1}		d_{e2}		H=101,1m	H=102,4 m	H=101 m	H=129 m	
		Met1	Met2	Met1	Met2	end-bearing piles		friction piles		
υs=0,2	36,67	8,99 26,02	25,39	19,24 33,95	32,66	22,29	22,65	25,34	26,5	28,10
υs=0,3	32,92	5,22 23,61	23,44	16,24 31,08	30,14	19,82	20,15	21,61	22,75	

New Law I

Rs	36,67	Mea.	28,1
Pier	33,95		
Raft	26,5		

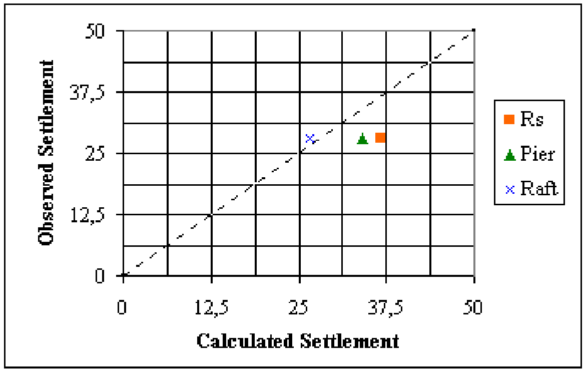

Figure A.38: Measured and computed settlements for New Law Court I (mm)

19. New Law Court Building II, Naples

a) Settlement Ratio Method

$n= 77$ $d= 1,8$ m $r_0= 0,9$ m

$L= 42$ m $s/d\approx 3,375$ m

$E_p= 47160$ MN/m^2

$\upsilon_s= 0,2$ $\upsilon_s=0,3$

$P=449220$ KN

$\lambda=E_p/G_l=47160/39,3\approx 1200$

$\rho=G_{l/2}/G_l=1\rightarrow 1,06$

$\log\lambda=3,079\rightarrow 1,01$

$s/d=3,375\rightarrow 0,98$

$L/d=23,33\rightarrow 0,54$

$\upsilon_s=0,2\rightarrow 1,03$

$\upsilon_u=0,5\rightarrow 0,93$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln\{[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0)\ \}$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=0,3$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

$P_{single}=449220/77=5834,02$ KN

	e	η_w	R_s	ζ	μL	tanhμL L/(μL r_0)	$P_t/(w_t G_l r_0)$
υ_s=0,2	0,583	0,079	12,614	3,588	1,005	35,449	68,073
υ_s=0,3	0,566	0,085	11,717	3,486	1,020	35,217	70,052

	P_t/w_t	$K=n\eta_w k$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,2	2407,75	14696,87	30,56	2,423	30,56
υ_s=0,3	2477,74	16282,97	27,58	2,354	27,58

$\delta_{measured}$=31,5 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$=46,14 m

$A_P=\Pi d^2 n/4$=195,94 m^2

E_p=47160 MPa

E_s'=94,32 MPa E_u=117,9 MPa

$d_e=1{,}13\ A_G^{0,5}$=52,13 m (for end bearing piles)

ρ=1   L=42 m $\xi=G_l/G_b$=0,3

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln\{[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0)\}$ (W. Fleming, et al., 1992)

$\zeta_2=\ln\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0)\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	4426,184	112,625	0,222	0,455	0,754	0,086	7,91
			1,832	0,158	0,798	0,233	21,29
υ_s=0,3	4433,32	112,807	0,120	0,618	0,716	0,059	5,02
			1,812	0,159	0,798	0,229	19,36

Method 2

L/d_e=42/52,13 =0,805 E_b/E_s=393/117,9=3,33 → I_δ=0,225 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	20,55	18,97

K ≈ 400 (pile stiffness factor) s/d ≈ 3,375 L/d ≈ 23,33 B=46,14 m

d_e/B ≈ 0,82 assumed, then d_e ≈ 37,83 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	4426,184	112,625	0,542	0,401	1,054	0,128	16,17
			1,905	0,214	1,093	0,221	27,91
υ_s=0,3	4433,32	112,807	0,441	0,445	1,042	0,118	13,78
			1,880	0,215	1,093	0,220	25,62

Method 2

L/d_e=42/37,83 ≈ 1,11 E_b/E_s=393/117,9=3,33 → I_δ=0,21 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	26,43	24,40

$\delta_{measured}$=31,5 mm

c) Equivalent Raft Method

L=59,12 m   B=51,21 m   H=102,4 m

L/B=1,154   D/B=0,806   H/B=2

P=449220 KN q_n=P/(B*L) = 148,378 KN

E_u=393 Mpa E_s'= 314,4 Mpa

μ_1 = 0,54 μ_0 = 0,92

$\delta_i=\mu_1\mu_0 q_n B/E_u$ =0,92 . 0,54 . 148,378 . 51,21 / 393 =9,60 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$ ≈ 0,0029

z/B= 1 σ_z/q=0,29 σ_z=43,03 kN/m^2

$D/(LB)^{0,5}$=0,750→ μ_d=0,775

Stiff Clay → μ_g=0,85

$\delta_c=m_\upsilon\,\sigma_z\,H\,\mu_d\,\mu_g$ =0,0029 . 43,03 . 102,4 . 0,775 . 0,85 = 8,311 mm

$\delta_T=\delta_i+\delta_c$ = 17,92 mm

$\delta_{measured}$=31,5 mm

Table A.20: Measured and computed settlements for New Law Court II (mm)

	Settlement (mm)									
	Set. Ratio	Equivalent Pier				Equivalent Raft				Mea.
		d_{e1}		d_{e2}		H=101,1m	H=102,4 m	H=101 m	H=129 m	
		Met1	Met2	Met1	Met2	end-bearing piles		friction piles		
υs=0,2	30,56	7,91 21,29	20,55	16,17 27,91	26,43	17,63	17,92	20,05	20,96	31,50
υs=0,3	27,58	5,02 19,36	18,97	13,78 25,62	24,40	15,68	15,94	17,09	18	

New Law II

Rs	30,56	Mea.	31,5
Pier	27,91		
Raft	20,96		

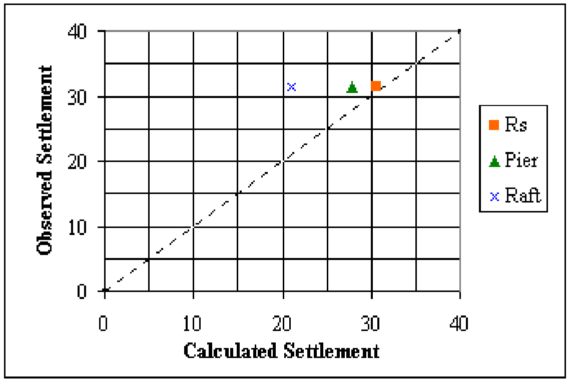

Figure A.39: Measured and computed settlements for New Law Court II (mm)

20. New Law Court Building III, Naples

a) Settlement Ratio Method

n= 82   d= 1,65 m r_0= 0,825 m

L= 42 m   s/d ≈ 3,55 m

E_p= 47160 MN/m^2

υ_s= 0,2 υ_s=0,3

P=409335 KN

$\lambda=E_p/G_l=47160/39,3 \approx 1200$

$\rho=G_{l/2}/G_l=1\rightarrow1,06$

$\log\lambda=3,079\rightarrow1,01$

s/d=3,55→0,97

L/d=25,45→0,545

υ_s=0,2→1,03

υ_u=0,5→0,93

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln\{[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0)\ \}$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=0,3$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=409335/82=4991,9 KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L$ L/(μL r_0)	$P_t/(w_t G_l r_0)$
υ_s=0,2	0,583	0,076	13,051	3,675	1,084	37,319	69,072
υ_s=0,3	0,566	0,082	12,110	3,573	1,099	37,057	70,926

	P_t/w_t	$K=n\eta_w k$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,2	2239,51	14070,39	29,09	2,229	29,09
υ_s=0,3	2299,619	15570,53	26,29	2,170	26,29

$\delta_{measured}$=25,1 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$=47,21 m

$A_P=\Pi d^2 n/4$=175,33 m^2

E_p=47160 MPa

E_s'=94,32 MPa E_u=117,9 MPa

$d_e=1,13\ A_G^{0,5}$=53,34 m (for end bearing piles)

ρ=1   L=42 m $\xi=G_l/G_b$=0,3

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln\{[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0)\}$ (W. Fleming, et al., 1992)

$\zeta_2=\ln\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0)\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L$ $L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	3796,930	96,613	0,199	0,507	0,726	0,083	6,74
			1,827	0,167	0,780	0,236	19,24
υ_s=0,3	3804,172	96,798	0,097	0,725	0,673	0,053	3,995
			1,808	0,168	0,779	0,233	17,52

Method 2

L/d_e=42/53,34 =0,787 E_b/E_s=393/117,9=3,33 → I_δ=0,225 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	18,30	16,89

K ≈ 400 (pile stiffness factor) s/d ≈ 3,55 L/d ≈ 25,45 B=47,21 m

d_e/B ≈ 0,82 assumed, then d_e ≈ 38,71 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L$ $L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	3796,930	96,613	0,519	0,433	1,021	0,129	14,46
			1,899	0,226	1,066	0,226	25,33
υ_s=0,3	3804,172	96,798	0,418	0,482	1,008	0,118	12,28
			1,874	0,227	1,066	0,225	23,28

Method 2

L/d_e=42/38,71 ≈ 1,08 E_b/E_s=393/117,9=3,33 → I_s=0,21 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	23,54	21,73

$\delta_{measured}$=25,1mm

c) Equivalent Raft Method

L=59,12 m   B=51,21 m   H=102,4 m

L/B=1,154   D/B=0,806   H/B=2

P=409335 KN q_n=P/(B*L) = 135,204 KN

E_u=393 Mpa E_s'= 314,4 Mpa

μ_1 = 0,54 μ_0 = 0,92

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$ =0,92 . 0,54 . 135,204 . 51,21 / 393 =8,75 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$ ≈ 0,0029

z/B= 1 σ_z/q=0,29 σ_z=39,209 kN/m^2

$D/(LB)^{0,5}$=0,750→ μ_d=0,775

Stiff Clay → μ_g=0,85

$\delta_c=m_\upsilon\,\sigma_z\,H\,\mu_d\,\mu_g$ =0,0029 . 39,209 . 102,4 . 0,775 . 0,85 = 7,57 mm

$\delta_T=\delta_i+\delta_c$ = 16,33 mm

$\delta_{measured}$=25,1 mm

Table A.21: Measured and computed settlements for New Law Court III (mm)

	Settlement (mm)									
	Set. Ratio	Equivalent Pier				Equivalent Raft				Mea.
		d_{e1}		d_{e2}		H=101,1m	H=102,4 m	H=101 m	H=129 m	
		Met1	Met2	Met1	Met2	end-bearing piles		friction piles		
υs=0,2	29,09	6,74 19,24	18,30	14,46 25,33	23,54	16,07	16,33	18,27	19,1	25,1
υs=0,3	26,29	3,99 17,52	16,89	12,28 23,28	21,73	14,29	14,52	15,58	16,4	

New Law III

Rs	29,09	Mea.	25,1
Pier	25,33		
Raft	19,10		

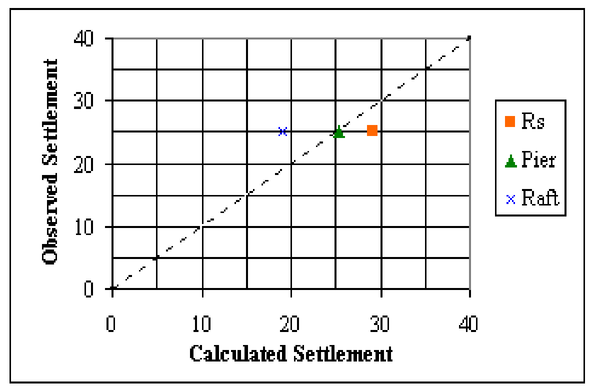

Figure A.40: Measured and computed settlements for New Law Court III (mm)

21. Congress Centre (n=98,43)

The Congress Centre Messe Frankfurt built in 1995-97 comprises a hotel with 13 storeys, a congress hall, and an office building with 14 storeys next to the hotel. This building complex is situated close to the Messeturm tower in the same subsoil conditions. The raft of the Congress Centre has a thickness of 0.8-2.7 m, and is founded 8 m below street level in the Frankfurt Clay. The raft of 10200 m^2 plan area is supported by 141 bored piles, which are concentrated under the highly loaded parts of the raft to minimise differential settlements. The length and spacing of the 1.3 m diameter piles varies according to the applied load distribution. (Katzenbach, R., Arslan,U., and Moormann, C., 2000)

Solution for Hotel

a) Settlement Ratio Method

n= 98   d= 1,3 m r_0= 0,65 m

L= 28 m   s= 5,85 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,1 υ_s=0,3   Frankfurt Clay

P=1251 MN

$\lambda=E_p/G_l=30000/48 \approx 625$

$\rho=G_{l/2}/G_l=0,708 \rightarrow 0,98$

$\log\lambda=2,79 \rightarrow 0,95$

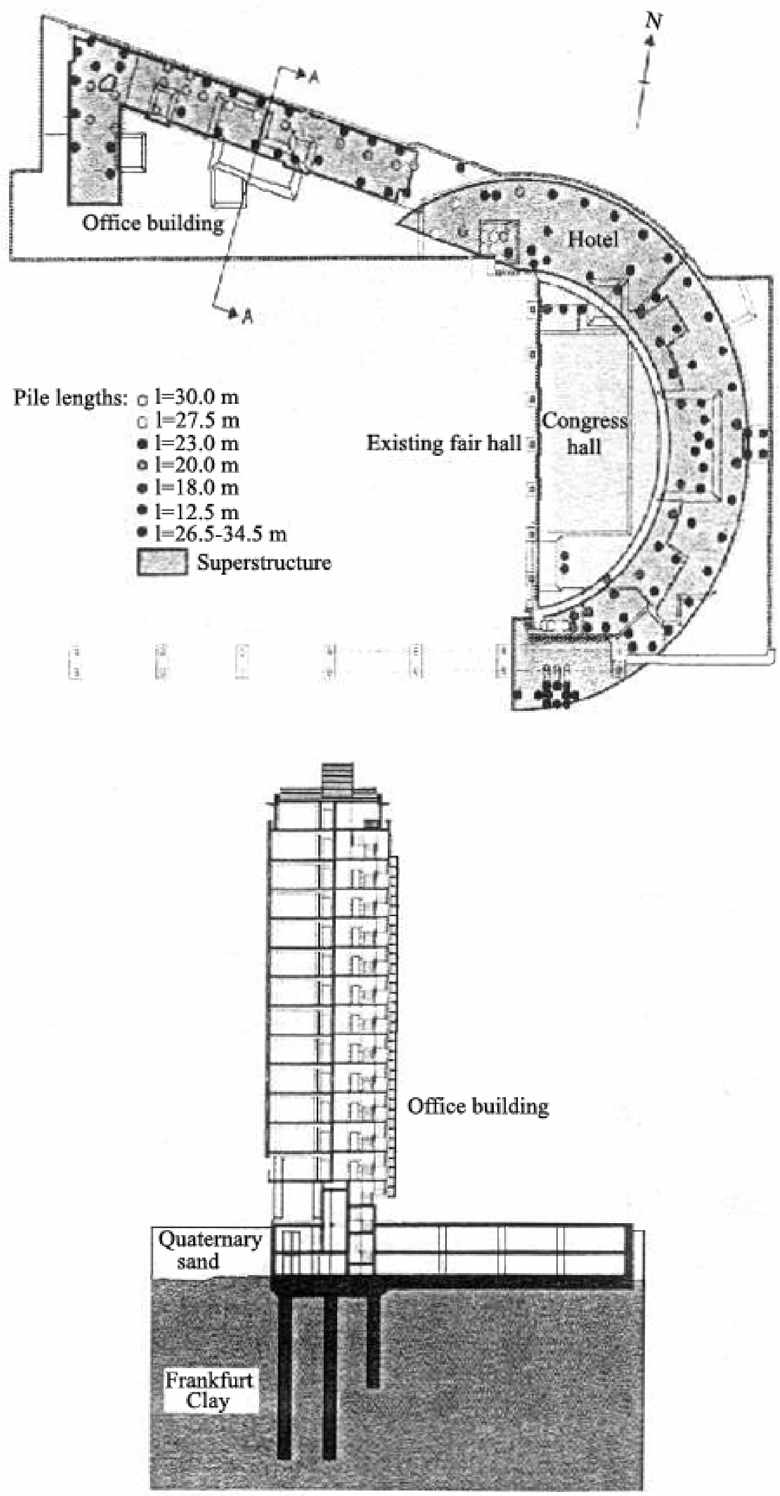

Figure A.41: Congress Centre Messe Frankfurt, ground plan and section A-A (Katzenbach et al, 2000)

$s/d=4,5 \rightarrow 0,9$

$L/d=21,53 \rightarrow 0,531$

$\upsilon_s=0,1 \rightarrow 1,05$

$\upsilon_s=0,3 \rightarrow 1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single}=1251000/98=12765,3$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0,1$	0,472	0,114	8,724	4,229	1,185	30,137	1056,153
$\upsilon_s=0,3$	0,449	0,127	7,869	3,978	1,221	29,622	35,773

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,1$	1056,153	11863,1	105,45	12,08	105,45
$\upsilon_s=0,3$	1116,140	13899,15	90,00	11,43	90,00

$\delta_{measured}=50$ mm

b)Equivalent Pier Method

$B=A_G^{\ 0.5}=79,19$ m

$A_P=\Pi d^2 n/4=130,078\ m^2$

E_p=30000 MPa

E_s'=105,6 MPa E_u=144 MPa

$d_e=1,27\ A_G^{0,5}=100,57$ (for friction piles)

ρ=0,708   L=28 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	725,593	15,116	-0,119	—	—	—	—
			1,772	0,151	0,276	0,793	93,465
υ_s=0,3	744,394	15,508	-0,370	—	—	—	—
			1,738	0,151	0,276	0,776	77,411

Method 2

$L/d_e=28/100,57=0,278 \rightarrow I_\delta=0,5$ (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	58,89	49,83

K ≈ 200 (pile stiffness factor)   s/d ≈ 4,5   L/d ≈ 21,53   B=79,19 m

$d_e/B \approx 0,75$ assumed, then $d_e \approx 59,39$ m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	725,593	15,116	0,407	0,537	0,430	0,343	68,442
			1,872	0,250	0,461	0,719	143,59
υ_s=0,3	744,394	15,508	0,155	0,857	0,382	0,205	34,73
			1,819	0,251	0,461	0,722	121,98

Method 2

L/d_e=28/59,39=0,471 → I_δ=0,46 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	99,72	84,38

$\delta_{measured}$=50 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
121,3	65,3	42,815	1,85	0,65	0,45
142,71	86,707	42,815	1,64	0,49	0,83

P=1251000 KN υ_s= 0,1

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,93	0,24	188,325	157,937	12,22
0,92	0,18	316,775	101,101	4,58

$\delta_{i\ ave}$ = 16,80 mm

m_υ=[(1+υ)(1-2υ)]/[E_s'(1-υ)]

$D/(LB)^{0,5}$=0,329 → μ_d=0,908

Frankfurt Clay → μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
138,105	0,00708	112,451	21,66
232,301	0,00421	55,277	6,33

δ_c= 27,99 mm

$\delta_T = \delta_{i\ ave} + \delta_c$ = 44,80 mm

$\delta_{measured}$=50 mm

Solution for Office Building

a) Settlement Ratio Method

n= 43   d= 1,3 m r_0= 0,65 m

L= 28 m   s= 5,85 m

G= 20+1,0z (MN/m^2) E_p= 30000 MN/m^2

υ_s= 0,1 υ_s=0,3   Frankfurt Clay

P=549 MN

$\lambda = E_p/G_l$=30000/48 ≈ 625

$\rho = G_{l/2}/G_l$=0,708→0,98

logλ=2,79→0,95

s/d=4,5→0,9

$L/d=21,53 \rightarrow 0,531$

$\upsilon_s=0,1 \rightarrow 1,05$

$\upsilon_s=0,3 \rightarrow 1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single}=549000/43=12767,44$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0,1$	0,472	0,169	5,911	4,229	1,185	30,137	33,851
$\upsilon_s=0,3$	0,449	0,184	5,432	3,978	1,221	29,622	35,773

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,1$	1056,153	7681,805	71,46	12,088	71,46
$\upsilon_s=0,3$	1116,140	8834,956	62,14	11,439	62,14

$\delta_{measured}=45$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=53,68$ m

$A_P=\Pi d^2 n/4=57,07\ m^2$

$E_s'=105,6$ MPa $E_u=144$ MPa

E_p=30000 MPa

d_e=1,27 $A_G^{0,5}$=68,17 (for friction piles)

ρ=0,708   L=28 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1995)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,1	697,717	14,535	0,269	0,587	0,369	0,283	21,60
			1,842	0,224	0,404	0,742	56,58
υ_s=0,3	716,536	14,927	0,018	2,237	0,179	0,057	3,720
			1,794	0,224	0,404	0,740	47,75

Method 2

L/d_e=28/68,17 =0,410 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	38,13	32,26

K ≈ 200 (pile stiffness factor)   s/d ≈ 4,5   L/d ≈ 21,53   B=53,68 m

d_e/B ≈ 0,75 assumed, then d_e ≈ 40,26 m   (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	697,717	14,535	0,796	0,578	0,627	0,431	55,65
			1,976	0,367	0,665	0,667	86,22
υ_s=0,3	716,536	14,927	0,544	0,689	0,602	0,383	41,87
			1,905	0,368	0,665	0,684	74,84

Method 2

L/d_e=28/40,26=0,695 → I_δ=0,48   (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	61,98	52,44

$\delta_{measured}$=45 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
90,7	44,7	42,815	2,03	0,95	0,65
112,1	66,1	42,815	1,69	0,64	1,09

P=549000 KN υ_s= 0,1

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,93	0,35	188,325	135,41	10,46
0,92	0,23	316,775	74,07	3,27

$\delta_{i\ ave}$ = 13,73 mm

m_υ=[(1+υ)(1-2υ)]/[E_s'(1-υ)]

$D/(LB)^{0,5}$=0,461 → μ_d=0,86

Frankfurt Clay → μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
138,105	0,00708	85,309	15,56
232,301	0,00421	30,738	3,33

δ_c= 18,90 mm

$\delta_T = \delta_{i\ ave} + \delta_c$ = 32,63 mm

$\delta_{measured}$=45 mm

Table A.22: Measured and computed settlements for Congress Centre (mm)

	Settlement (mm)										
	Set. Ratio		Equivalent Pier (Hotel)				Equivalent Pier (Office B.)				Mea.
	Hotel	Office Build.	d_{e1}		d_{e2}		d_{e1}		d_{e2}		
			Met1	Met2	Met1	Met2	Met1	Met2	Met1	Met2	
υs=0,1	105,45	71,47	—	58,89	68,42	99,72	21,60	38,13	55,65	61,98	40-60
			93,46		143,59		56,58		86,22		
υs=0,3	90,00	62,14	—	49,83	34,73	84,38	3,72	32,26	41,87	52,44	
			77,41		121,98		47,75		74,84		

	Equivalent Raft							
	Hotel				Office Building			
	H=85,63 m	H=76,3 m (at the pile tip)	H=85,63 m (1/6)	H=85,63 m (1/8)	H=85,63 m	H=76,3 m	H=85,63 m (1/6)	H=85,63 m (1/8)
	Ave.	Ave.	Ave.	Ave.	Ave.	Ave.	Ave.	Ave.
υs=0,1	44,8	43,93	47,31	48,85	32,63	33,52	34,34	35,38
υs=0,3	34,8	34,24	36,57	37,83	25,88	26,89	27,25	28,01

Congress Centre - Hotel

Rs	105,45	Mea.	50
Pier	93,69		
	58,89		
	68,42		
Raft	44,8		

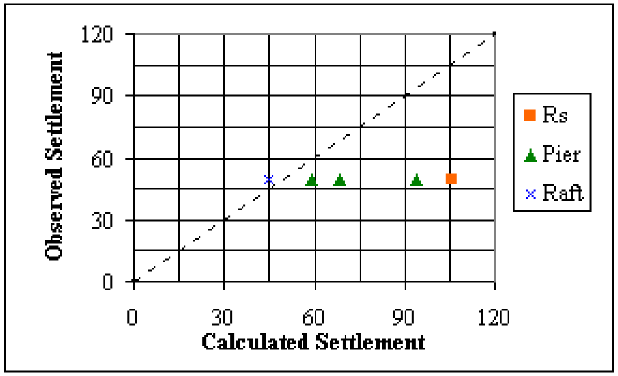

Figure A.42: Measured and computed settlements for Congress Centre Hotel (mm)

Congress Centre - Office Building

Rs	71,46	Mea.	45
Pier	56,58		
	38,13		
	55,65		
Raft	32,63		

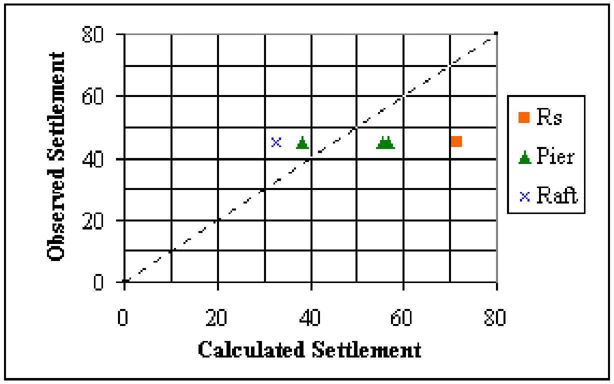

Figure A.43: Measured and computed settlements for Congress Centre Office Building (mm)

22. Commerz Bank (n=111)

The tower stands as the highest office structure in Europe and ranks 24th tallest in the world. The building plan is a rounded equilateral triangle 60 meters wide. Three 52*131-ft-wide sections join to form the triangular structure with a central atrium. The mat was placed in a 7.5 meter deep excavation and varies from 2.5 to 4.5 meters in thickness. There are 111 piles concentrated in clusters under each of the Tower's columns. A foundation on 111 piles of up to 48.5 m in length and up to 1.8 m in diameter driven into the lower rock was chosen. This rock lies about 30 m beneath the Frankfurt clay. Calculated range of settlement is 60-70 mm using different methods. (Katzenbach, R. et al, 2000, Poulos, H.G., 2000)

a) Settlement Ratio Method

n= 111 d= 1,66 m r_0= 0,83 m

L= 45 m s= 4,5 m E_p= 40000 MN/m^2

G= 20+1,0z (MN/m^2) Frankfurt Clay

E_u=20000 MN/m^2 Frankfurt Limestone

υ_s= 0,1 υ_s=0,3

P=1300 MN

$\lambda=E_p/G_l=40000/49 \approx 816{,}326$

$\rho=G_{l/2}/G_l=0{,}704 \rightarrow 0{,}98$

$\log\lambda=2{,}911 \rightarrow 0{,}97$

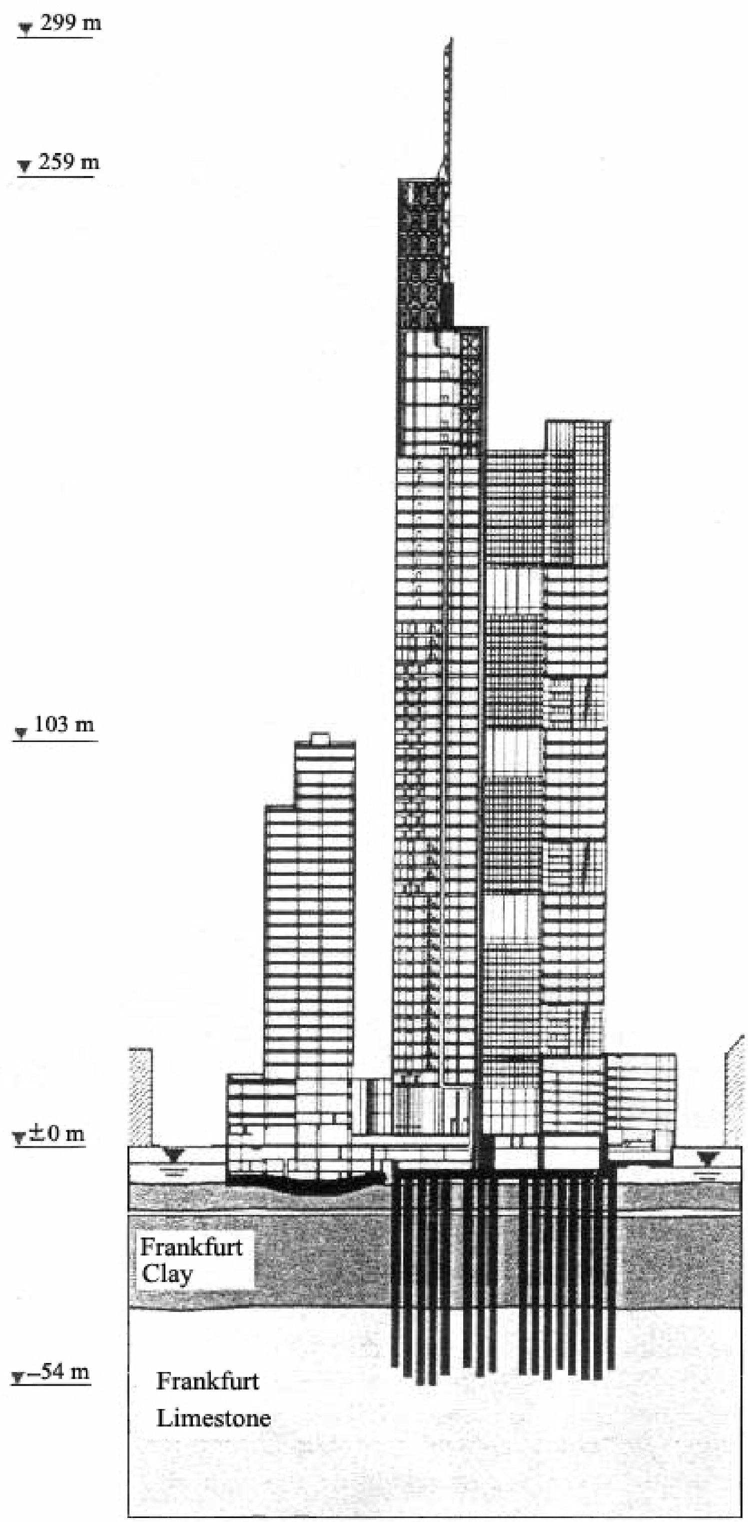

Figure A.44: Sectional elevation of new Commerzbank Tower (Katzenbach et al, 2000, Poulos, 2000)

$s/d=4,5 \rightarrow 0,9$

$L/d=27,108 \rightarrow 0,547$

$\upsilon_s=0,1 \rightarrow 1,05$

$\upsilon_s=0,3 \rightarrow 1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=0,00735$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single}=1300000/111=11711,71$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0,1$	0,419	0,098	10,116	2,645	1,650	30,520	80,003
$\upsilon_s=0,3$	0,467	0,110	9,060	2,635	1,653	30,477	80,926

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,1$	3253,725	35700,51	36,41	3,599	36,41
$\upsilon_s=0,3$	3291,282	40319,68	32,24	3,558	32,24

$\delta_{measured}=15\text{-}19$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=46,37$ m

$A_P=\Pi d^2 n/4=240,231\ m^2$

E_p=40000 MPa

E_s'=107,8 Mpa E_u=147 MPa

$d_e=1,13\ A_G^{0,5}=52,398$ (for end-bearing piles)

ρ=0,704 L=45 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,1	4564,80	93,159	-0,806	————			
			1,694	0,193	0,848	0,032	7,489
υ_s=0,3	4582,21	93,514	-0,816	————			
			1,694	0,192	0,848	0,036	7,11

Method 2

L/d_e=45/52,398 =0,858 → I_δ=0,023 (Fig. 2.10.)

	υ_s=0,1	υ_s=0,3
δ (mm)	5,29	4,48

K ≈ 270 (pile stiffness factor) s/d ≈ 4,5 L/d ≈ 27,108 B=46,37 m

d_e/B ≈ 0,78 assumed, then d_e ≈ 36,16 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	4564,80	93,159	-0,436	———			
			1,731	0,277	1,213	0,043	14,44
υ_s=0,3	4582,21	93,514	-0,446	———			
			1,729	0,276	1,213	0,049	13,89

Method 2

L/d_e=45/36,16=1,244 → I_δ=0,0328

	υ_s=0,1	υ_s=0,3
δ (mm)	9,33	7,89

$\delta_{measured}$=15-19 mm

c) Equivalent Raft Method

δ=%0,01-0,05 B (M. J. Tomlinson, 1986)

For B=46,37

δ=4,637-23,185 (ave=13,911)

For B=39,48

δ=3,948-19,74 (ave=11,844)

$\delta_{measured}$=15-19 mm

Table A.23: Measured and computed settlements for Commerz Bank (mm)

	Settlement (mm)											
	Set. Ratio	Equivalent Pier								Equivalent Raft		Mea.
		B*L=2150 m2				B*L=1558 m2				B=46,37 m	B=39,48 m	
		d_{e1}		d_{e2}		d_{e1}		d_{e2}		Ave	Ave	
		Met1	Met2	Met1	Met2	Met1	Met2	Met1	Met2			
υs=0,1	36,41	7,49	5,29	14,44	9,33	7,81	6,75	14,91	11,74	13,91	11,84	17
υs=0,3	32,24	7,11	4,48	13,89	7,89	7,37	5,71	14,29	9,94	(4,63 - 23,18)	(3,94 - 19,74)	(15-19)

Commerz Bank

Rs 36,41 Mea. 17

Pier 14,44

Raft 13,91

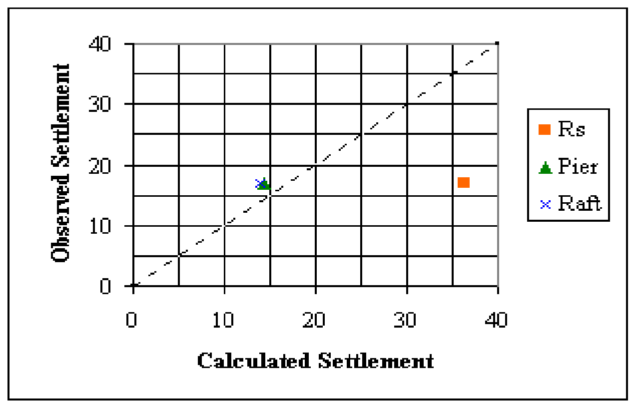

Figure A.45: Measured and computed settlements for Commerz Bank (mm)

23. Main Tower (n=112)

The new Main Tower skyscraper with five basement levels, ground floor and a further 57 storeys above grade, will rise to a height of 198 m. The raft is founded at the considerable depth of 21 m below street level, which is 14 m below groundwater level. The entire excavation for the Main Tower building has a plan area 50m*85m and fully equipped with a five-storey parking basement. The Main Tower core shaft has dimensions of 30m*50m in plan, and arranged asymmetrically with respect to tha basement. The total load of the Main Tower building is about 2000 MN. The raft has a plan area 3800 m^2 with a thickness of 3,8 m in the centre, and 3,0 m in the remaining area The piled raft incorporates 112 large-diameter bored piles and a secant bored pile wall, which is connected to the raft. The pile have a diameter of 1.5 m and a length of 30 m, except for some 20 m long piles near the edge of the raft. The bases of the 30 m piles are situated 5-8 m above the upperboundary of the Frankfurt Limestone. (Katzenbach, R., Arslan,U., and Moormann, C., 2000)

a) Settlement Ratio Method

n= 112 d= 1,5 m r_0= 0,75 m

L= 30 m s= 4,5 m

G= 20+1,0z (MN/m^2) Frankfurt Clay E_p= 35000 MN/m^2

E_u=20000 MN/m^2 Frankfurt Limestone

P=2000 MN υ_s= 0,1 υ_s=0,3

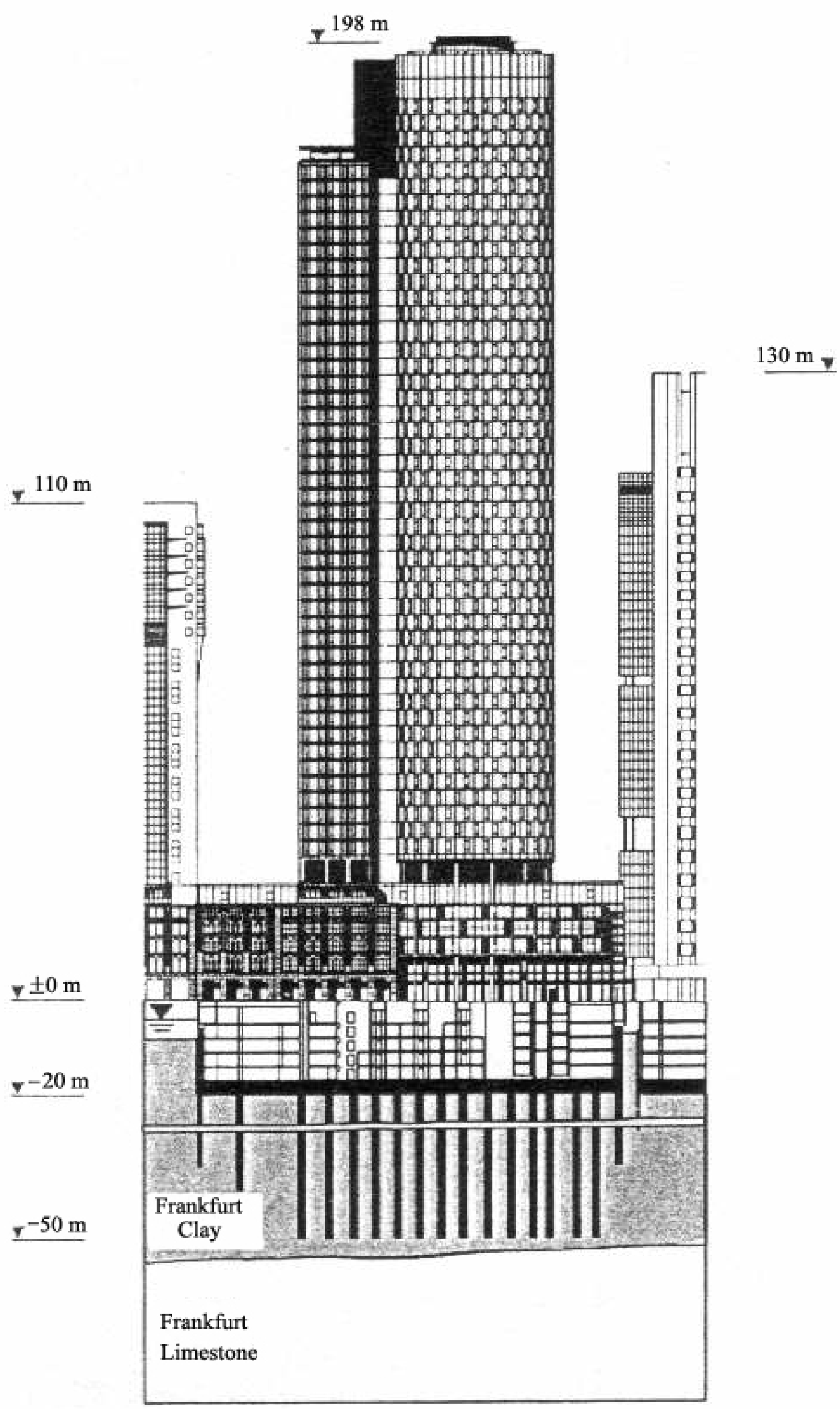

Figure A.46: Sectional elevation of Main Tower building (Katzenbach et al, 2000)

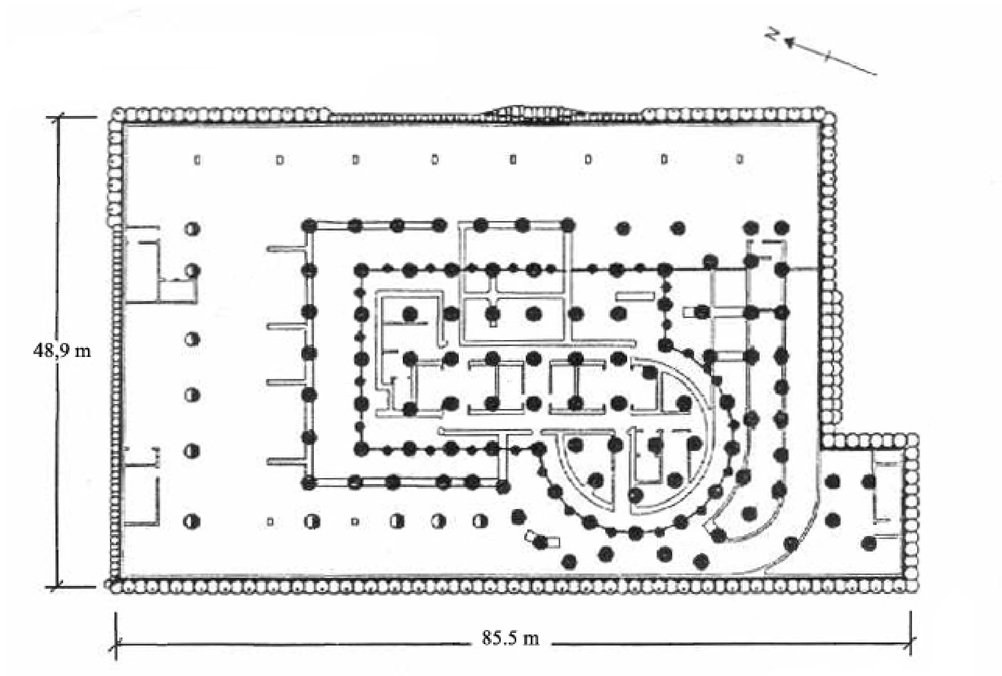

Figure A.47: Plan of piled raft foundation for Main Tower building (Katzenbach et al, 2000)

$\lambda = E_p/G_l = 35000/61 \approx 573{,}770$

$\rho = G_{l/2}/G_l = 0{,}754 \rightarrow 1$

$\log\lambda = 2{,}758 \rightarrow 0{,}94$

$s/d = 4{,}5 \rightarrow 0{,}9$

$L/d = 20 \rightarrow 0{,}536$

$\upsilon_s = 0{,}1 \rightarrow 1{,}05$

$\upsilon_s = 0{,}3 \rightarrow 1$

$\eta_w = n^{-e}$ $R_s = n^e$ $\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 0{,}00915$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\mu L = (2/(\lambda\zeta))^{0{,}5} L/r_0$

$P_{single} = 2000000/112 = 17857{,}142$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,476	0,105	9,455	2,354	1,539	23,700	72,216
υ_s=0,3	0,453	0,117	8,496	2,341	1,543	23,653	73,127

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	3303,915	39134,09	51,10	5,404	51,10
υ_s=0,3	3345,559	44101,82	45,34	5,337	45,34

$\delta_{mea.}$=20 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =52,32 m

$A_P=\Pi d^2 n/4$=197,92 m^2

E_p=35000 MPa

E_s'=134,2 Mpa E_u=183 MPa

$d_e=1,13\ A_G^{0,5}$=59,12 (for end-bearing piles)

ρ=0,754   L=30 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	2654,53	43,516	-1,32	—	—	—	—
			1,661	0,168	0,502	0,041	10,37
υ_s=0,3	2677,16	43,887	-1,33	—	—	—	—
			1,660	0,168	0,502	0,046	9,81

Method 2

L/d_e=30/59,12 =0,507 → I_δ=0,018 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	4,53	3,83

K ≈ 190 (pile stiffness factor) s/d ≈ 4,5 L/d ≈ 20 B=53,32 m

d_e/B ≈ 0,78 assumed, then d_e ≈ 40,81 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	2654,53	43,516	-0,949	—	—	—	—
			1,683	0,242	0,721	0,055	20,08
υ_s=0,3	2677,16	43,887	-0,962	—	—	—	—
			1,683	0,241	0,721	0,062	19,25

Method 2

L/d_e=30/40,81=0,735 → I_δ=0,02 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	7,30	6,17

$\delta_{mea.}$=20 mm

c) Equivalent Raft Method

L=74 B=37 L/B=2

H=6,5 D=51 D/B=1,37 H/B=0,175

E_{uave}=192,75 E_s'=141,35

μ_0→ 0,91 μ_1→0,04

q_n=2000000/(BL) = 730,46 KPa

$\delta_{iave}=q_n B \mu_0 \mu_1 / E_u$ = 5,10 mm

$D/(LB)^{0,5}$=0,974→ μ_d=0,732

Frankfurt Clay→ μ_g=0,7

z/B=0,087 σ_z/q=0,93 σ_z=679,328 KPa

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0{,}00701$

$\delta_c = m_\upsilon \sigma_z H \mu_d \mu_g$ = 0,00701 . 679,328. 6,5 . 0,732 . 0,7 ≈ 15,87 mm

$\delta_{Taverage}$ = 20,98 mm

$\delta_{mea.}$=20 mm

Table A.24: Measured and computed settlements for Main Tower (mm)

	Settlement (mm)											
	Set. Ratio	**Equivalent Pier**								**Equivalent Raft**		**Mea.**
		B*L(37*74)				B*L(43*80)				B*L (43*80)	**B*L (37*74)**	
		d_{e1}		d_{e2}		d_{e1}		d_{e2}				
		Met1	Met2	Met1	Met2	Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,1	51,10	10,37	4,53	20,08	7,30	10,03	4,04	19,53	6,51	17,22	20,98	20 (ave)
υs=0,3	45,35	9,81	3,83	19,25	6,18	9,51	3,42	18,74	5,51	12,22	15,16	25 (Max)

Main Tower

Rs 51,1 Mea. 20

Pier 20,08
19,53

Raft 20,98

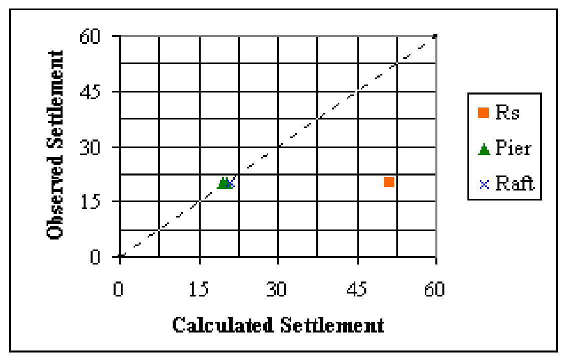

Figure A.48: Measured and computed settlements for Main Tower (mm)

24.Cambridge Road (n=116)

This is one of three 23-storey blocks of maisonettes constructed for the G.L.C. at a site in London Borough of Waltham Forest. One side of the structure is connected to a semi-basement car park.

The structure was founded on 0,62 m diameter straight shafted piles taken to depths of 15 m. The site investigation showed 3 of Toplow Gravel overlying London Clay. (Morton, K., and Au, E., 1974)

a) Settlement Ratio Method

n= 116 d= 0,62 m r_0= 0,31 m

L= 15,3 m s= 4,5 m

c_u= 70-400 (kN/m^2) London Clay

N=23-94 Taplow Gravel

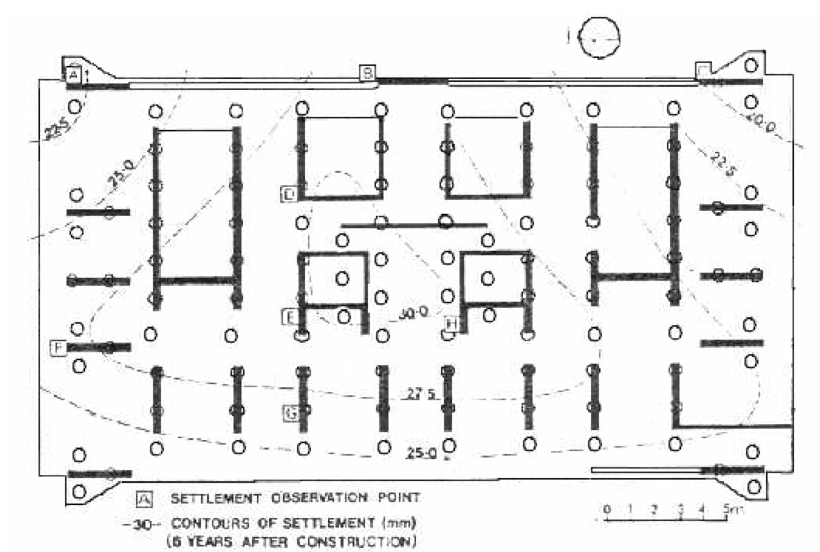

Figure A.49: Cambridge Road foundation plan (Morton and Au, 1974)

E_p= 25000 MN/m^2

P=122 MN υ_s= 0,1 υ_s=0,3

$\lambda=E_p/G_l=25000/29,1 \approx 859,106$

$\rho=G_{l/2}/G_l=23,3/29,1=0,8\rightarrow1,02$

$\log\lambda=2,934\rightarrow0,97$

s/d=4,5→0,9

L/d=24,67→0,545

$\upsilon_s=0,1\rightarrow1,05$

$\upsilon_s=0,3\rightarrow1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

P_{single}=122000/116=1051,72 KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,1	0,509	0,088	11,27	4,400	1,124	35,519	41,817
υ_s=0,3	0,485	0,099	10,04	4,236	1,157	34,982	43,998

	P_t/w_t	$K=n\eta_w k$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	377,232	3882,309	31,42	2,787	31,42
υ_s=0,3	396,909	4584,23	26,61	2,649	26,61

$\delta_{measured}$=27,5 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=21{,}908$ m

$A_P=\Pi d^2 n/4=35{,}02\ m^2$

$E_p=25000$ MPa

$E_s'=64{,}02$ MPa $\quad E_u=87{,}3$ MPa

$d_e=1{,}27\ A_G^{0{,}5}=27{,}82$ (for friction piles)

$\rho=0{,}8 \quad L=15{,}3$ m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}1$	1883,37	64,72	0,663	0,233	0,540	0,363	24,89
			1,943	0,138	0,548	0,619	42,42
$\upsilon_s=0{,}3$	1894,16	65,09	0,432	0,293	0,534	0,294	17,09
			1,878	0,140	0,546	0,620	35,94

Method 2

$L/d_e=15{,}3/28{,}28=0{,}549 \rightarrow I_\delta=0{,}5$ (Fig. 2.10)

	$\upsilon_s=0{,}1$	$\upsilon_s=0{,}3$
δ (mm)	34,24	28,97

$K \approx 280$ (pile stiffness factor) $s/d \approx 4{,}5$ $L/d \approx 24{,}67$ $B=21{,}908$ m

$d_e/B \approx 0{,}78$ assumed, then $d_e \approx 17{,}37$ m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	$\tan\mu L\, L/(\mu L\, d_e)$	I_δ	δ
$\upsilon_s=0{,}1$	1883,37	64,72	1,171	0,290	0,870	0,383	42,70
			2,107	0,216	0,881	0,528	58,88
$\upsilon_s=0{,}3$	1894,16	65,09	0,919	0,327	0,864	0,359	33,90
			2,016	0,221	0,881	0,539	50,91

Method 2

$L/d_e=15{,}3/17{,}37=0{,}880 \rightarrow I_\delta=0{,}43$

	$\upsilon_s=0{,}1$	$\upsilon_s=0{,}3$
δ (mm)	47,95	40,57

$\delta_{measured}=27{,}5$ mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
35,1	21,1	21,1	1,66	1	0,48
45,65	31,65	21,1	1,44	0,66	0,98

P=122000 KN $\upsilon_s= 0{,}1$

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_0	μ_1	E_{uave}	q	δi
0,93	0,36	105	164,73	11,08
0,92	0,24	191,79	84,44	3,07

$\delta_{i\ ave}$= 14,16 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}=0{,}374 \rightarrow \mu_d=0{,}9$

London Clay $\rightarrow \mu_g=0{,}7$

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

E_{mid-dr}	m_v	σ_z	δ_c
77	0,01269	97,190	16,40
140,646	0,00695	31,298	2,89

δ_c= 19,29 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 33,45 mm

$\delta_{mea.}$=27,5 mm

Table A.25: Measured and computed settlements for Cambridge Road (mm)

<table>
<tr><th></th><th colspan="11">Settlement (mm)</th></tr>
<tr><th rowspan="3"></th><th rowspan="3">Set. Ratio</th><th colspan="4">Equivalent Pier</th><th colspan="5">Equivalent Raft</th><th rowspan="3">Mea.</th></tr>
<tr><th colspan="2">d_{e1}</th><th colspan="2">d_{e2}</th><th rowspan="2">H=32 m</th><th rowspan="2">H=42,2 m</th><th rowspan="2">H=32 m (at the tip)</th><th rowspan="2">H=38,8 m (1/6)</th><th rowspan="2">H=37,1 m (1/8)</th></tr>
<tr><th>Met1</th><th>Met2</th><th>Met1</th><th>Met2</th></tr>
<tr><td rowspan="2">υs=0,1</td><td rowspan="2">31,42</td><td>24,89</td><td rowspan="2">34,24</td><td>42,70</td><td rowspan="2">47,95</td><td rowspan="2">26,15</td><td rowspan="2">33,46</td><td rowspan="2">30,43</td><td rowspan="2">35,68</td><td rowspan="2">37,3</td><td rowspan="4">27,5</td></tr>
<tr><td>42,42</td><td>58,88</td></tr>
<tr><td rowspan="2">υs=0,3</td><td rowspan="2">26,61</td><td>17,09</td><td rowspan="2">28,97</td><td>33,90</td><td rowspan="2">40,57</td><td rowspan="2">20,99</td><td rowspan="2">26,56</td><td rowspan="2">24,96</td><td rowspan="2">28,35</td><td rowspan="2">29,62</td></tr>
<tr><td>35,93</td><td>50,91</td></tr>
</table>

Cambridge

Rs	31,42	Mea.	27,5
Pier	24,89		
	34,24		
Raft	33,46		

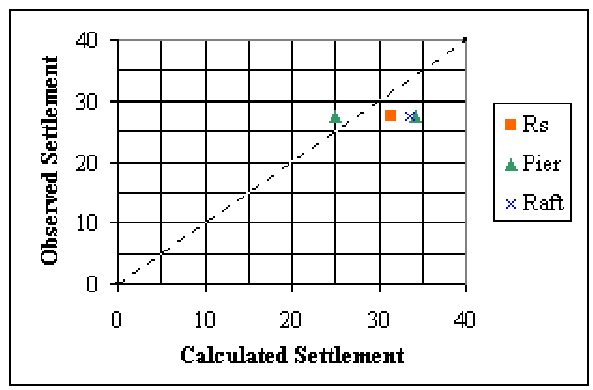

Figure A.50: Measured and computed settlements for Cambridge Road (mm)

25. 19-Storey Reinforced Concrete Building (n=132)

The building was constructed in the USA in the period 1967 to 1970; the overall dimensions in plan area 34 m * 24 m. It is founded on 132 permanently cased driven piles with expanded base with a length of 7.6 m, a shaft diameter of 0.41 m and a base diameter of 0.76 m. The subsoil consists essentially of cohesionless soils, with a layer of highly compressible organic silt between depths of 3 and 7 below the ground surface. In this case, the LE (Randolph (1994)) and NL (GRUPPALO) analyses grossly underestimate the actual values of the settlement. 25.1 mm (LE) and 27.8 mm (NL) settlement predictions are obtained. (Mandolini, A., and Viggiani, C., 1997, Randolph, M.F., and Guo, W.D., 1999)

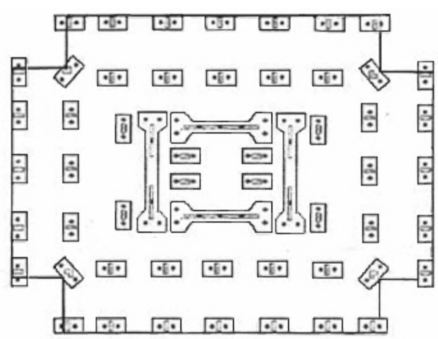

Figure A.51: Layout of the foundations of the building; overall dimensions are 33.6 m * 24.4 m (Mandolini and Viggiani, 1997, Randolph and Guo, 1999)

a) Settlement Ratio Method

n= 132 r_b= 0,38 m r_0= 0,205 m

L= 7,6 m   s ≈ 2,5 m

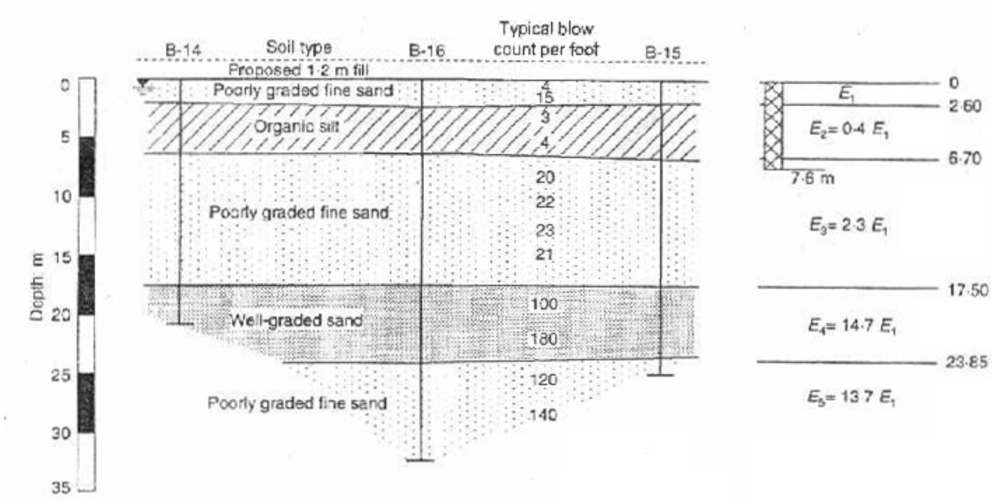

Figure A.52: Typical soil profile and properties at the building site; the subsoil model adopted in the analysis is shown on the right-hand side (Mandolini and Viggiani, 1997, Randolph and Guo, 1999)

E_1=21 MN/m^2 E_p= 25000 MN/m^2

υ_s= 0,3 υ_s=0,4

P=158,4 MN

$\lambda=E_p/G_l=25000/16{,}1 \approx 1552{,}795$

$\rho=G_{l/2}/G_l=0{,}372\rightarrow0{,}92$

$\log\lambda=3{,}191\rightarrow1{,}03$

s/d=6→0,825

L/d=18,53→0,532

$\upsilon_s=0{,}3\rightarrow1$

$\upsilon_s=0{,}4\rightarrow0{,}97$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming et al., 1992)

$\eta = r_b/r_0 = 1{,}853$ $\xi = G_l/G_b = 1$

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single} = 158400/132 = 1200$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0,3$	0,415	0,131	7,619	3,185	0,745	31,451	31,556
$\upsilon_s=0,4$	0,403	0,139	7,169	3,031	0,764	31,219	33,800

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,3$	104,153	1804,255	87,79	11,52	87,79
$\upsilon_s=0,4$	111,559	2053,94	77,12	10,75	77,12

$\delta_{measured} = 64$ mm

b) Equivalent Pier Method

$B = A_G^{\ 0.5} = 28{,}632$ m

$A_P = \Pi d^2 n/4 = 17{,}427$ m^2

$E_p = 25000$ MPa

$E_s' = 41{,}86$ MPa $E_u = 48{,}3$ MPa

$d_e = 1{,}27\ A_G^{0,5} = 36{,}36$ m (for friction piles)

$\rho = 0{,}372$ $L = 7{,}6$ m

$E_e = E_p A_p/A_G + E_s(1 - A_p/A_G)$

$\zeta_1 = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,3	572,395	35,552	-1,299	———			
			1,662	0,076	0,208	0,842	87,69
υ_s=0,4	575,547	35,748	-1,453	———			
			1,655	0,076	0,208	0,790	76,41

Method 2

L/d_e=7,6/36,33=0,208→ I_δ=0,5 (Fig. 2.10)

	υ_s=0,3	υ_s=0,4
δ (mm)	52,03	48,31

K ≈ 520 (pile stiffness factor) s/d ≈ 6 L/d ≈ 18,53 B=28,63 m

d_e/B ≈ 0,8 assumed, then d_e ≈ 22,90 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,3	572,395	35,552	-0,837	———			
			1,692	0,121	0,330	0,811	133,98
υ_s=0,4	575,547	35,748	-0,991	———			
			1,681	0,121	0,330	0,767	117,66

Method 2

L/d_e=7,6/22,90=0,331 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,3	υ_s=0,3
δ (mm)	82,59	76,69

$\delta_{measured}$=64 mm

c) Equivalent Raft Method

L	B	H	D	D/B	L/B	H/B	z/B
36,53	26,53	1,63	5	0,190	1,377	0,061	0,030
38,41	28,41	10,8	6,7	0,235	1,352	0,380	0,2646

P=158400 KN

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$

μ_0	μ_1	q_n	B	E_u	δ_i
0,96	0,01	163,44	26,53	8,4	4,95
0,96	0,13	145,15	28,41	48,3	10,65

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}=0,162 \rightarrow \mu_d=0,975$

Sand → $\mu_g=1$

N=22 $q_c/N=5$ $M=5q_c$

$\delta_c=m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

σ_z/q	σ_z	m_v	δ_c
0,97	158,54	0,102	25,71
0,74	120,94	0,018	22,92

$\delta_T=\delta_i+\delta_c$ = 64,24 mm

$\delta_{measured}$=64 mm

Table A.26: Measured and computed settlements for 19-Storey Reinforced Concrete Building (mm)

	Settlement (mm)						
	Set. Ratio	Equivalent Pier				Eq. Raft	Mea.
		d_{e1}		d_{e2}			
		Met1	Met2	Met1	Met2	Ave.	
υs=0,3	87,79	87,69	52,03	133,98	82,59	64,24	64
υs=0,4	77,12	76,41	48,31	117,66	76,69	53,53	

19 - Storey Reinforced Concrete

Rs	87,79	Mea.	64
Pier	87,69 52,03		
Raft	64,24		

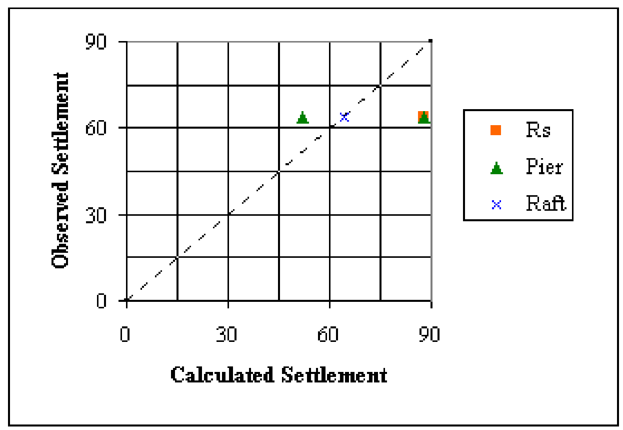

Figure A.53: Measured and computed settlements for 19-Storey Reinforced Concrete Building (mm)

26. Hotel Japan (n=157)

The steel frame structure has up to 21 storeys above ground level (125 m in height) and generally a 3-storey basement, increasing to 4 storeys (19 m depth) beneath the tower. The plan area of the building complex is about 3300 m2. The raft thickness varies from 2.0 m to 3.7 m, and the 157 cast-in-place piles (1.0-1.8 m diameter) were designed to carry the entire building load. Modelling of piled raft foundation as beam grillage supported on springs of variable stiffness, average settlemet is calculated between 20-25 mm (Nagao, T., and Majima, M., 2000)

a) Settlement Ratio Method

n= 157 d= 1,5 m r_0= 0,75 m

L= 20 m s= 2 m

Depth: 32-38 m E_u= 87,2 MN/m^2

38-65 m E_u= 129,0 MN/m^2

65-105 m E_u= 245,0 MN/m^2

E_p= 35000 MN/m^2

υ_s= 0,33

P=196,250 MN

$\lambda=E_p/G_l=35000/43 \approx 813,95$

$\rho=G_{l/2}/G_l=0,675 \rightarrow 0,975$

$\log\lambda=2,91 \rightarrow 0,97$

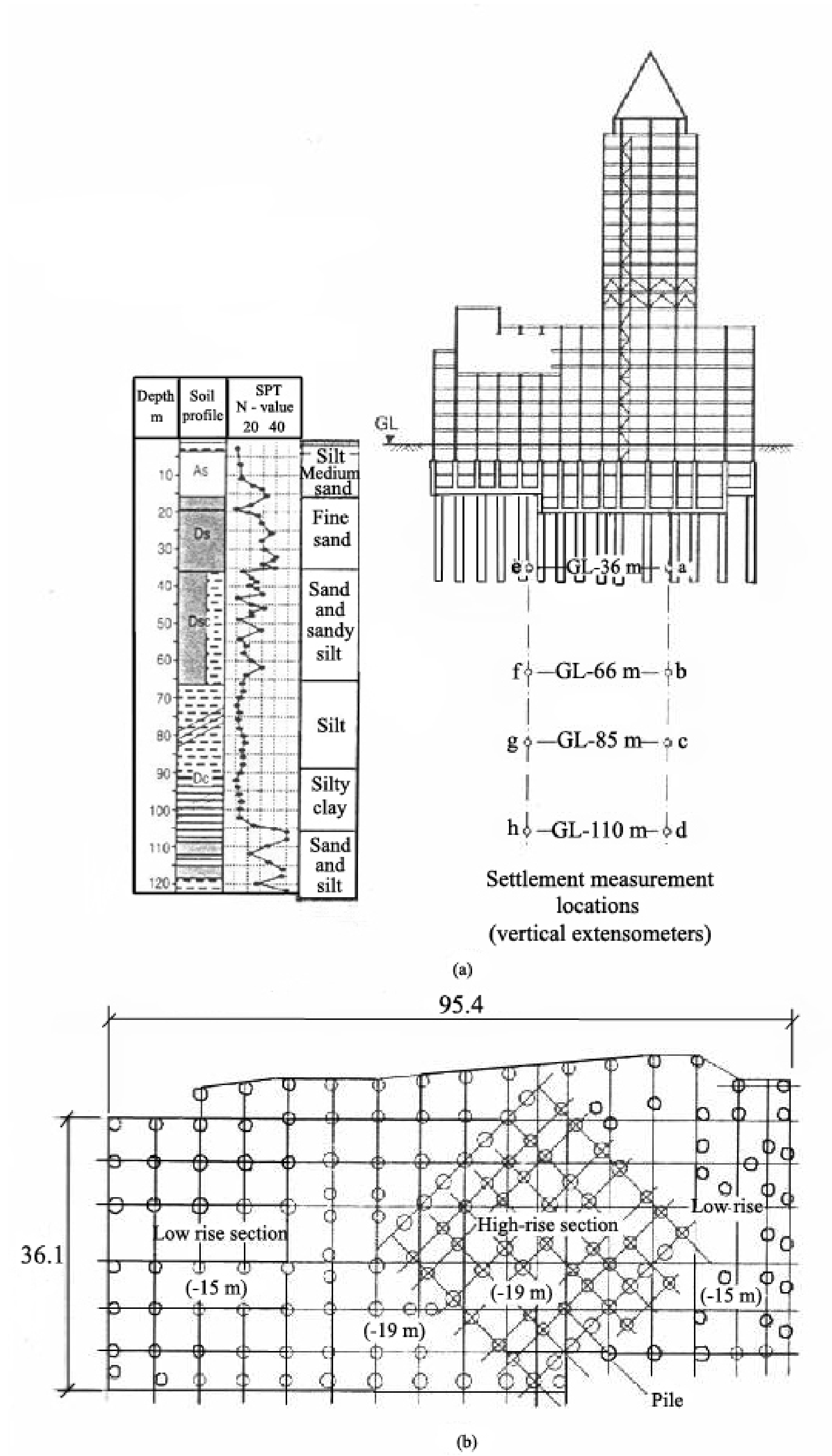

Figure A.54: Building complex in Nigita City, Japan; **(a)** longitudinal cross-section and soil profile; **(b)** foundation plan (Nagao and Majima, 2000)

$s/d=2 \rightarrow 1{,}09$

$L/d=13{,}33 \rightarrow 0{,}518$

$\upsilon_s=0{,}33 \rightarrow 0{,}99$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al, 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single}=196250/157=1250$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0{,}33$	0,528	0,069	14,483	3,407	0,716	22,882	32,737

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0{,}33$	1055,78	11444,94	17,14	1,184	17,14

$\delta_{measured}=17{,}5$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=58{,}68$ m

$A_P=\Pi d^2 n/4=277{,}44\ m^2$

$E_p=35000$ MPa

$E_s'=114{,}38$ Mpa $E_u=129$ MPa

$d_e=1,27\ A_G^{0,5}=74,53$ (for friction piles)

$\rho=0,675$ $L=20$ m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0,25+(2,5\ \rho\ (1-\upsilon)-0,25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0,33$	2924,75	68,017	-0,498	———			
			1,724	0,070	0,267	0,740	17,05

Method 2

$L/d_e=20/74,53=0,268 \rightarrow I_\delta=0,5$ (Fig. 3.10)

$\delta=11,51$ mm

$K \approx 270$ (pile stiffness factor) $s/d \approx 2$ $L/d \approx 13,33$ $B=58,68$ m

$d_e/B \approx 0,88$ assumed, then $d_e \approx 51,64$ m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0,33$	2924,75	68,017	-0,131	———			
			1,771	0,099	0,385	0,694	23,08

Method 2

L/d_e=20/51,64=0,387 → I_δ=0,5 (Fig. 2.10)

δ=11,51 mm

$\delta_{measured}$=17,5 mm

c) Equivalent Raft Method

L	B	H	D	σ_z/q	σ_z	E_s'	m_v
102,1	42,76	6	32	0,93	41,82	77,31	0,014
109	49,68	27	38	0,64	28,78	114,38	0,018
140,2	80,85	40	65	0,29	13,04	217,23	0,003

$D/(LB)^{0,5}$=0,484→m_d=0,855

Fine sand→ μ_g=1

Silt, silty clay→ μ_g=0,7

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$ = 3,00 + 11,96 +0,97

= 15,93 mm

μ_0	μ_1	E_{uave}	q	δ_i
0,925	0,04	87,2	39,457	0,71
0,925	0,2	129	31,805	2,26
0,92	0,17	245	15,196	0,78

$\delta_i = q_n\ B\ \mu_0\ \mu_1 / E_u$ = 3,76 mm

$\delta_T = \delta_i + \delta_c$ = **19,69 mm**

$\delta_{measured}$=17,5 mm

Table A.27: Measured and computed settlements for Hotel-Japan (mm)

	Settlement (mm)						
	Set. Ratio	Equivalent Pier				Eq. Raft	Mea.
		d_{e1}		d_{e2}		Ave	
		Met1	Met2	Met1	Met2		
υs=0,33	17,14	— 17,05	11,51	— 23,08	16,61	19,69	17,50

Hotel Japan

Rs 17,14

Mea. 17,5

Pier 17,08
11,51
23,08

Raft 19,69

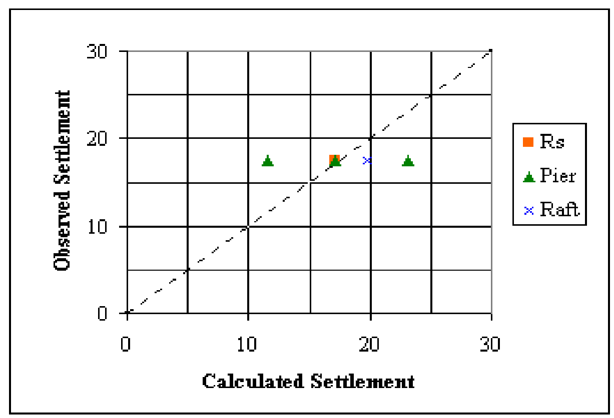

Figure A.55: Measured and computed settlements for Hotel-Japan (mm)

27. Izmir Hilton Complex (n=189)

The construction of the Izmir Hilton Complex founded on a sigle raft supported by piles. The soil profile indicates that the foundation soils are deep stiff clays containing sand-gravel layers except the upper 15 m where fills and soft and loose to medium dense recent alluvial deposits lie. These include sands, silts and clays. The ground water level is 3 m deep from the ground surface. There are 189 piles under the raft, 138 of them under the tower block. Foundation piles under the raft were bored piles 1.20 m in diameter when cased (1.06 m uncased). The ends of piles were located at depths of 33 m to 42 m from the ground surface. The closest spacing of the piles under the tower is 3.00 m by 2.50 m. Settlement of the piled raft has been estimated independently using the group interaction analysis, and the group settlement is obtained as 74,5 mm. (Ergun, M. U., 1995)

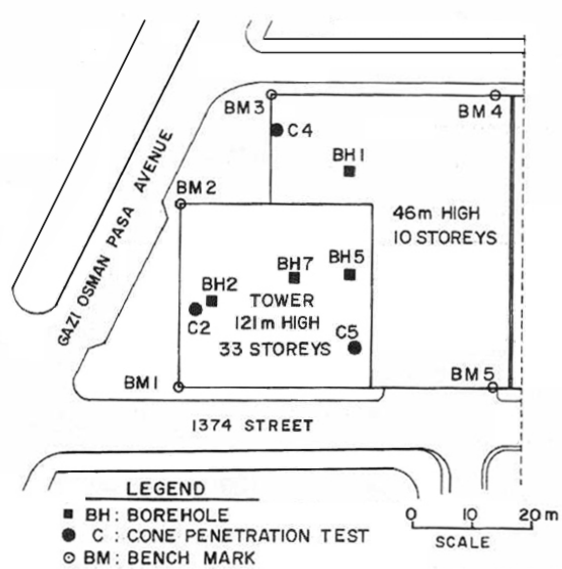

Figure A.56: Plan view of the tower and the site (Ergun, 1995)

Table A.28: Summary of soil properties (Ergun, 1995)

Sym.	Desription	Engineering Properties
GS	Gravelly sand, contains silty and clayey bands	N=20-36 N=4 Clay bands q_c=4-12 Mpa
C1	Sand silty clay soft to firm black and grey	N=2-5 q_c=0,6-0,9 Mpa C_u=30 kPa(UU Tests) m_v=0,05-0,06*10^{-2} m^2/kN (100-400 kPa Interval)
C	Silty clay light brown contains some gravel	N=20 (15m-40m) q_c=1,5-2 Mpa (15m-35m) N=29 (40m 63m) C_u=80-120 kPa (UU Tests, 0-30m) m_v=0,01-0,02*10^{-2} m^2/kN (100-400 kPa)
SG	Sandy gravel gray	N=36 q_c=16-20 MPa

a) Settlement Ratio Method

n= 189 d= 1,06 m r_0= 0,53 m

L= 28,5 m s= 2,756 m

υ_s= 0,2 υ_s=0,3 E_p= 30000 MN/m^2

P=819315 KN

$\lambda=E_p/G_l$ =30000/50 ≈ 600

$\rho=G_{l/2}/G_l$=30/50=0,6⟶0,955

logλ =2,778⟶0,945

s/d=2,6⟶1,04

L/d_{ave}=26,9⟶0,547

υ_s=0,2→1,03

υ_s=0,3→1

$\eta_w = n^{-e}$ $R_s = n^e$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 1$

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

P_{single}=819315/189=4335 KN

	e	η_w	R_s	ζ	μL	tanhμL L/(μL r_0)	$P_t/(w_t G_l r_0)$
υ_s=0,2	0,528	0,062	15,988	4,167	1,520	32,134	31,395
υ_s=0,3	0,513	0,067	14,748	4,034	1,545	31,763	32,291

	P_t/w_t	$K=n\eta_w k$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,2	831,975	9835,013	83,30	5,21	83,30
υ_s=0,3	855,729	10966,37	74,71	5,06	74,71

$\delta_{measured}$=69,6 mm

b) Equivalent Pier Method

$B = A_G^{\ 0.5}$ =51,3 m

$A_P = \Pi d^2 n/4$=166,78 m^2

E_p=30000 MPa

E_s'=120 MPa E_u=150 MPa

$d_e=1{,}27\ A_G^{0{,}5}=65{,}15$ m (for friction piles)

$\rho=0{,}6$ $L=28{,}5$m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}2$	2013,65	40,272	0,048	0,883	0,350	0,083	8,71
			1,800	0,145	0,434	0,728	76,29
$\upsilon_s=0{,}3$	2023,01	40,460	-0,084	———			
			1,778	0,145	0,434	0,715	69,17

Method 2

$L/d_e=28{,}5/65{,}15=0.437 \rightarrow I_\delta=0{,}5$ (Fig. 2.10)

	$\upsilon_s=0{,}2$	$\upsilon_s=0{,}3$
δ (mm)	52,39	46,36

$K \approx 220$ (pile stiffness factor) $s/d \approx 2{,}6$ $L/d \approx 26{,}88$ $B=51{,}3$ m

$d_e/B \approx 0{,}8$ assumed, then $d_e \approx 41{,}04$ m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tan\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,2	2013,65	40,272	0,510	0,433	0,654	0,344	57,31
			1,897	0,224	0,683	0,655	109,10
υ_s=0,3	2023,01	40,460	0,377	0,502	0,641	0,296	45,57
			0,865	0,226	0,682	0,651	100,01

Method 2

L/d_e=28,5/41,04=0,694 $\longrightarrow$ I_δ=0,47 (Fig. 2.10)

	υ_s=0,2	υ_s=0,3
δ (mm)	78,19	72,17

$\delta_{measured}$=69,6 mm

c) Equivalent Raft Method

L=64,5 m B=57,35 m H=114,7 m

L/B=1,124 D/B=0,331 H/B=2

P=819315 KN q_n=P/(B*L) = 221,49 KN

E_u=150 Mpa E_s'= 120Mpa υ_s=0,2

μ_1 = 0,52 μ_0 = 0,95

$\delta_{i\ ave}=\mu_1\mu_0 q_n B/E_u$ =0,95 . 0,52 . 221,49 . 57,35 / 150 = 41,83 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0{,}0075$

$z/B = 1 \quad \sigma_z/q = 0{,}294 \quad \sigma_z = 65{,}118$ kN/m^2

$D/(LB)^{0,5} = 0{,}312 \rightarrow \mu_d = 0{,}92$

Stiff Clay $\rightarrow \mu_g = 0{,}7$

$\delta_c = m_\upsilon \, \sigma_z \, H \, \mu_d \, \mu_g \quad = 0{,}0075 \, . \, 65{,}118 \, . \, 114{,}7 \, . \, 0{,}92 \, . \, 0{,}7 = 36{,}07$ mm

$\delta_T = \delta_i + \delta_c = 77{,}91$

$\delta_{measured} = 69{,}6$ mm

Table A.29: Measured and computed settlements for Izmir Hilton (mm)

	Settlement (mm)							
	Set. Ratio	Equivalent Pier				Equivalent Raft		Mea.
		d_{e1}		d_{e2}		H=96 m	H=114,7 m	
		Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,2	83,30	8,71 76,29	52,39	57,31 109,10	78,19	74,11	77,91	69,60
υs=0,3	74,71	___ 69,17	48,36	45,57 100,01	72,17	66,04	69,32	

İzmir Hilton

Rs	83,3	Mea.	69,6
Pier	76,29		
	52,39		
	57,31		
Raft	77,91		

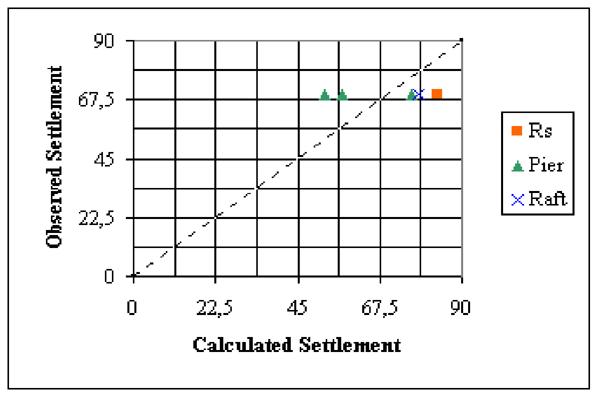

Figure A.57: Measured and computed settlements for Izmir Hilton (mm)

28. Frame-Type Building 6 (n=192)

The building is 18 m square in plan, and is located on 192 piles with a length of 21 m. Shingle, E_u=80 Mpa, is located under 35*35 cm section piles. The over load on the foundation is 268,8 MN. The piles in the group are arranged in 1,3 m spacing. Different formulations are used to obtain settlement value. Settlement predictions and methods are given below:

USSR standarts	Poulos	Vesic	Skempton	Bartolomey:
20 mm	71 mm	67 mm	108 mm	24 mm

(Bartolomey, A.A., 1981)

a) Settlement Ratio Method

n= 192   d= 0,394 m r_0= 0,197 m

L= 21 m   s= 1,3 m

E_u=80 MN/m^2 Shingle E_p= 25000 MN/m^2

P=268,8 MN υ_s= 0,35 υ_s=0,4

$\lambda=E_p/G_l=25000/26,7 \approx 936,329$

$\rho=G_{l/2}/G_l=1\rightarrow1,06$

$\log\lambda=2,971\rightarrow0,99$

$s/d=3,3\rightarrow0,98$

$L/d=53,299\rightarrow0,548$

$\upsilon_s=0,35\rightarrow0,98$

$\upsilon_s=0,4\rightarrow0,97$

$\eta_w = n^{-e}$ $R_s = n^e$ $\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 1$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al, 1992)

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single} = 268800/192 = 1400$ KN

	e	η_w	R_s	ζ	μL	$\tan\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0{,}35$	0,552	0,054	18,241	5,155	2,169	47,859	58,623
$\upsilon_s=0{,}4$	0,546	0,056	17,71	5,075	2,187	47,528	59,144

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0{,}35$	308,351	3245,519	82,82	4,54	82,82
$\upsilon_s=0{,}4$	311,093	3372,85	79,69	4,50	79,69

$\delta_{measured} = 19$ mm

b) Equivalent Pier Method

$B = A_G^{\ 0.5} = 18$

$A_P = \Pi d^2 n/4 = 4{,}712\ m^2$

$E_p = 25000$ MPa

$E_s' = 72$ MPa $E_u = 80$ MPa

$d_e = 1{,}13\ A_G^{0,5} = 20{,}34$ (for end-bearing piles)

$\rho = 1$ $L = 21$ m

$E_e = E_p A_p/A_G + E_s(1 - A_p/A_G)$

ζ_1=ln(2,5 ρ (1-υ) L/r_0) (W. Fleming, et al, 1992)

ζ_2=ln/{5+[0,25+(2,5 ρ (1-υ)-0,25)ξ] L/r_0} (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,35	1873,051	70,239	1,210	0,316	0,999	0,344	63,31
			2,123	0,239	1,013	0,469	86,17
υ_s=0,4	1875,525	70,332	1,130	0,327	0,997	0,334	59,20
			2,091	0,240	1,012	0,465	82,47

Method 2

L/d_e=21/20,34=1,032→ I_δ=0,415 (Fig. 2.10)

	υ_s=0,35	υ_s=0,4
δ (mm)	76,17	73,45

K ≈ 300 (pile stiffness factor)   s/d ≈ 3,3   L/d ≈ 53,29   B=18 m

d_e/B ≈ 0,7 assumed, then d_e ≈ 12,6 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanμL L/(μL d_e)	I_δ	δ
υ_s=0,35	1873,051	70,239	1,689	0,432	1,569	0,329	97,59
			2,343	0,367	1,595	0,399	118,45
υ_s=0,4	1875,525	70,332	1,609	0,443	1,565	0,324	92,70
			2,302	0,370	1,594	0,399	114,12

Method 2

L/d_e=21/12,6=1,667 → I_δ=0,33 (Fig. 2.10)

	υ_s=0,35	υ_s=0,4
δ (mm)	97,77	94,28

$\delta_{measured}$=19 mm

c) Equivalent Raft Method

L=18 B=18 L/B=1

H=36 D=21 D/B=1,667 H/B=2

μ_0=0,91 μ_1=0,53

q_n=268800/(BL) = 829,63 KPa

$\delta_{i\ ave}=q_n\ \mu_0\ \mu_1 B/E_u$= 829,63 0,91 0,53 18 / 80 = 90,03

$\delta_{measured}$=19 mm

Table A.30: Measured and computed settlements for Frame Type Building 6 (mm)

	Settlement (mm)						
	Set. Ratio	**Equivalent Pier**				**Eq. Raft**	**Mea.**
		d_{e1}		d_{e2}		Ave.	
		Met1	Met2	Met1	Met2		
υs=0,35	82,82	63,31	76,17	97,59	97,77	90,03	19,00
		86,17		118,45			
υs=0,4	79,69	59,20	73,45	92,70	94,28		
		82,47		114,12			

Frame 6 (Shingle)

Rs	79,69	Mea.	19
Pier	82,47		
	73,45		
Raft	90,03		

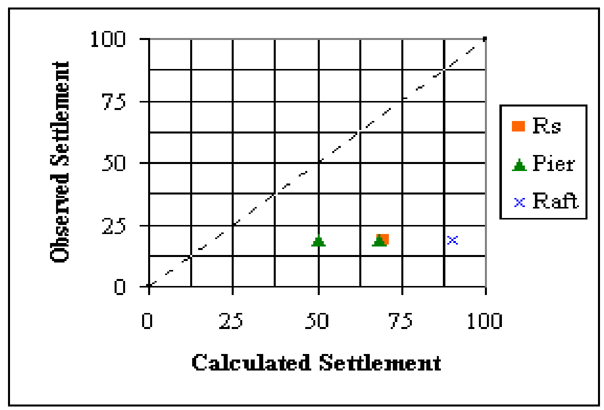

Figure A.58: Measured and computed settlements for Frame Type Building 6 (mm)

29. Stonebridge Park Flats (n=351)

16-storey block of flats built at Stonebridge Pak in London borough of Brent. The actual foundation involved the use of a raft 0.9 mm thick, with 351 piles, 450 mm diameter and 13 m long, driven into London clay. The stiffness of the pile group is estimated, using program PIGLET (simplified continiuum analysis), and the average settlement under a load of 156 MN may be calculated as 27 mm. (H.G. Poulos 2001, W.G.K. Fleming, et al. 1992)

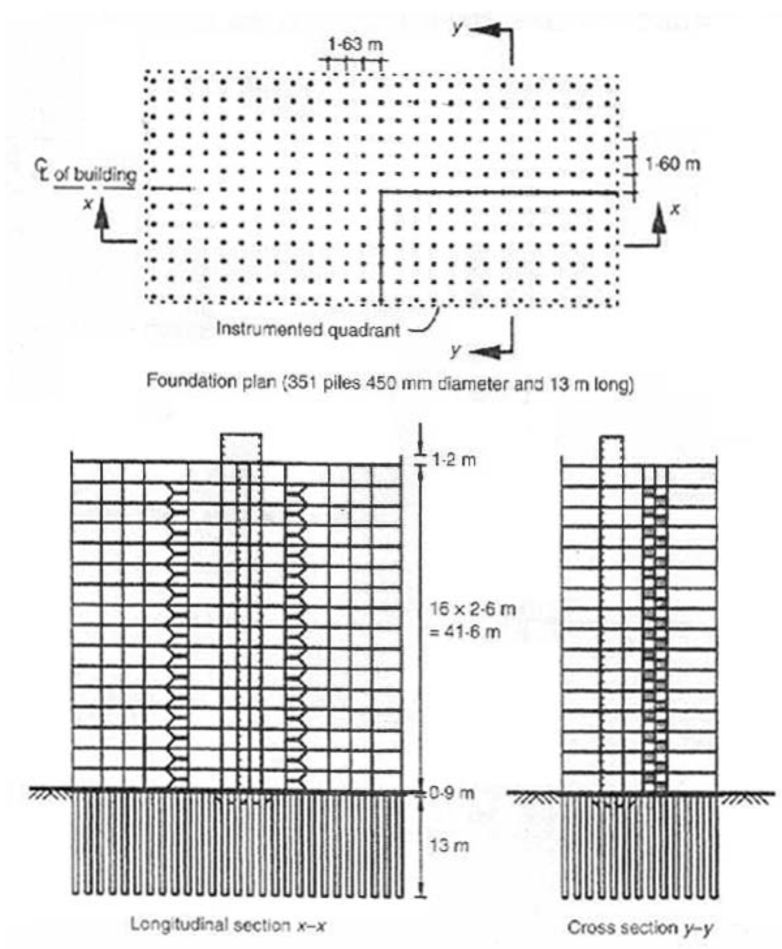

Figure A.59: Stonebridge foundation details (Poulos, 2001, Fleming et al, 1992)

a) Settlement Ratio Method

$n = 351$ $d = 0{,}45$ m $r_0 = 0{,}225$ m

$L = 13$ m $s = 1{,}60\text{-}1{,}63$ m

$G = 1{,}44z+20$ (MN/m^2)   London Clay $E_p = 25000$ MN/m^2

$P = 156$ MN $\upsilon_s = 0{,}1$ $\upsilon_s = 0{,}3$

$\lambda = E_p/G_l = 25000/38{,}72 \approx 645{,}66$

$\rho = G_{l/2}/G_l = 0{,}758 \rightarrow 1$

$\log\lambda = 2{,}810 \rightarrow 0{,}95$

$s/d = 3{,}58 \rightarrow 0{,}96$

$L/d = 28{,}89 \rightarrow 0{,}548$

$\upsilon_s = 0{,}1 \rightarrow 1{,}05$

$\upsilon_s = 0{,}3 \rightarrow 1$

$\eta_w = n^{-e}$ $R_s = n^e$

$\zeta = \ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\eta = r_b/r_0 = 1$ $\xi = G_l/G_b = 1$

$\mu L = (2/(\lambda\zeta))^{0,5} L/r_0$

$P_{single} = 156000/351 = 444{,}4$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0{,}1$	0,524	0,046	21,661	4,591	1,500	34,851	37,731
$\upsilon_s=0{,}3$	0,499	0,053	18,710	4,339	1,543	34,162	39,426

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,1	328,720	5326,55	29,28	1,35	29,28
υ_s=0,3	343,486	6443,676	24,20	1,29	24,20

$\delta_{measured}$=25 mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =29,494 m

$A_P=\Pi d^2 n/4$=55,82 m^2

E_p=25000 MPa

G= 1,44z+20 (MN/m^2) E_s'=85,184 MPa E_u=116,16 MPa

$d_e=1{,}27\ A_G^{0{,}5}$ =37,45 m (for friction piles)

ρ=0,758 L=13 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
υ_s=0,1	1683,99	43,491	0,169	0,331	0,332	0,193	9,47
			1,822	0,110	0,345	0,719	35,18
υ_s=0,3	1698,49	43,866	-0,082	—	—	—	—
			1,778	0,111	0,345	0,707	29,24

Method 2

L/d_e=13/37,45=0,347 → I_δ=0,5 (Fig. 2.0)

	υ_s=0,1	υ_s=0,3
δ (mm)	24,44	20,63

K ≈ 300 (pile stiffness factor) s/d ≈ 3,58 L/d ≈ 28,89 B=29,49 m

d_e/B ≈ 0,8 assumed, then d_e ≈ 23,59 m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1\text{-}2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,1	1683,99	43,491	0,631	0,297	0,535	0,363	28,21
			1,928	0,170	0,546	0,638	49,52
υ_s=0,3	1698,49	43,866	0,379	0,381	0,525	0,287	18,85
			1,865	0,172	0,545	0,639	41,99

Method 2

L/d_e=13/23,59=0,55 → I_δ=0,5 (Fig. 2.10)

	υ_s=0,1	υ_s=0,3
δ (mm)	38,80	32,83

$\delta_{measured}$=25 mm

c) Equivalent Raft Method

L	B	H	L/B	H/B	D/B
47,17	23,98	23,98	1,96	1	0,36
50,15	35,97	23,98	1,64	0,66	0,90

P=156000 KN υ_s= 0,1

$\delta_{i\ ave} = \mu_1 \mu_0 q_n B / E_u$

μ_0	μ_1	E_{uave}	q	δi
0,95	0,35	149,38	137,94	7,36
0,92	0,23	252,97	73,32	2,20

$\delta_{i\ ave}$ = 9,56 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)]$

$D/(LB)^{0,5}$=0,258 → μ_d=0,94

London Clay → μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$

$E_{mid\text{-}dr}$	m_v	σ_z	δ_c
109,54	0,0089	84,14	11,85
304,77	0,0052	27,58	2,29

δ_c= 14,14 mm

$\delta_T = \delta_{i\ ave} + \delta_c$ = 23,71 mm

$\delta_{measured}$=25 mm

Table A.31: Measured and computed settlements for Stonebridge Park (mm)

	Settlement (mm)							
	Set. Ratio	Equivalent Pier				Equivalent Raft		Mea.
		d_{e1}		d_{e2}		H=39,3 m	H=47,96 m	
		Met1	Met2	Met1	Met2	Ave	Ave	
υs=0,1	29,28	9,47 35,18	24,44	28,21 49,52	38,80	24,33	23,71	25
υs=0,3	24,21	—— 29,24	20,68	18,85 41,99	32,83	18,83	18,66	

	Stonebridge		
Rs	29,28	Mea.	25
Pier	35,18		
	24,44		
	28,21		
Raft	24,33		
	23,71		

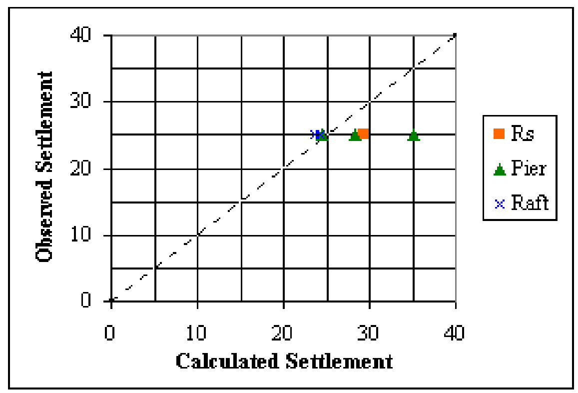

Figure A.60: Measured and computed settlements for Stonebridge Park (mm)

30. Dashwood House (n=462)

The pile group consisted of 462 bored piles with a diameter of 0,485 m and length of 15 m was capped by a rectangular raft of 33.8*32.6 m. The piles in the group were arranged in a grid of 1.5-m square spacing. The overall load on the foundation was 279 MN. Based on the computed settlement of single pile and group settlement ratio of actual pile group, the settlement of the pile group is obtained as 36.2 mm (W.Y. Shen, Y.K. Chow and K.Y. Yong 2000)

a) Settlement Ratio Method

$G= 30+1,33z$ (MN/m^2) London Clay $E_p= 30000$ MN/m^2

$n= 462$ $d= 0,485$ m $r_0= 0,2425$ m

$L= 15$ m $s= 1,5$ m

$\upsilon_s= 0,15$ $\upsilon_s=0,3$ $P=279$ MN

$\lambda=E_p/G_l=30000/49,95 \approx 600$

$\rho=G_{l/2}/G_l=0,8\longrightarrow1,015$

$\log\lambda=2,778\longrightarrow0,94$

$s/d=3,093\longrightarrow0,99$

$L/d=30,927\longrightarrow0,55$

$\upsilon_s=0,15\longrightarrow1,04$

$\upsilon_s=0,3\longrightarrow1$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2,5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming et al. 1992)

$\eta=r_b/r_0=1$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

$P_{single}=279000/462=603,896103$ KN

	e	η_w	R_s	ζ	μL	$\tanh\mu L\ L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
$\upsilon_s=0,15$	0,540	0,036	27,521	4,656	1,654	34,753	38,87
$\upsilon_s=0,3$	0,519	0,041	24,227	4,462	1,689	34,192	40,09

	P_t/w_t	$K=n\eta_w k$	$\delta=P/K$(mm)	P_{single}/k	$\delta=\delta_s R_s$
$\upsilon_s=0,15$	470,845	7903,998	35,29	1,28	35,29
$\upsilon_s=0,3$	485,706	9262,208	30,12	1,24	30,12

$\delta_{measured}=33$ mm

b) Equivalent Pier Method

$B=A_G^{\ 0.5}=31,241$ m

$A_P=\Pi d^2 n/4=85,35\ m^2$

$E_p=30000$ MPa

$E_s'=114,885$ MPa $E_u=149,85$ MPa

$d_e=1,27\ A_G^{0,5}=39,67$ (for friction piles)

$\rho=0,8$ $L=15$ m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

ζ_1=ln(2,5 ρ (1-υ) L/r_0) (W. Fleming, et al. 1992)

ζ_2=ln{5+[0,25+(2,5 ρ (1-υ)-0,25)ξ] L/r_0} (K. Horikoshi, M. Randolph,1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,15	2728,373	54,622	0,251	0,288	0,368	0,241	14,79
			1,838	0,106	0,376	0,694	42,47
υ_s=0,3	2742,047	54,895	0,057	0,603	0,338	0,081	4,42
			1,801	0,107	0,376	0,681	36,92

Method 2

L/d_e=15/39,67 =0,378 → I_δ=0,5   (Fig. 2.10)

	υ_s=0,15	υ_s=0,3
δ (mm)	30,60	27,07

K ≈ 200 (pile stiffness factor)   s/d ≈ 3,093   L/d ≈ 30,92   B=31,24 m

d_e/B ≈ 0,8 (Fig. 2.9) assumed, then d_e ≈ 28 m

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	tanhμL L/(μL d_e)	I_δ	δ
υ_s=0,15	2728,373	54,622	0,713	0,271	0,585	0,366	35,59
			1,951	0,164	0,594	0,611	59,39
υ_s=0,3	2742,047	54,895	0,519	0,317	0,580	0,318	27,37
			1,899	0,166	0,594	0,609	52,41

Method 2

L/d_e=15/28=0.535 ⟶ I_δ=0,5 (Fig. 2.10)

	υ_s=0,15	υ_s=0,3
δ (mm)	48,58	42,97

$\delta_{measured}$=33 mm

c) Equivalent Raft Method

L=36,985 m B=35,485 m H=70,97 m

L/B=1,042 D/B=0,281 H/B=1,719

P=279000 KN q_n=P/(B*L) = 212,585 KN

E_u=271,48 MPa E_s'= 208,13 MPa υ_s=0,15

μ_1 = 0,525 μ_0 = 0,95

$\delta_{i\ ave} = \mu_1\mu_0 q_n B/E_u$ =0,95 . 0,525 . 212,585 . 35,485 / 271,48 = 13,85 mm

$m_\upsilon=[(1+\upsilon)(1-2\upsilon)]/[E_s'(1-\upsilon)] \approx 0{,}00455$

z/B= 1 σ_z/q=0,32 σ_z=68,027 kN/m^2

$D/(LB)^{0,5}$=0,276⟶ μ_d=0,935

London Clay ⟶ μ_g=0,7

$\delta_c = m_\upsilon\ \sigma_z\ H\ \mu_d\ \mu_g$ =0,00455 . 68,027 . 70,97 . 0,935 . 0,7 = 14,37 mm

$\delta_T=\delta_{i\ ave}+\delta_c$ = 28,23 mm

$\delta_{measured}$ = 33 mm

Table A.32: Measured and computed settlements for Dashwood House (mm)

	Settlement (mm)							
	Set. Ratio	Equivalent Pier				Equivalent Raft		Mea.
		d_{e1}		d_{e2}		H=61 m	H=70,97 m	
		Met1	Met2	Met1	Met2	Ave.	Ave.	
υs=0,15	35,29	14,79	30,6	35,59	48,58	27,57	28,23	33
		42,47		59,39				
υs=0,3	30,12	4,42	27,07	27,37	42,97	23,49	23,83	
		36,92		52,41				

Dashwood

Rs	35,29	Mea.	33
Pier	42,53		
	30,60		
	35,59		
Raft	27,57		
	28,23		

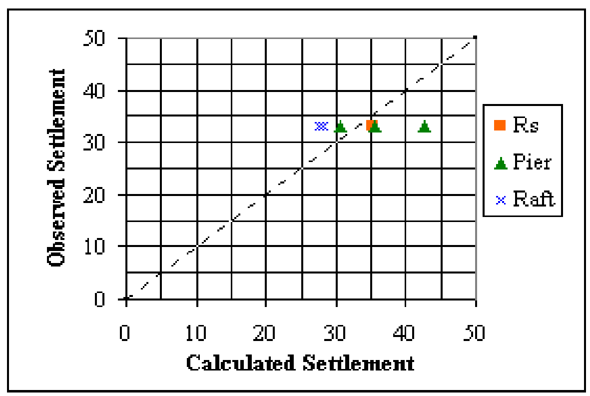

Figure A.61: Measured and computed settlements for Dashwood House (mm)

31. Ghent Grain Terminal (n=697)

In 1975 a block of 40 cylindrical reinforced concrete grain silo cells was erected in Ghent, within a new terminal for storage and transit. The inner diameter of each cell is 8 m, the total height 52 m and the wall thickness 0,18 m. The foundation consists of 1.2 m thick slab, 34 m * 84 m in plan, resting on 697 driven, cast in situ, reinforced concrete piles with a length of 13.4 m, a shaft diameter of 0.52 m and a diameter of expanded base of 0.8 m. Using the GASGROUP(using superposition principle, with interaction factors) analysis, the settlement can be estimated to be 186,3 mm and the settlement of the pile group is obtained approximately as 150 mm using the program GRUPPALO (based on the use of interaction factors). (Mandolini, A., and Viggiani, C. 1997, Poulo, H.G., 1993, Randolph, M.F., and Guo, W.D., 1999)

a) Settlement Ratio Method

n= 697 d= 0,52 m d_{base}=0,80 m r_0= 0,26 m

L= 13,4 m s= 2,08 m

E_p= 30000 MN/m^2

υ_s= 0,15 Clayey Sand

P=906,1 MN

P_{single}=906100/697=1300 KN

$\lambda=E_p/G_l$=30000/66,75 ≈ 449,43

$\rho=G_{l/2}/G_l$=0,749→1

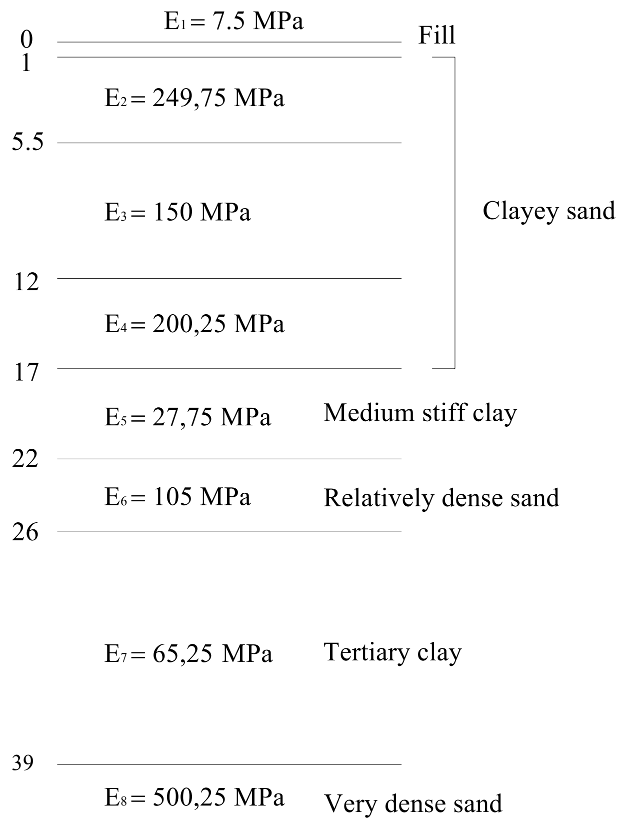

Figure A.62: Subsoil profile and subsoil model adopted in the analysis (Mandolini and Viggiani, 1997, Poulo, 1993, Randolph and Guo, 1999)

$\log\lambda=2{,}65\rightarrow0{,}92$

$s/d=4\rightarrow0{,}93$

$L/d=25{,}77\rightarrow0{,}545$

$\upsilon_s=0{,}15\rightarrow1{,}04$

$\eta_w=n^{-e}$ $R_s=n^e$

$\zeta=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$r_m=2{,}5\rho(1-\upsilon)L$ $\eta=r_b/r_0=1{,}538$ $\xi=G_l/G_b=1$

$\mu L=(2/(\lambda\zeta))^{0,5}L/r_0$

	e	η_w	R_s	ζ	μL	$\tanh\mu L$ $L/(\mu L\ r_0)$	$P_t/(w_t G_l r_0)$
υ_s=0,15	0,484	0,041	23,924	4,407	1,637	29,177	33,401

	P_t/w_t	$K=n\eta_w k$	δ=P/K(mm)	P_{single}/k	$\delta=\delta_s R_s$
υ_s=0,15	579,690	16888,52	53,65	2,24	53,65

Effect of soft layers (E_u=27,75 – 65,25 MPa for clay, 105 Mpa for sand)

r_m	B_{raft}	L_{raft}	D	H	z	q_n	E_u	E_s'	υ
21,329	55,329	105,329	13,4	3,6	1,8	155,477	200,25	153,52	0,15
	57,129	107,129	17	5	6,1	148,049	27,75	22,2	0,2
	59,629	109,629	22	4	10,6	138,607	105	84	0,2
	61,629	111,629	26	13	19,1	131,706	65,25	52,2	0,2

$D/\sqrt{LB}$	m_d	m_g	σ_z/q	σ_z	μ_0	μ_1	m_v	δ_i	δ_c
0,175	0,97	0,85	0,98	152,368	0,95	0,01	0,00616		
	0,97	0,85	0,90	139,930	0,95	0,01	0,04054	2,89	23,38
	0,97	1	0,81	125,937	0,95	0,01	0,025	0,74	12,21
	0,97	0,85	0,71	110,389	0,94	0,0	0,01724	5,84	20,40

δ_{Total}= 53,65+2,89+0,74+5,84+23,38+12,21+20,40=119,14

b) Equivalent Pier Method

$B=A_G^{\ 0.5}$ =53,44 m

$A_P=\Pi d^2 n/4$=148,023 m^2

E_p=30000 Mpa E_s'=153,525 MPa E_u=200,25 MPa

$d_e=1,27\ A_G^{0,5}$ =67,87 m (for friction piles)

ρ=0,749 L=13,4 m

$E_e=E_pA_p/A_G+E_s(1-A_p/A_G)$

$\zeta_1=\ln(2{,}5\ \rho\ (1-\upsilon)\ L/r_0)$ (W. Fleming, et al., 1992)

$\zeta_2=\ln/\{5+[0{,}25+(2{,}5\ \rho\ (1-\upsilon)-0{,}25)\xi]\ L/r_0\}$ (K. Horikoshi, M. Randolph, 1999)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μL	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}15$	1700,432	25,474	-0,464				
			1,727	0,084	0,196	0,814	70,82

Method 2

$L/d_e=13{,}4/67{,}87=0{,}197\rightarrow I_\delta=0{,}5$ (Fig. 2.10)

$\delta=43{,}48$ mm

$K\approx150$ (pile stiffness factor) $s/d\approx4$ $L/d\approx25{,}77$ $B=53{,}44$ m

$d_e/B\approx0{,}75$ assumed, then $d_e\approx40{,}08$ m (Fig. 2.9)

Method 1

	E_e	λ	$\zeta_{(1-2)}$	μl	$\tanh\mu L\ L/(\mu L\ d_e)$	I_δ	δ
$\upsilon_s=0{,}15$	1700,432	25,474	0,062	0,750	0,283	0,100	14,75
			1,802	0,139	0,332	0,742	109,27

Method 2

$L/d_e=13{,}4/40{,}08=0{,}334\rightarrow I_\delta=0{,}5$ (Fig. 2.10)

$\delta=73{,}62$ mm

Effect of soft layers (E_u=27,75 – 65,25 MPa for clay, 105 Mpa for sand)

H/L	I_p	$H_{k+}1/L_e$	I_{p1}	E_s'	$(I_k\_I_{p1})/E_s$	δ
1,2686	0,32	1,6418	0,315	22,2	0,000225	15,23
1,6417	0,315	1,9403	0,31	84	0,0000595	4,02
1,9402	0,31	2,9104	0,2	52,2	0,002107	142,49

Drained	δtotal					
	------	232,57	205,22	176,49	271,02	235,37

$\delta_{measured}$=185 mm

c) Equivalent Raft Method

P=906100 KN q_n=P/(B*L) $\delta_i=\mu_1\mu_0 qB/E_u$

For sand q_c=10 M_0=40 E_u=27,75 – 65,25 MPa for clay, 105 Mpa for sand

L	B	H	D	L/B	H/B	D/B	μ_0	μ_1	q_n	E	δ_i
88	38	8,07	8,93	2,315	0,212	0,235	0,96	0,06	270,96	200,25	2,96
97,31	47,31	5	17	2,056	0,105	0,359	0,95	0,03	196,91	27,75	9,56
103,08	53,08	4	22	1,942	0,075	0,414	0,94	0,01	165,60	105	0,79
107,69	57,69	13	26	1,866	0,225	0,450	0,93	0,06	145,84	65,25	7,19
											20,50

$D/(LB)^{0,5}=0,154 \rightarrow \mu_d=0,975$

$\delta_c = m_\upsilon\ \sigma_z H\ \mu_d\ \mu_g$

υ	$E_{middle\text{-}dr.}$	m_v	μ_g	z/B	σ_z/q	σ_z	δ_c
0,15	153,525	0,006169	0,85	0,106	0,91	246,57	10,17
0,2	22,2	0,040541	0,85	0,278	0,78	211,35	35,50
0,2	84	0,025	1	0,396	0,68	184,25	17,96
0,2	52,2	0,017241	0,85	0,620	0,55	149,03	27,68
							91,32

$\delta_T=\delta_i+\delta_c$ =111,83 mm

$\delta_{measured}$=185 mm

Table A.33: Measured and computed settlements for Ghent Grain Terminal (mm)

	Settlement (mm)						
	Set. Ratio	Equivalent Pier				Eq. Raft	Mea.
		d_{e1}		d_{e2}		Ave1	
		Met1	Met2	Met1	Met2		
υs=0,15	119,14	—— 232,57	205,22	176,49 271,02	235,37	111,83	185

Ghent Grain

Rs	119,14	Mea.	185
Pier	232,57 205,22 176,49		
Raft	111,83		

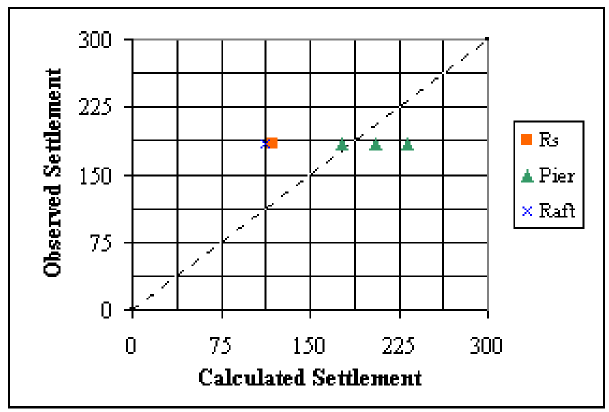

Figure A.63: Measured and computed settlements for Ghent Grain Terminal (mm)