AN EXPERIMENTAL STUDY OF VERTICAL AND INCLINED SOIL NAILS UNDER FOOTINGS AS SETTLEMENT REDUCERS

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ABSTRACT

AN EXPERIMENTAL STUDY OF VERTICAL AND INCLINED SOIL NAILS UNDER FOOTINGS AS SETTLEMENT REDUCERS

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Vertical and inclined soil nails under footings as settlement – reducing elements is investigated using a physical 1g model in the laboratory. Nails are not connected to footing, they are not so long and vertical settlement of nails is very large compared to usual limits encountered for piles or micropiles. Following the settlement of footing, they share the load together with the footing. The skin friction is mostly mobilized and end-bearing failure occurs continuously during the settlement. The system of footing- soil nail is studied by model square footings of 30 mm x 30 mm and 50 mm x 50 mm breadth dimensions and remoulded kaolin clay consolidated under constant controlled stress of 50 kPa in 200 mm cube boxes.

In the first section of the testing series 4, 5, 9 and 12 nails were inserted into soil in 3B, 2.4B, 1.33B and B lengths, respectively. In the second section, 4 and 6 nails in 1.5B and 2B lengths were tested for vertical and 15^0 and 30^0 inclined cases. Settlements of footings were measured under constant footing pressure for all groups. Several tests were repeated in each group of testing series.

It is concluded that keeping the total nail length constant, decreasing the nail number thus using longer individual nails is more effective in decreasing the footing settlements. 15^0 inclined nails decrease total settlements more.

Key Words: Soil Nails, Kaolin Clay, Model Test, Settlement.

ÖZ

ZEMIN ÇIVILERININ SÖMELLER ALTINDA OTURMA AZALTICI OLARAK DIK VE EGIK KULLANILMASININ DENEYSEL OLARAK INCELENMESI

ENGIN, Harun Kürsat

Yüksek Lisans, Insaat Mühendisligi Bölümü

Tez Yöneticisi : Prof. Dr. Mehmet Ufuk ERGUN

Ocak 2005, 98 sayfa

Dik ve egik zemin çivilerinin sömeller altinda oturma azaltici olarak kullanilmasi laboratuvar ortaminda 1g fiziksel model üzerinde arastirilmistir. Çok uzun olmayan zemin çivileri sömele yapisal olarak baglanmamakta ve dikey oturmalari kazik veya mini-kazik oturma sinirlarini fazlasiyla geçmektedir. Sömel ile birlikte oturan çiviler, temel yükünü birlikte paylasirlar. Sürtünme direnci tamama yakin mobilize olmustur ve oturma devam ederken sürekli bir uç dayanim göçmesi olmaktadir. Sömel - zemin çivisi sistemi 30 mm x 30 mm and 50 mm x 50 mm model kare temel boyutlarinda, 200 mm'lik küp kutulara yogrulduktan sonra yerlestirilen ve 50 kPa basinç altinda konsolide edilen kaolin kil kullanilarak incelenmistir.

Deney serilerinin ilk bölümünde, 4, 5, 9 ve 12’li çivi gruplari 3B, 2.4B, 1.33B ve B boylarinda zemine sokulmustur. Ikinci bölümde, 1.5B ve 2B boylarinda, zemine dikey, 15^0 ve 30^0 egik olarak sokulan 4’lü ve 6’li çivi gruplari test edilmistir. Temel oturmalari tüm gruplarda sabit temel basinci altinda ölçülmüstür. Her bir grup için çok sayida deney tekrarlanmistir.

Toplam çivi boyu sabit tutularak daha az sayida dolayisiyla daha uzun çivilerin kullanilmasinin sömel oturmalarinin azaltilmasinda daha etkili oldugu sonucuna varilmistir. 15^0 egik çiviler toplam oturmalari daha çok azaltmaktadir.

Anahtar Kelimeler : Zemin Çivisi, Kaolin Kil, Model Deney, Oturma.

To My Mother

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The author wishes to express his gratitude to *Prof. Dr. M. Ufuk Ergun* who expressed sincere supervision, guidance, advice, criticism and encouragement in every stage of this study.

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TABLE OF CONTENTS

LIST OF SYMBOLS

B	Foundation width
b	Radius for circular / half-width for strip foundation
C_L	Centre line
D	Depth of embedment
e	Eccentricity of load
E_F	Young's modulus of foundation material
E_S	Young's modulus of soil
$F_{Rv15}{}^o$, $F_{Rv30}{}^o$	Vertical resultant forces of inclined nail skin friction
f_s, q_s	Skin friction, unit shaft resistance
K_r	Non dimensional stress distribution factor
L	Pile, reinforcement length
L_e	Effective embedded length of nail
LL	Liquid limit
LVDT	Linearly varying displacement transducer
OCR	Overconsolidation ratio
P_f	Foundation load
PI	Plasticity index
PL	Plastic limit
P_w	Working load
P-δ	Load – displacement
q	Uniform foundation load
q_c	Contact stress under footing
R	Extension of reinforcement
R_s	Settlement reduction (%)
s	Reinforcement spacing

S_c	Consolidation settlement
S_f	Settlement of foundation
S_i	Initial settlement
S_{Nailed}	Settlement of model footing with nails
$S_{No\ Nail}$	Settlement of model footing without nail
S_r	Settlement of raft
SRPs	Settlement Reducing Piles
S_T	Total settlement
T	Thickness of foundation
W	Settlement of footing
w_{ac}	Water content after consolidation
w_{at}	Water content after testing
w_{bc}	Water content before consolidation
δ	Angle of friction on pile soil interface
δ_{crit}	Ultimate shaft friction in Settlement Reducing Piles
ϕ'	Effective friction angle
$\phi'_{triaxial}$	Angle of internal friction found from triaxial test
ν_F	Poisson's ratio for foundation material
ν_S	Poisson's ratio for soil
θ, β	Batter angle
σ_f	Footing pressure
σ_h	Horizontal consolidation pressure
σ_h'	Effective horizontal consolidation pressure
σ_v	Vertical consolidation pressure
σ_θ'	Effective angular consolidation pressure

1 INTRODUCTION

Man had used pile in order to overcome the difficulties of building foundations on soft soils since very old times. The design of pile foundations was primarily based on experience. Modern literature on piles is dated from the publication of Piles and Pile Driving, edited by Wellington of the Engineering News in 1893, in which the wide-known Engineering News pile-driving formula was proposed. From thereafter, a great volume of field experience and empirical data on the performance of pile foundations has been published. In recent years, to predict the behaviour of piles, more complex theoretical research had been performed (*Poulos & Davis, 1980*).

One of the most popular foundation type used for high-rise buildings is raft foundation. Rafts are suitable when an adequate bearing stratum is present at a shallow depth. Katzenbach and Reul (1997, cited in *Cao et al., 2004*) stated that the piled raft foundation is an innovative design concept to reduce both the maximum and differential settlements caused by concentrated structural loads and load eccentricities, and to reduce bending moments of the rafts. The concept of using piles as "settlement reducers" was proposed by Burland et al. (1977, cited in *Cao et al., 2004*).

In conventional piled raft design, the number of piles is normally large and the load carried by each individual pile is relatively small. There is a safety margin before the piles reach their ultimate geotechnical bearing capacities or structural failure load. However, when piles are considered as soil reinforcement or as settlement reducers, the ultimate geotechnical capacity could be fully mobilized. Randolph and Clancy (1993, cited in *Cao et al., 2004*) proposed that an efficient design of rafts with settlement reducing piles, the geotechnical capacity of the piles be assumed to be 80% mobilized under the working load condition. But this can lead to high axial stress in piles. Knowing most building codes and specifications impose very strict limitations on the allowable stress level in foundation piles, practical use of settlement reducing piles may be restricted as long as the piles are considered as structural members. In areas subjected to high wind or seismic loading, the situation is aggravated by the unacceptably high shear force that may be attracted by these relatively small number of piles that are usually connected to the raft, despite the fact that friction at the base of the raft and possibly soil pressures on the basements walls generally help resist the lateral loads.

One way of settling the problem of high stresses in piles is to disconnect the piles from the raft, hence consider these piles as subsoil reinforcement, rather then structural members (*Wong et al., 2000*). As the piles are reinforcement to the base soil, much lower factor of safety against structural failure can be applied to these piles without violating the building codes. Since the ultimate geotechnical capacity of disconnected piles can be assumed as fully mobilised, the structural considerations of such settlement-reducing piles are no longer critical in the design. Small differences and uncertainties in loads and material strengths may be ignored because these piles will not act as the main load carrying members. Even some small cracks in the piles may not significantly reduce their role of reinforcing the base soil. A factor of safety, as low as 1.3, against structural failure can be applied to the pile materials. Also number of piles are reduced compared to that are connected to the rafts in order to satisfy the structural requirements, with resulting economical benefits.

The use of soil nails is generally for slope stabilization as well as in-situ reinforced retaining structure. Another potential use of soil nails is under footings as settlement reducers (*Kul, 2003*). This type of use is very new and is different than micropiles support. Nails are not connected to footings, they are not so long and vertical settlement of nails is very large compared to usual limits encountered for piles or micropiles. Following the settlement of footing, they share the load together with the footing. The skin friction is mostly mobilized and end-bearing failure occurs continuously during the settlement (*Figure5.8*).

The present research work stems from partly *Kul*'s (2003) series of tests on model footings on inclined nails which were not comprehensive enough, and secondly from the interesting question faced in the design: nail length vs. number when the total length of nails is constant.

In this study, the behaviour of a model footing carrying a constant vertical load in clay, supported by nails of different length, number and inclination is investigated to find out the optimum pattern and length. The soil sample, which intended to resemble a semi-infinite elastic half space, is prepared by consolidating kaolin clay under 50 kPa constant pressure in cubical Plexiglas boxes. Settlement of footings supported by different nailing patterns has been observed until more than 90% of the consolidation is completed. *Chapter II* gives information about the topic. *Chapter III* outlines the experimental work. Preparation of soil samples, consolidation process, testing program, preparation of nails, instrumentation and monitoring all are described in this chapter. In *Chapter IV* the results of all model tests performed are presented in order. In *Chapter V*, the interpretation and discussion of test results are presented and major conclusions are drawn.

2 LITERATURE REVIEW

2.1 Distribution of Contact Stress over Footings

In practice, no foundation is perfectly flexible nor is infinitely rigid. The actual profile of the contact stress distribution depends on the elastic properties of the foundation and the soil on which the foundation rests.

Borowicka (*Das, 1983*), analyzed the distribution of contact stress under uniformly loaded circular and strip rigid foundations resting on semi-infinite elastic mass. He assumed the shearing at base to be zero. In his analysis, he found out that the stress distribution depends on a non-dimensional factor, K_r which is:

$$K_r = \frac{1}{6}\left(\frac{1-n_S^{\ 2}}{1-n_F^{\ 2}}\right)\left(\frac{E_F}{E_S}\right)\left(\frac{T}{b}\right)^3$$

where; ν_S : Poisson's ratio for soil

ν_F: Poisson's ratio for foundation material

E_F: Young's modulus of foundation material

E_S: Young's modulus of soil

b= radius for circular / half-width for strip foundation

T = thickness of foundation

Figure 2.1 shows the distribution of contact stress for circular foundation. $K_r = 0$ indicates a perfectly flexible foundation, whereas $K_r = 8$ means a perfectly rigid foundation. At the centre, contact stress over load per unit area, q_c / q is equal to 0.5. In rigid foundations q_c approaches infinity at the edges.

2.1.1 Foundations on clay

A rigid foundation on clay shows a uniform settlement with uniformly distributed load. The contact stress distribution will be as in the *Figure 2.1*. But at the edges, the contact stress cannot be infinity. Soil is not an infinitely elastic material; after a certain limiting stress ($q_{c(max)}$), plastic flow will begin.

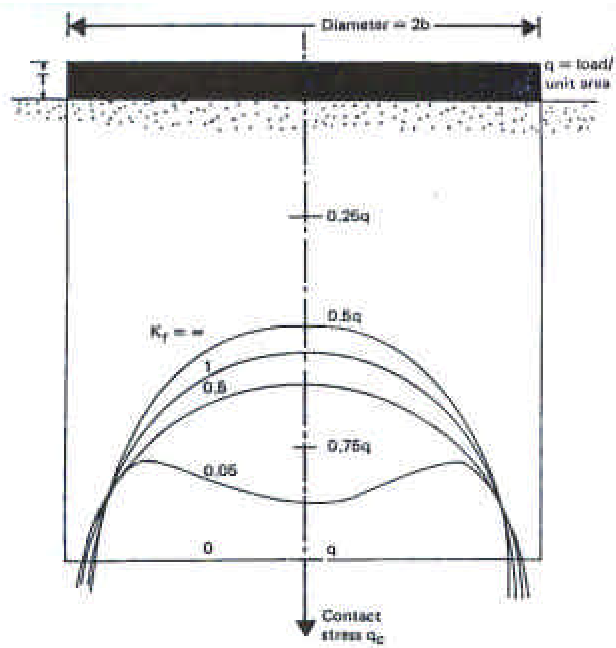


Figure 2.1 Contact Stress over Rigid Circular Foundations Resting on an Elastic Medium (*Das, 1983*)

2.2 Pile Foundations

Piles often provide the only foundation system that can reliably transmit loads to the foundation soils in cases involving weak surficial deposits (e.g., soft clays) and hostile environments (e.g., offshore). Most existing methods of pile design start with estimating the axial capacity of a single pile. When no competent end-bearing layer exists, the point resistance is relatively small and the major portion of pile capacity is derived from skin friction along the shaft. Thus, the limiting skin friction, f_s that can be provided by the soil is of most importance in pile design.

2.2.1 Stress Bulbs of Piled Foundations

In an experimental study by Terzaghi it is found that the pile lengths should be long enough (i.e. at least 1.5 times the breadth of the foundation) to contribute soil improvement. Otherwise, due to the disturbance during pile driving, it may also be detrimental. Therefore, the length of nails should be sufficient enough for an effective improvement of soil.

In *Figure 2.2*, for the narrow building case A, the piles effectively lowered the stress bulbs. Whereas in a wide building case B, piles of same length for the case A could lower the stress bulbs very little, practically not. Moreover, as stated in the former paragraph, the intended improvement had failed and it may even come out to be detrimental. In *Figure 2.3* the effect of piles and pile length in terms of lowering the stress bulbs is shown.

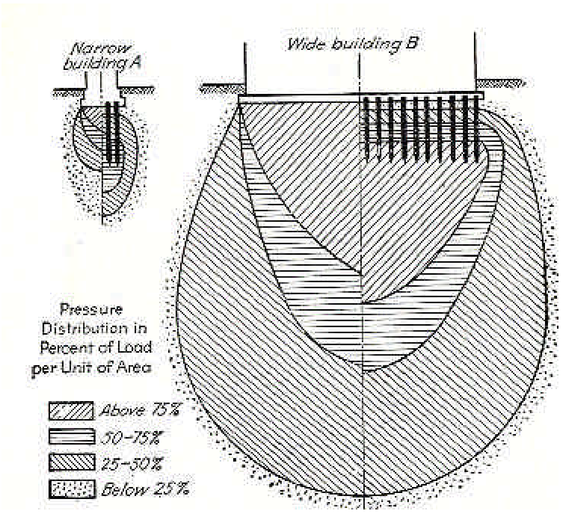


Figure 2.2 Effect of Relation between Foundation Width and Pile Length (*Terzaghi and Peck, 1948*)

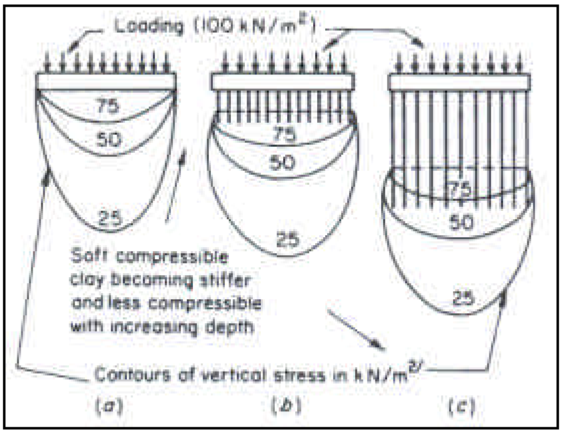


Figure 2.3 Comparison of Vertical Stress Distribution between (a) Surface Raft, (b) Raft with Short Piles, (c) Raft with Long Piles (*Tomlinson, 1995*)

In *Figures 2.4.d - 2.5* merging of stressed zones of piles can be seen. In this case the soil may not bear this stressed zone. Therefore, if the soil is incapable of carrying the load intensity coming from vertical piles, it is recommended to use batter piles in order to separate stress bulbs instead of having a single bulb of high intensity (*Figure 2. 4. b*). This is useful if the soil has adequate cohesion and friction to prevent the piles from bending.

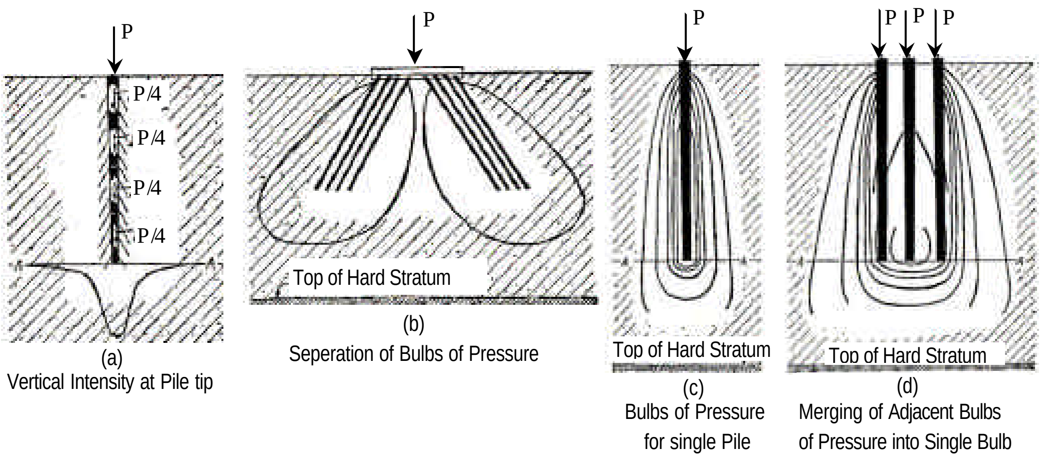


Figure 2.4 Friction - Pile Stress Distribution in Soil (*Chellis, 1961*)

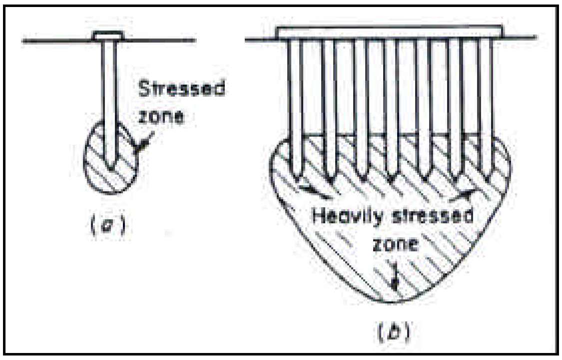


Figure 2.5 Comparison of Stressed Zones Beneath Single Pile and Beneath Pile Group (a) Single Pile, (b) Pile Group (After *Tomlinson, 1995*)

2.2.2 Friction Piles

Use of skin friction piles in a very deep soft deposit of fairly uniform consistency is for the purpose of reducing the intensity of pressure acting at the ground level and shifting the zone of maximum stress to the lower portions, where less settlement will be caused, and is governed by the basic principle: Settlement produced by a uniform load increases in proportion to the diameter of the loaded area for cohesive soils. Settlement under a unit load decreases with increasing depth of foundation. However, in addition to depending upon the depth of the foundation, settlement also depends upon the ratio of depth to diameter of the loaded area, so that for equal potential settlement reductions, the depth-to-diameter ratios should be kept equal. This principle indicates that the value of the piles may be greatly affected by the relation of their lengths to the width of the loaded area. Under a narrow structure, every effort should be made to keep the piles longer than the width of the structure, so that the lowered bulb of pressure will occur.

The studies indicated that even the adequate evaluation of existing methods is difficult to achieve, due to the complicated nature of pile-soil interaction and its dependence on many factors including site conditions (stratigraphy, soil properties, water table, etc.), pile characteristics (diameter, length, material, surface roughness, etc.), installation procedures (closed versus open ended, time history of driving versus jacking), and loading conditions (set-up time after driving, rate of axial loading, etc.). As a result studies conducted on empirical methods could not establish their reliability under general site and pile characteristics, driving and loading conditions. Instead, efforts were limited to comparing predictions of various methods with one another and/or with results of pile load tests.

2.2.2.1 Skin (Shaft) Friction

Friction values depend on type of soil, depth in the ground, degree of natural consolidation and saturation, shape of the pile, amount of compaction by pile, surface texture of pile, and on the time interval between driving and testing.

Mochtar and Edil (1988) carried out an experimental study for the investigation of shaft resistance of a model pile in clay. They found that the vertical consolidation pressure and overconsolidation ratio had no effect on pile-clay friction angle. Also pile length had no influence on angle of pile-clay friction. The surface roughness clearly affected pile-clay friction angle. Also it was concluded that the pile-clay friction angle is approximately equal to the Hvorslev true friction angle of the clay for smooth surface piles and the effective friction angle for the rough piles.

The ultimate displacement of the model pile consists of an elastic displacement and a slip displacement. The magnitude of displacement to mobilize the maximum shaft resistance is relatively small and dependent on the length of the pile soil contact surface.

The relative amount of load compared with failure load is a factor. Upper strata pick up the smaller loads, and no load may reach the lower strata or tip until large loads are applied, possibly beyond the range of working loads.

They also found out a linear relationship between unit shaft resistance and horizontal consolidation pressure σ_h (*Figure 2.6*).

It is found that there is no effect of vertical consolidation pressure, σ_v and overconsolidation ratio, OCR. In general, the length of pile soil contact does not exert a significant influence on the value of tan δ.

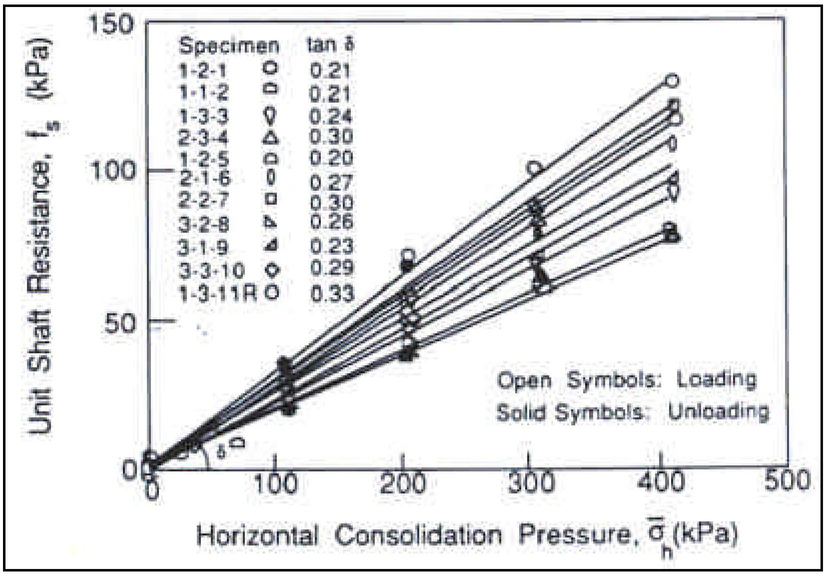


Figure 2.6 Unit Shaft Resistance versus Horizontal Consolidation Pressure (After *Mochtar* and *Edil, 1988*)

An experimental study by *Chandler and Martins (1982)* was carried out through load tests on a model pile installed in Speswhite kaolin to investigate the skin friction around the pile. The soil surrounding the pile was consolidated under a range of stress ratios K, and the pile was loaded under drained conditions such that only the shaft resistance was generated.

The model pile tests were also simulated by finite element techniques, using Modified Cam Clay as the constitutive law. The results of the analysis showed that on loading, conditions down the length of the pile are significantly uniform with some end effects extending for a few millimetres at top and bottom (*Figure 2.7*).

It was found out that the mobilized angle of friction at the pile-soil interface appeared to be slightly less than $\phi'_{triaxial}$, and was independent of the initial stress ratio in the soil. The radial stress varied during pile loading, reducing in the case of normally consolidated soil and probably increasing in the case of overconsolidated soil. Therefore, it would be unwise to use the radial stress predicted immediately before pile loading as the value applying at peak shaft friction. It is also claimed that there was a narrow shearing zone around pile, which enabled the excess pore water dissipation quickly, and hence the most pile loadings would tend to be fully drained conditions. Finally, it is concluded that the brittle post-peak load behaviour of the model piles indicated a strong possibility of progressive failure in practice. The degree of brittleness would depend on the method of pile installation and on the nature of soil: less for sandy boulder clay than very plastic clay.

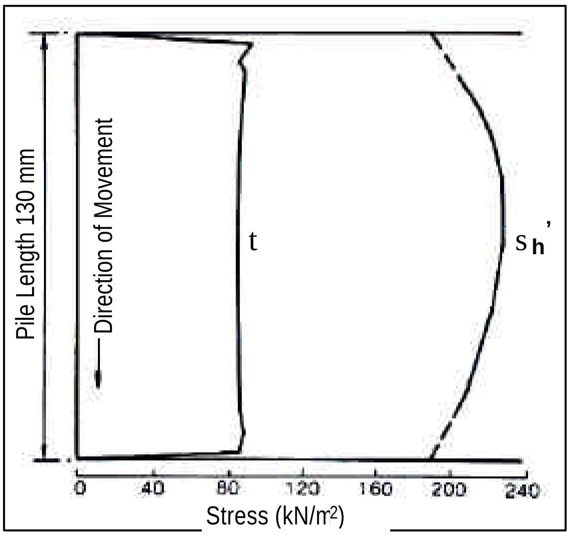


Figure 2.7 Computed Radial and Shear Stresses along the Model Pile at an Axial Load of 75% of Peak; K = 1.5; the Initial Values of σ_H' and σ_θ' are Assumed Equal (*Chandler&Martins, 1982*)

2.2.2.1.1 Effect of Soil Type on Skin Friction

Unit value of skin friction for a pile in soft clay depends upon the properties of the clay. Point resistance is negligible in soft clay. While driving resistance may remain small and be practically constant with depth, skin friction that will develop varies, in general, with depth. The rate of transfer of load to soil is low for shallow depths. Unit value of skin friction in soft silt is low during driving, because of liquefaction, but within a few days or weeks the silt apparently regains its original strength.

The unit value of skin friction for a pile in clay may vary widely for the same clay, depending upon the method used in placing the pile. Driving may have remoulded the soil to such an extent that the original structure has broken down and the clay has become more plastic around the pile. For a pile poured in a bored hole, hydrostatic pressure may prevent. The time factor is important because bearing capacity is likely to increase with time, as the water dissipates and the clay structure re-forms itself.

Jaime et al (1990) carried out some pile load tests (quick and slow penetration tests) on Mexico City Clay from which they concluded that:

1. The maximum load capacity in penetration of a pile embedded in Mexico City clay depends on the applied load rate: the higher the rate, the higher the capacity (i.e. the maximum load capacity in tests lasting about a half minute is at least 1.5 times the one obtained in a test lasting about 24 hours (*Figure2.8*).

2. The average slope of P-δ curve increases with the rate of applied load. The slope measured in quick tests is about 1.6 times than that of the slow tests.

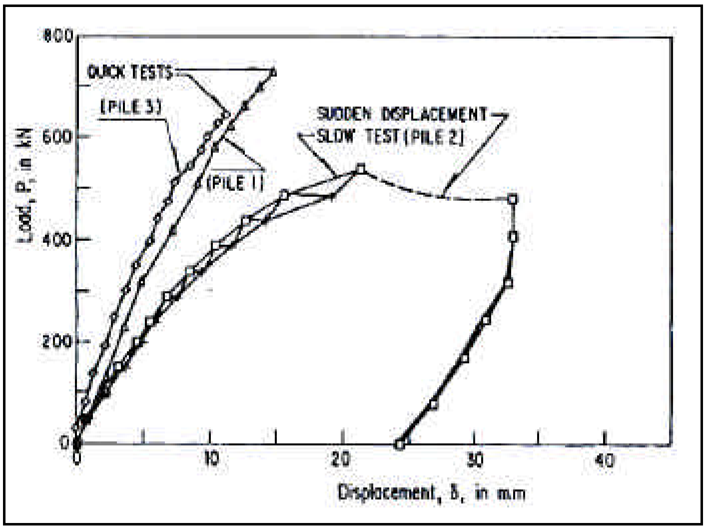


Figure 2.8 Slow- and Quick-Penetration Test Results (After *Jaime et al, 1990*)

Blanchet et al. (1980) carried out a detailed load test program on friction piles, on the north shore of the St. Lawrance Valley, during the construction of heavy structures on soft clays was performed to investigate the most suitable type of pile and to study its long term behaviour.

The results obtained from this testing program show that the pore pressures induced by *pile driving* are related to the pre-consolidation of the clay and that is much greater for tapered piles. The magnitude of excess pore water pressure depends on the distance to the pile, but apparently not effected by the group size. Full dissipation of the excess pore pressure required about 30 days around a single pile and up to 300 days in large pile groups.

For straight-walled rough piles (timber or concrete), they suggested that δ can be taken equal to the effective friction angle of the clay, ϕ'; for steel piles δ is reduced to tan δ = ¾ tan ϕ'.

The total settlements of the monitored pile groups were mainly due to the reconsolidation of the clay as the driving pore pressures dissipate and shear creep deformations in the clay close to the pile wall.

2.2.2.2 Settlement Reducing Piles (SRPs)

SRPs are a type of friction piles but they differ such that they operate in large displacements in order to mobilise ultimate shaft friction, δ_{crit} (*Figures 2.9- 2.10*). The load – deflection behaviour should be as near as possible to that shown by solid line in *Figure 2.10*, but may deviate to upper or lower bound in real case.

The factor of safety of the piles is unity as a consequence of SRPs operating at ultimate shaft friction and having little or no end-bearing capacity. This is acceptable since this type of piles are settlement limiters or stress reducers (*Love, 2003*).

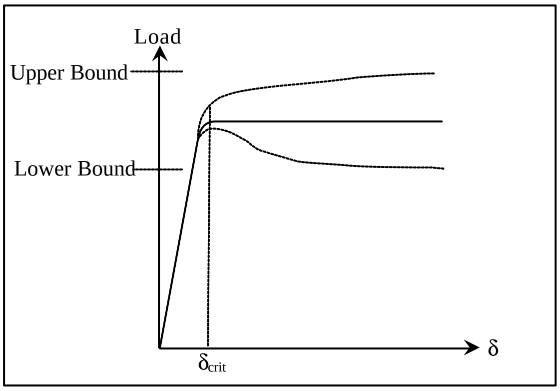


Figure 2.9 Ideal SRP Load-Deflection Response Shown by Solid Line (After *Love, 2003*)

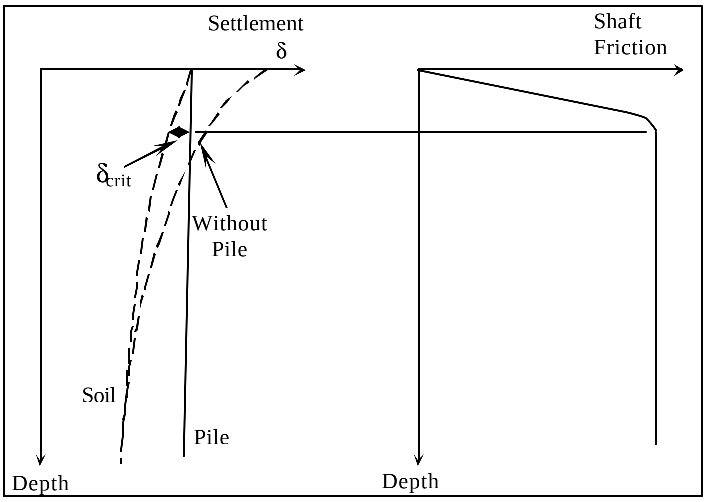


Figure 2.10 Concept of SRPs (After *Love, 2003*)

2.2.3 Batter Piles

Batter piles are generally used when lateral loads are so large that the vertical piles cannot withstand (*Figure 2.11*). They are placed, inclined such that the resultant external forces act on the pile axially. There are some experimental studies to find the effective batter angle to resist the lateral loads. The bearing capacities and settlement behaviour of the batter piles were also investigated.

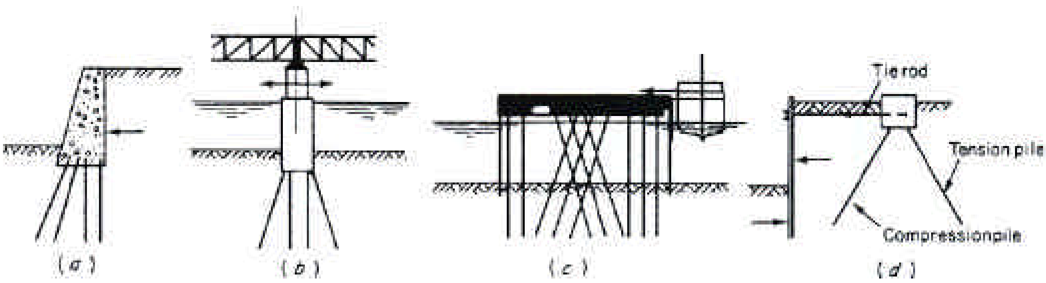


Figure 2.11 Uses of Batter Piles. (a) Retaining Wall, (b) Bridge Pier, (c) Wharf, (d) Sheet Pile Retaining Wall (*Tomlinson, 1995*)

A batter pile has an improved resistance to lateral loads, since the large portion of the horizontal component of lateral forces is carried axially. Negative and positive batter piles are as described in *Figure 2.12*.

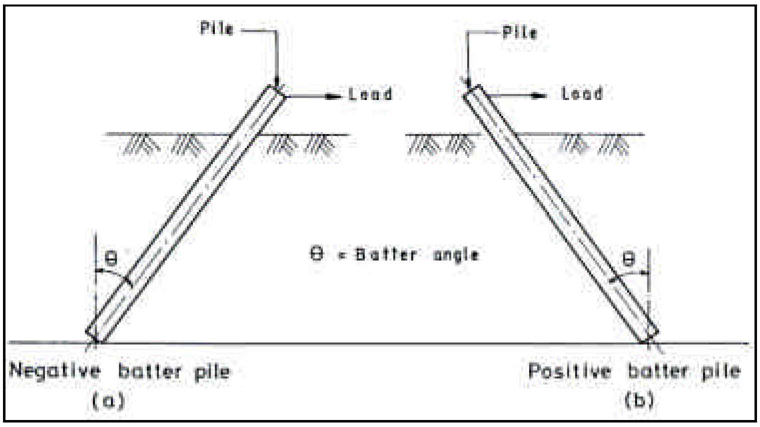


Figure 2.12 Types of Batter Piles (After *Rao, 1994*)

Rao et al. (1994) had shown in an experimental study on model batter piles in clay under lateral loads that negative batter angle gives more resistance compared to vertical and positive batter angles (*Figure 2.13 and 2.14*). It was also verified in this study that this trend holds true for different embedment ratios (L/D). As expected, the lateral resistance increased with more embedment (*Figure 2.15*).

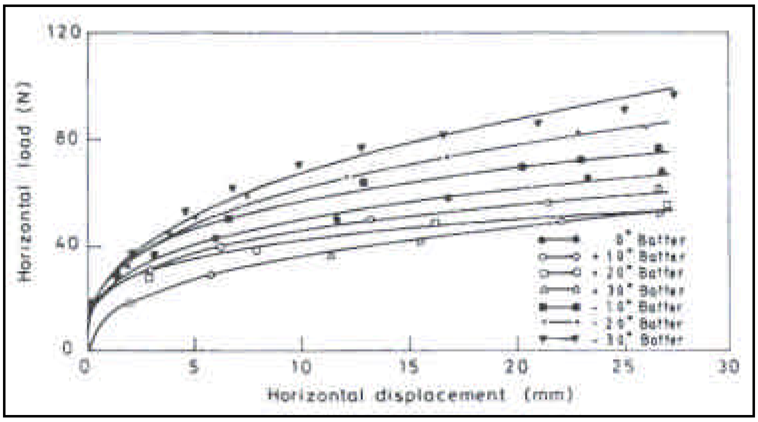


Figure 2.13 Lateral Load vs. Displacement Curves (After Rao, 1994)

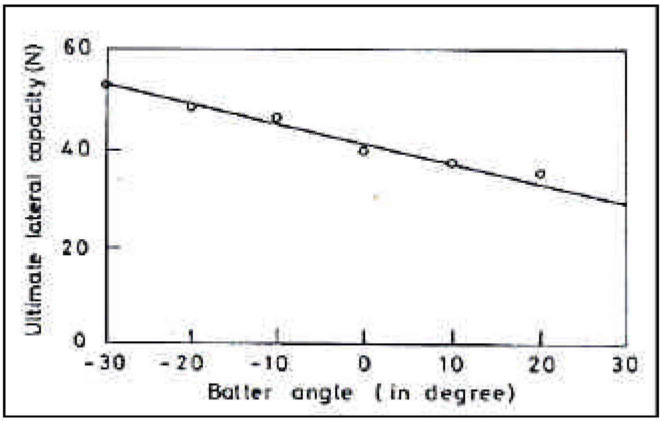


Figure 2.14 Ultimate Lateral Capacity Batter Angle Relations (After *Rao, 1994*)

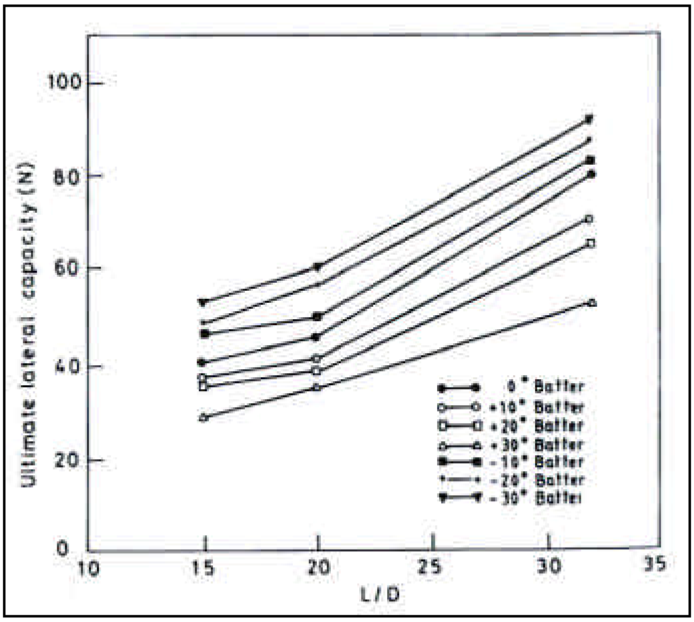


Figure 2.15 Ultimate Lateral Capacity Variations with Embedment Ratio and Batter Angles (After Rao, 1994)

Yalçin and Meyerhof (1993) has studied the displacements of flexible vertical and batter model piles under eccentric and inclined loads in layered soil of clay overlying sand. In continuation of this work, the behaviour of single free-head model flexible vertical and batter piles under general case of eccentric and inclined loads in two-layered soil was also investigated (*Meyerhof and Yalçin, 1994*). They found out that the bearing capacity of piles depend on the layered structure, the eccentricity and inclination of the load and pile batter. The ultimate loads of vertical and batter piles decreased rapidly with increasing eccentricity. The maximum capacity of the batter piles was reached under axial loads regardless of the eccentricity and embedment ratios (e/D and H/D). The minimum capacity was developed under horizontal loading. The capacity of negative batter angle $\beta = -30^0$ was found to be more than positive batter angle $\beta=+30^0$ at all e/D and H/D ratios.

2.2.4 Micropiles

Babu et al. (2004) used micropiles of 100 mm diameter and 4 m long to improve the bearing capacity of foundation soil and in the rehabilitation of a foundation system. The micropiles were inserted around the individual footings at an inclination of 70^0 with the horizontal (*Figure 2.17*). The actual design was based on the assumption that the vertical component of the frictional force between the soil and the micropile resists the additional load coming from the structure and above the bearing capacity. The technique was successful.

Detailed finite element analysis was conducted to examine this case study in terms of its overall performance. *Figure 2.16* shows the load displacement response obtained from numerical simulations. Curve 1 is the load displacement curve obtained for the in-situ soil. Curve 2 is the load displacement curve obtained for micropiled case. Curve 2 shows the overall improvement in bearing capacity obtained.

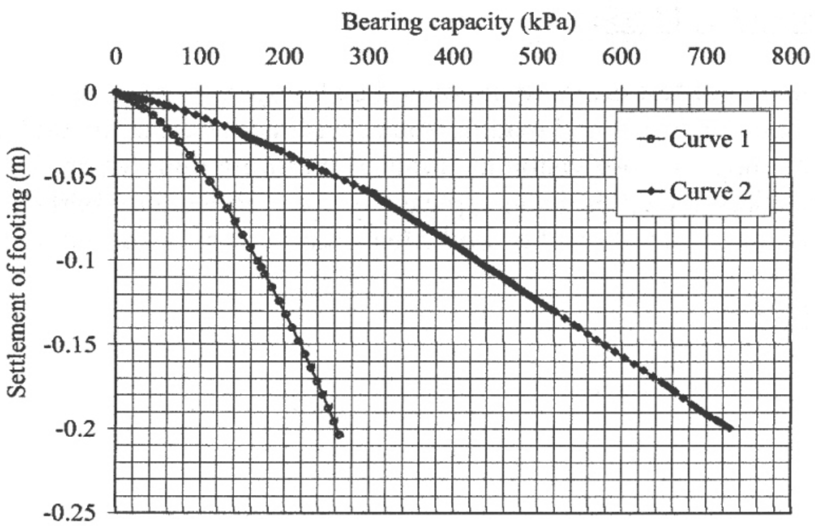


Figure 2.16 Load-Settlement Curves with and without Micropiling (After *Babu et al., 2004*)

Densification of soil surrounding micropiles and the frictional resistance between the micropiles and the foundation were considered in the numerical analysis. The results confirm that the methodology used was effective in obtaining the desired level of improvement.

Kalkan (2004) carried out a series parametric study based on nonlinear three-dimensional finite element models to evaluate the influences of geometrical parameters on the global load transferring patterns, bearing capacity and deformability of the raft foundation. The local stress concentrations and stress flow were examined under static loading condition using a discrete finite element simulation of the pile-cap and micropile.

The major parameters considered were the rake angle of micropiles, pile configuration and their spacing in the pile-cap. The pile layout and the batter angles for different test cases are given in *Figure 2.18* and *Table 2.1* respectively.

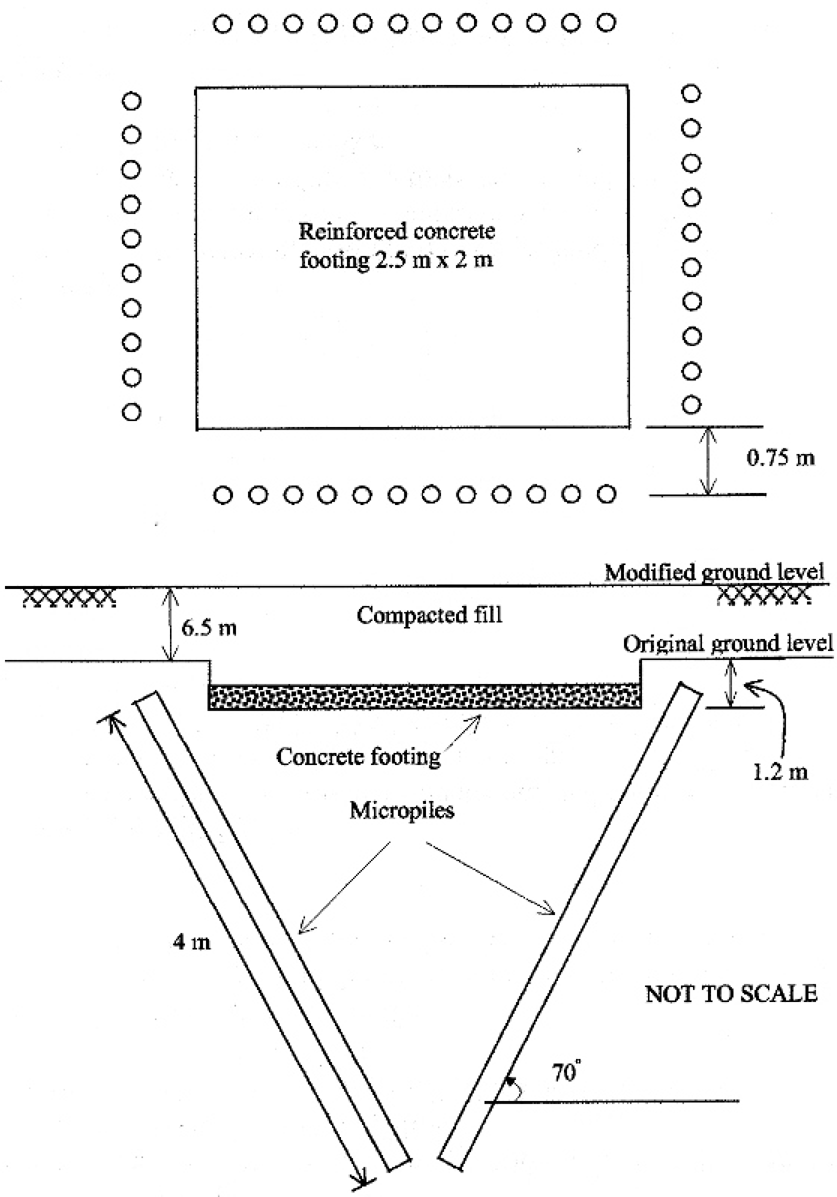


Figure 2.17 Plan and Cross Sectional View of Footing Strengthened by Micropile System (After Babu et al., 2004)

It was concluded that the vertical response of the piles are irrespective of their orientation. After the critical load 13.34 MN outward piles carry more compared to the inward piles.

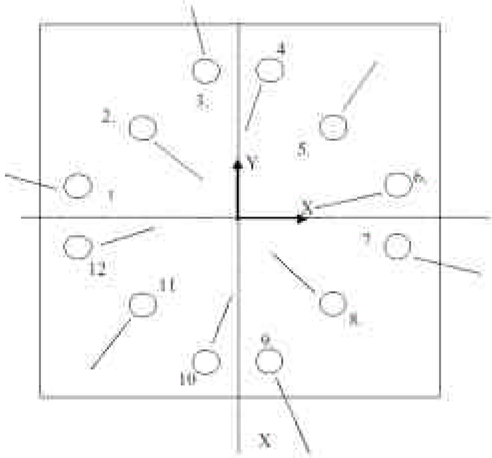


Figure 2.18 Pile Layout (After *Kalkan, 2004*)

Table 2.1 Batter angle of piles test cases (After *Kalkan, 2004*)

Case	Outward going pile angle (degrees)	Inward going pile angle (degrees)
1	0	0
2	15	15
3	20	15
4	25	15
5	30	15

In all cases the lateral load was employed after applying 10.68 MN vertical load (approximately 0.89 MN on each pile), which is 68% of the maximum axial load at which the reinforcement start yielding. This percentage might vary a little for different batter angle cases. In *Figures 2.19* to *2.21*, the results of the finite element analysis are given. In *Table 2.2* maximum lateral load capacities and displacement are presented.

As a conclusion, *Kalkan (2004)* found the significant effect of batter angle on the lateral load capacity. As the piles were spread wider in the soil, more lateral load could be resisted.

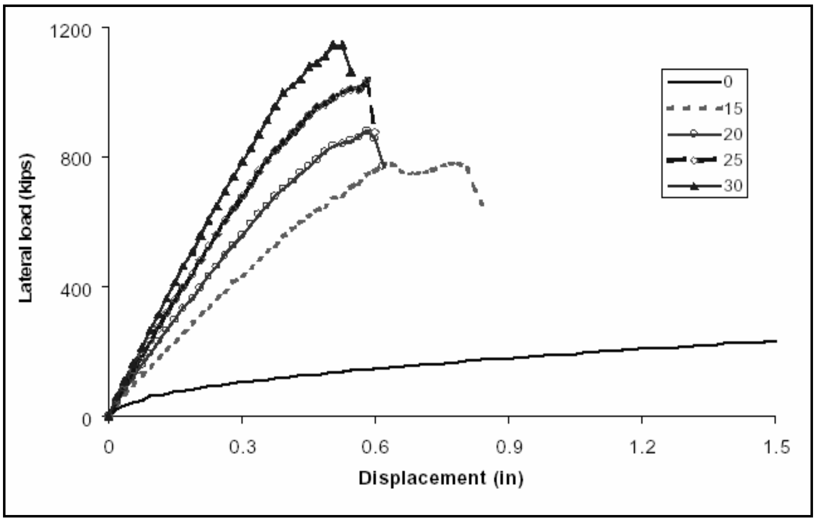


Figure 2.19 Lateral Load Capacity Curves for Different Batter Angles

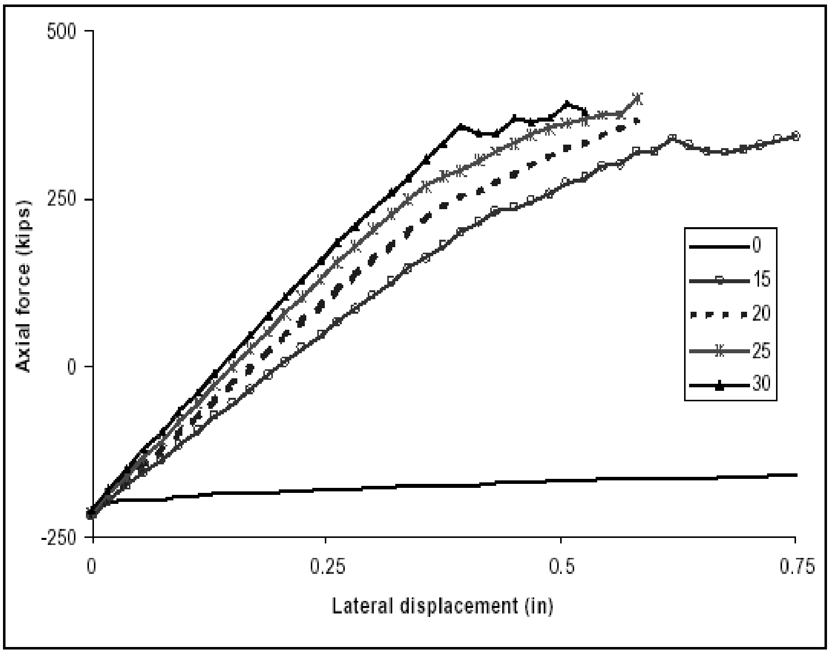


Figure 2.20 Axial Force Comparison in the Pile Experiencing Maximum Tension (Pile # 1) (After *Kalkan, 2004*)

Table 2.2 Comparison of Maximum Lateral Load and Corresponding Displacements for Different Batter Angle Cases (After *Kalkan, 2004*)

CASE (Outward angle in degrees)	0	15	20	25	30
Maximum lateral load (MN)	1.03	3.47	3.91	4.62	5.10
Lateral displacement (mm)	38.100	20.003	14.764	14.764	12.859

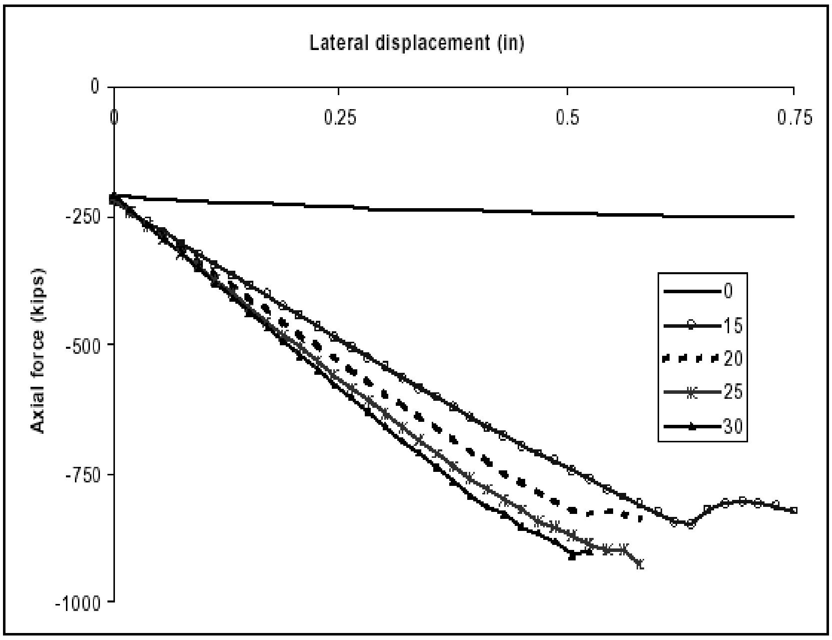


Figure 2.21 Axial Force Comparison in the Pile Experiencing Maximum Compression (pile # 7) (After *Kalkan, 2004*)

2.3 Raft Foundations with settlement-reducing piles

The essential objective in the design of piled raft foundations is proper judgement both on relative proportions of load carried by the raft and piles, and the effect of pile support on the maximum and differential settlements.

The use of piles as settlement reducers is an efficient way to reduce total and differential settlements if the raft alone has an adequate bearing capacity.

In conventional piled raft design, the number of piles is large compared to the load taken by each one. There is a high factor of safety for the pile before reaching to its geotechnical or structural capacity. The capacity of piles is generally governed by their geotechnical capacity. Due to the use of large number of piles in amount, the lateral resistance is generally adequate if the connections are designed accordingly.

If settlement-piles are designed as structural components, the settlements are often relatively large that the ultimate geotechnical capacity is fully mobilized. Randolph and Clancy (1993, cited in *Cao et al., 2004*) indicated that for an efficient design of rafts with settlement-reducing piles, pile capacity could be assumed to be 80% mobilised under working load. Thus, a lower factor of safety can be applied to the geotechnical capacity of the piles, since generally the bearing capacity of the raft is alone adequate. However, if settlement piles are connected to the raft, because of being less in number relatively, high axial stresses may develop in piles and their load carrying capacity will be governed by their structural capacities. Most building codes and specifications restrict allowable stress levels in foundation piles. So, the practical use of settlement reducing piles is restricted if considered as structural members. Also, settlement-reducing piles may not provide adequate horizontal resistance to lateral loads such as seismic and wind loads.

Since the main objective of using piles under rafts is for settlement control, using settlement piles as reinforcement to the base soils can be an economical and innovative design for the above mentioned problems compared to connected piles.

In *Figure 2.22* the concept of settlement-reducing piles is illustrated. For a working load of P_w the settlement of the raft, S_r may be excessive. At the same working load, addition of piles will decrease the settlement of the foundation, S_f. The number of piles required can be determined according to the purpose of controlling the settlements within acceptable limits.

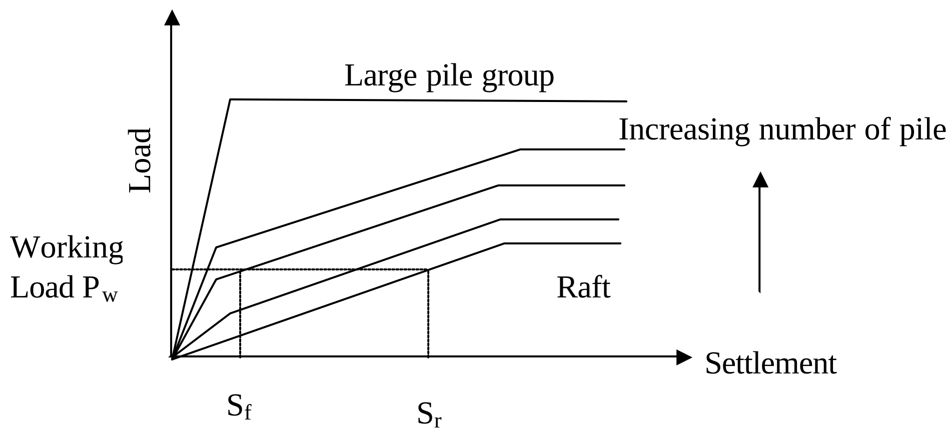


Figure 2.22 Concept of Raft-Pile Foundation System, (after *Poulos and Davis, 1980*)

When a raft has an adequate bearing capacity, piles should be used under raft as settlement reducers rather than main load-carrying elements. For the ideal case in which the piles reach their ultimate geotechnical capacity, the factor of safety for this capacity can be taken as 1.0. An increase in factor of safety of the piles may lead to over-design of the foundation, but with minimal additional effect in settlement reduction. Although a small number of piles is used, the performance of the raft—pile system would be satisfactory if the bearing capacity of the raft is adequate.

Therefore, with a more realistic analysis the piled raft behaviour may enable savings to be made in the number and size of piles and also, in the thickness of the raft.

The main function of settlement-reducing piles is to control the total and differential settlements of the raft. An alternative way is to design piles as sub- soil reinforcement by structurally disconnecting them from the raft. A gap can be provided between piles and the raft such that the piles would not carry the loads coming from the superstructure directly (*Figure 2.23.b*).

Disconnecting the piles from raft and treating them as subsoil reinforcement rather than structural members can avoid high stresses in piles (Wong et al., 2000). Thus, use of piles as settlement reducers is effective for controlling the total and differential settlements of a raft, which already has an adequate bearing capacity. In this way, a much smaller number of piles are often adequate than the ones calculated by conventional design methods for reducing the raft settlement to an acceptable limit.

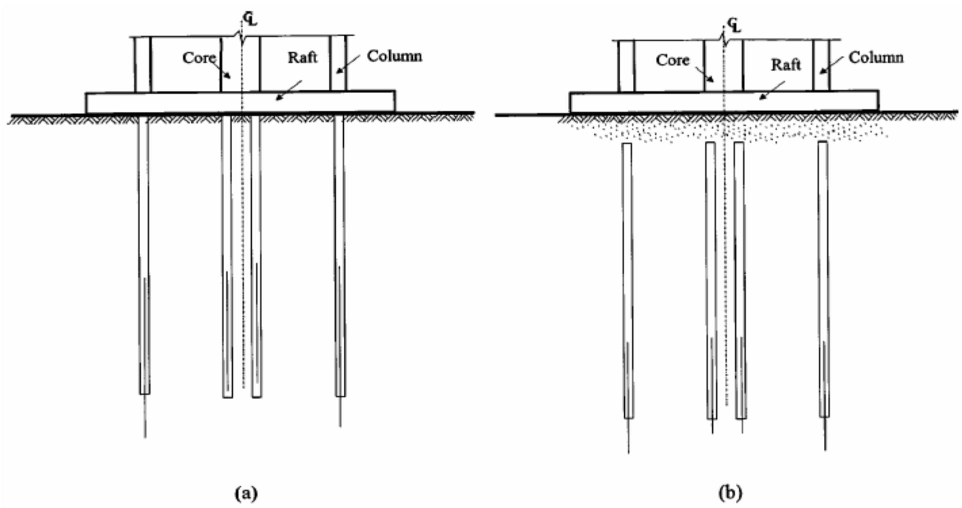
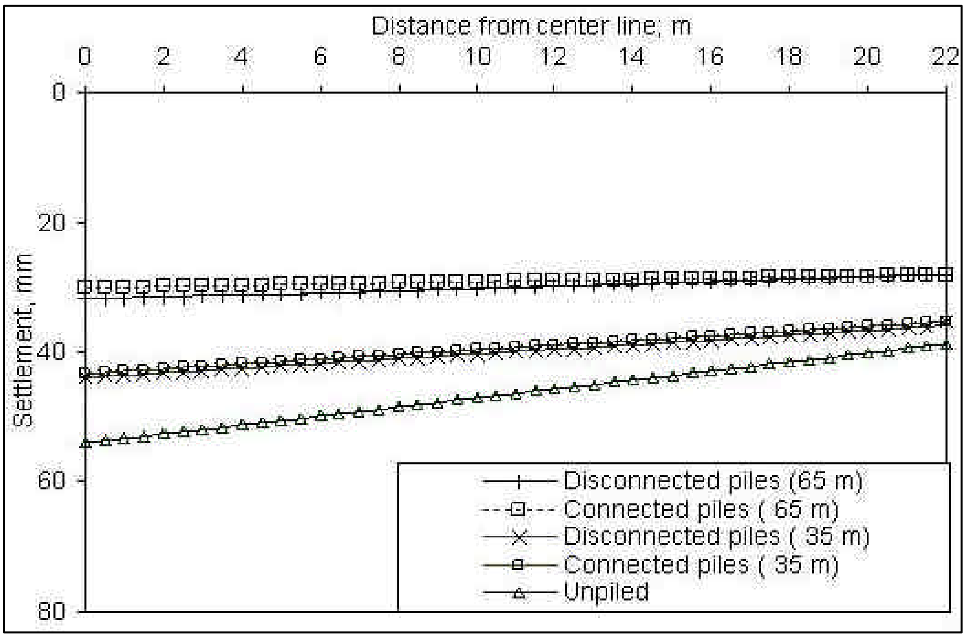
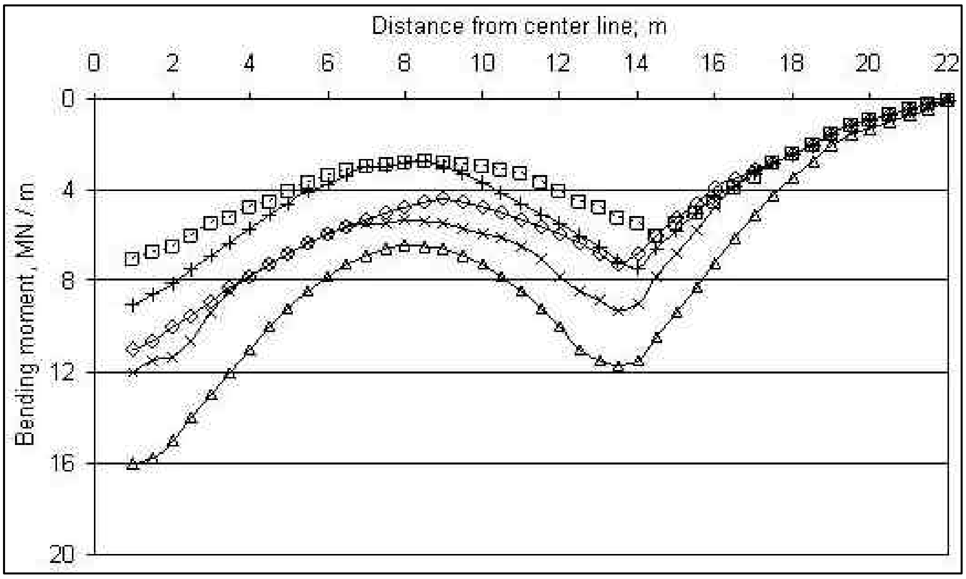


Figure 2.23 Piles as Soil Reinforcement (a) Structurally Connected; (b) Structurally Disconnected (After *Wong et al., 2000*)

Figure 2.24.b shows the distribution of bending moment along the raft for three different systems. Although the maximum bending moment in the raft with disconnected piles is slightly higher than the one with structurally connected piles, the effectiveness of introducing disconnected piles under the raft in terms of reduction in bending moments is clearly seen. It is obvious that the concept of disconnected settlement-reducing piles is worth studying.



(a)



(b)

Figure 2.24 Effect of Configuration of Settlement-Reducing Piles on Computed Behavior of Raft Foundation for Proposed High-Rise Building in Jakarta, Indonesia: (a) Settlement; (b) Bending Moment; (after Wong et al., 2000)

The structurally disconnected settlement-reducing piles may act more as soil-reinforcing members to stiffen the base soil rather than as direct load-carrying members. A part of the applied load would still be transmitted to the piles through the soil between the pile heads and the raft. Negative skin friction along the upper part of the piles may increase with the use of disconnected settlement-reducing piles.

Cao et al. (2004) verified the conclusions of former studies related to disconnected settlement piles experimentally. It is concluded that when settlement piles were added, the differential settlement and bending moment in the model rafts were found to decrease in sand. On the other hand, increasing the pile length for the same pile group was found to be effective for improving the stiffness of a pile-raft system.

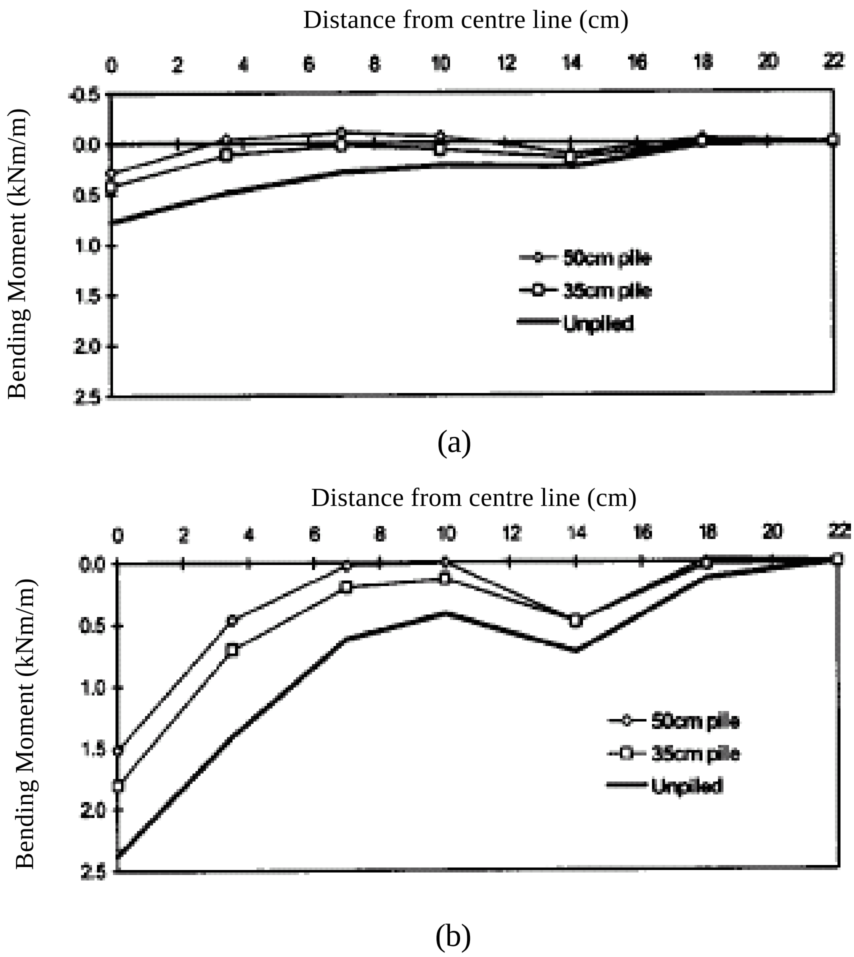


Figure 2.25 Distributions of Bending Moment of 25 mm Thick Plate at Two Levels of Average Applied Stress: (a) 60 kPa and (b) 260 kPa (After *Cao et al., 2004*)

Figure 2.25 shows that the inclusion of piles reduced the bending moments in the plates. It was also observed that the increase in pile length did not lead to a significant reduction in the bending moment.

2.4Reinforcing Elements for Improving Sand Subgrades

Basset and Last (1978) investigated the possibility of introduction of the reinforcing tendons aligned where possible with the principle tensile strain directions (at right angles to the principle compressive stress trajectories, *Fig. 2.26*) in a model test.

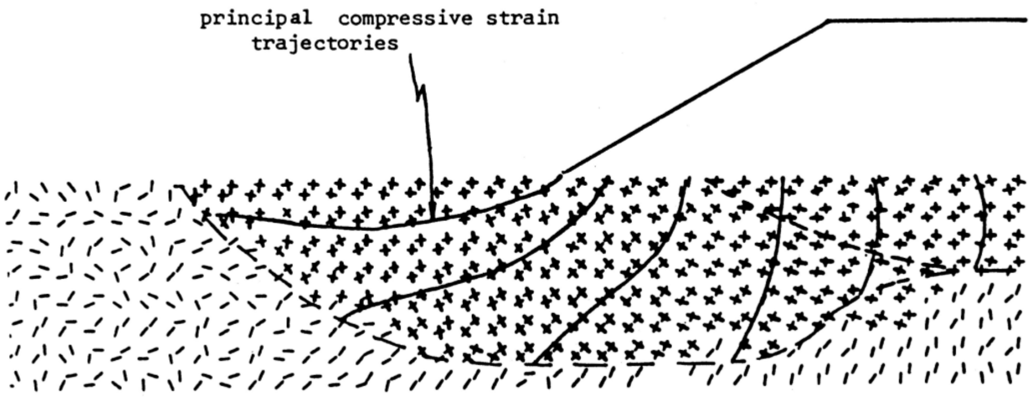


Figure 2.26 Principle Strain Directions (After *Basset & Last, 1978*)

The ideal and practical model reinforcement arrangements are given in *Figures 2.27, 2.28* and *2.29*.

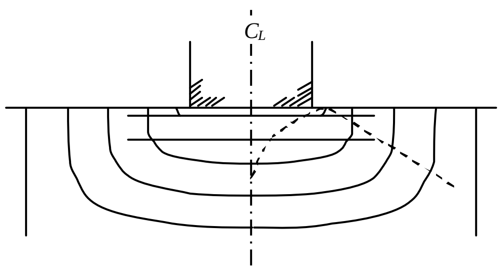


Figure 2.27 Ideal Arrangement of Reinforcement (After *Basset & Last, 1978*)

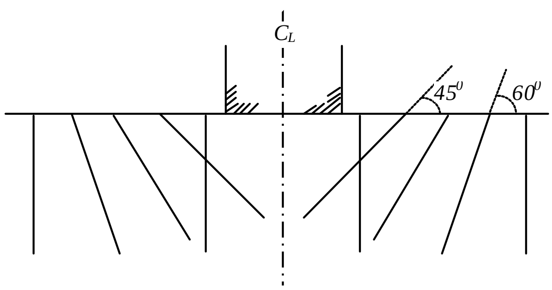


Figure 2.28 Practical Model Reinforcement-Pattern A (After *Basset & Last, 1978*)

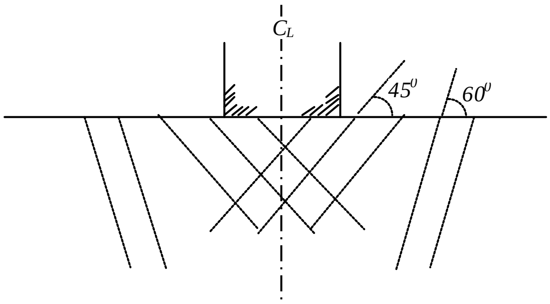


Figure 2.29 Practical Model Reinforcement-Pattern B (After *Basset & Last, 1978*)

Pattern A (*Figure 2.28*) was adopted in tests. Because it resembles the performance of an existing footing for improvement without disturbance and difficulties associated with underpinning. For a new construction Pattern B (*Figure 2.29*) was offered as the best compromise solution.

The reinforcement should be of sufficiently large diameter or closely spaced enough for the whole soil mass to be influenced (i.e. the maximum spacing should be 8D, 1% area ratio). The reinforcement displaced the distortion mechanism to greater depth hence increased the bearing capacity.

Improvement of load carrying capacity and settlement characteristics of sand subgrades with horizontal reinforcement under footing foundations has been studied and verified by many investigators. However the greatest disadvantage of the horizontal reinforcement is that it can not be used in existing conditions and the subsoil has to be re-laid and compacted after placing the reinforcing elements.

Verma and Jha (1992) made an experimental study in which vertical reinforcements were placed on all four sides of a model footing except directly under the footing (*Figure 2.30*). The improvement in bearing capacity ratios ($BCR=q/q_o$) and settlement ratios (SR=100(W/B)) were used to analyse the test data. It is concluded that subgrades could be improved without reinforcing directly below the footing and the bearing capacity was a function of spacing and extent of reinforcement (*Figures 2.31 and 2.32*). As expected, it was found out that the bearing capacity could be increased twice the un-reinforced case.

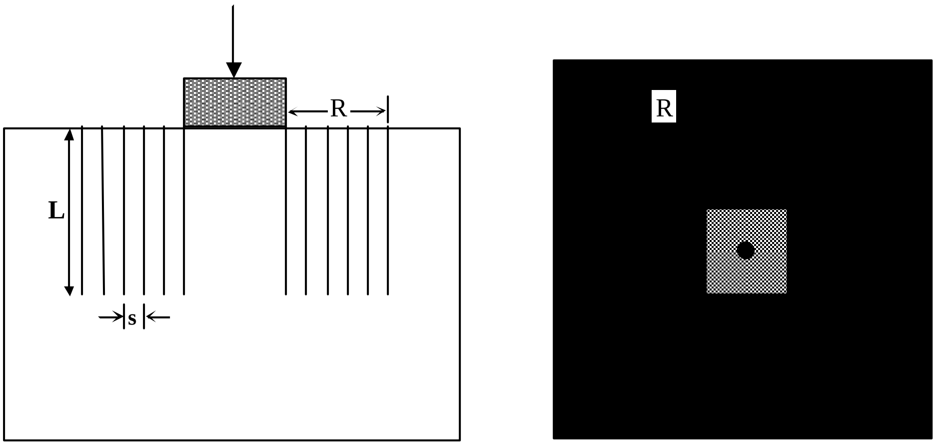


Figure 2.30 The Pattern of Reinforcements Used in the Tests *(After Verma and Jha, 1992)*

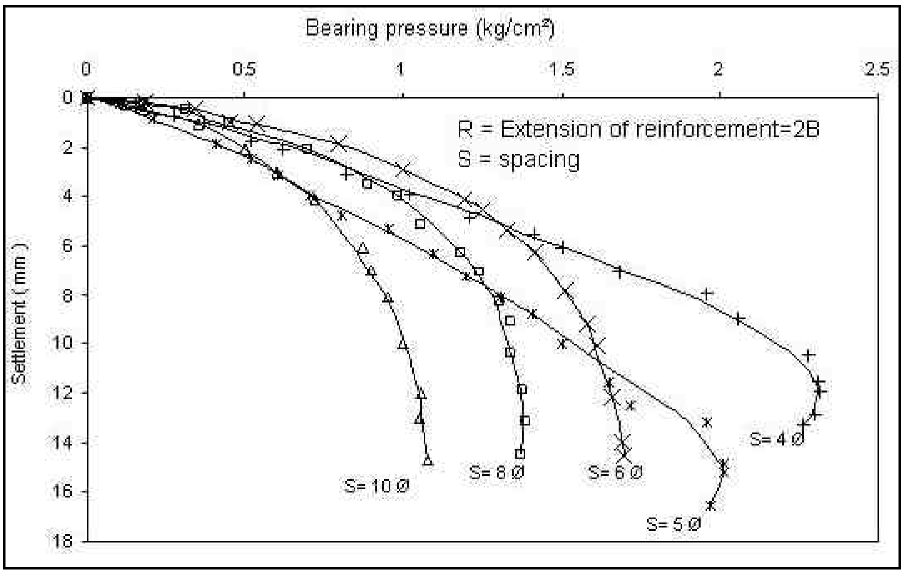


Figure 2.31 Relationship between Bearing Pressure and Settlement for Different Spacing of Reinforcement (L= 1.5B, R= 2B) (After *Verma and Jha, 1992*)

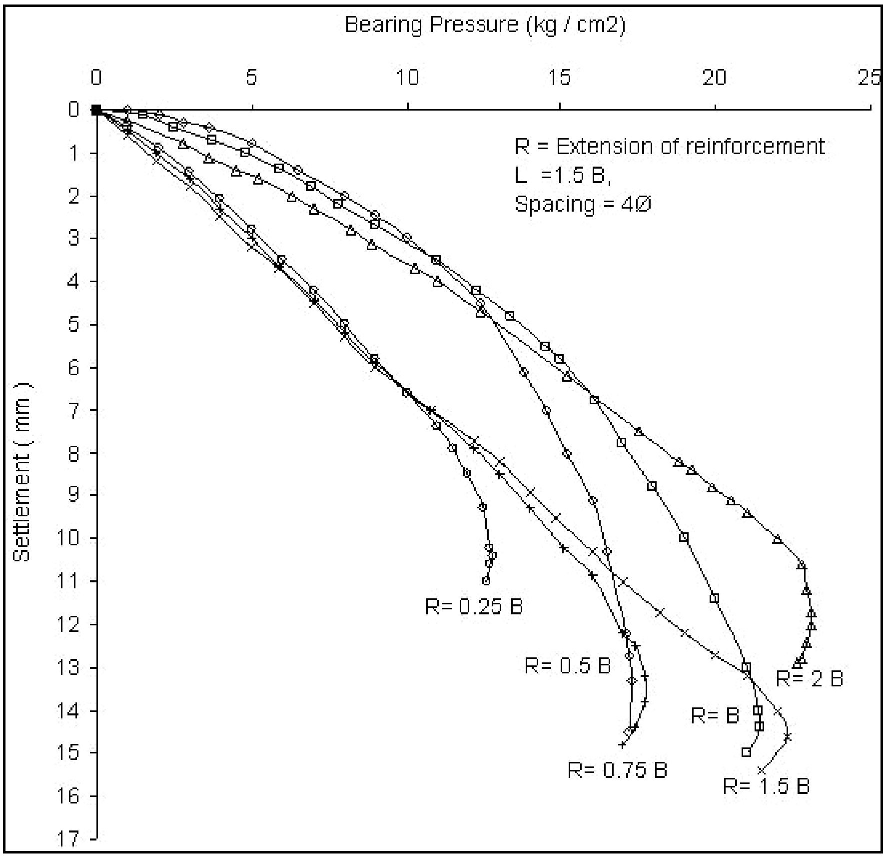


Figure 2.32 Relationship between Bearing Pressure and Settlement for Different Extent of Reinforcement (After *Verma and Jha, 1992*)

2.5 Use of Soil Nails as Settlement Reducers under Footings

Kul (2003) had studied footing-soil nail system with a model 30 mm x 30 mm square footing. In the testing series remoulded kaolin clay was consolidated under controlled stresses in 200mm cube Plexiglas containers. The prepared soil was then improved by using 4 and 9 nail groups of nails of B, 1.5B, 2B and 3B lengths inserted under footing (before placement of footing). Settlements of the footings were measured under a constant stress. Also nails which made 15^0 and 30^0 angle with the vertical were tested in limited numbers.

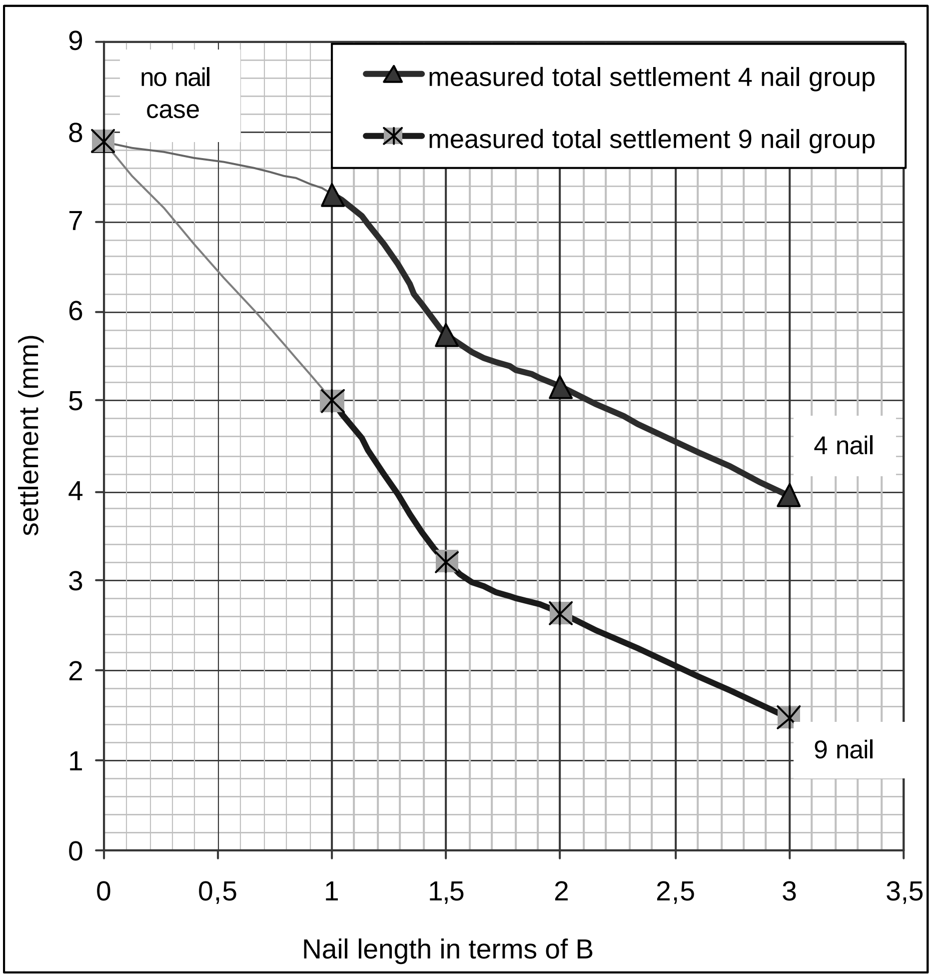


Figure 2.33 Settlement of Nailed Footings versus Nail Length in Terms of B (After *Kul, 2003*)

In *Figure 2.33*, the results were summarized. It was concluded that the rate of decrease in settlements with respect to increasing nail lengths is clearly greater up to 1.5B length. Use of longer nails than 1.5B length contributed settlement reduction in linear-like manner.

It was clearly observed that the use of nails under footings to limit settlements was a potential improvement technique. The effective minimum nail length was found out to be 1.5B. 15^0 inclined nails were found out to be more effective in reducing the settlements. (*Kul, 2003)*

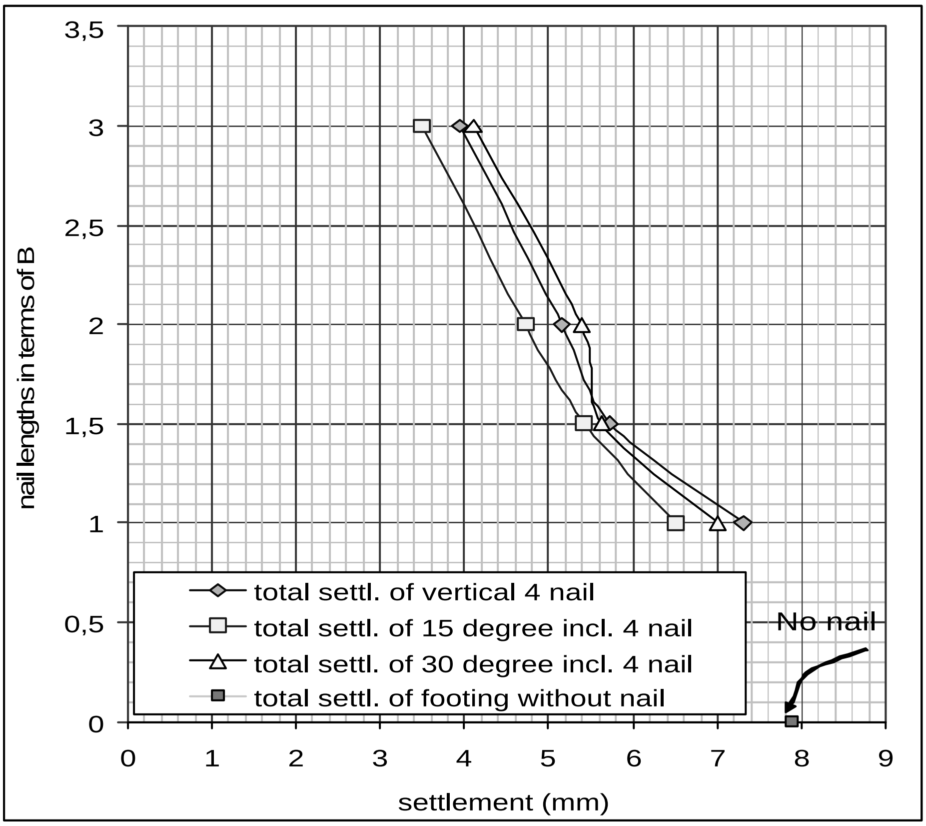


Figure 2.34 Total Settlement of Four Nails Groups in Vertical Direction, 15^0 and 30^0 Inclined vs. Nail Length in Terms of B (After *Kul, 2003*)

It was also found out that nails which make 15^0 with the vertical had consistently given the smallest settlements, even lower than the vertical nails in all series (*Figure 2.34*). Vertical nails longer than 2B length were more effective in settlement reduction compared to 30^0 inclined nails.

The experimental results were also verified with a two-dimensional finite-element program as an axis-symmetric problem. Results obtained in this routine were in agreement with the experimental results.

3 EXPERIMENTAL WORK

3.1 Experimental Setup

In this study, the effect of rigid (brass) soil nails as settlement reducers under a model footing in cohesive soil was investigated throughout an experimental study in the laboratory. For having a standard soil sample, remoulded kaolonite type clay was placed in the Plexiglas boxes, and consolidated under 50 kPa pressure. After the consolidation period for about a 3 weeks minimum, soil nails were inserted in pre-determined number, length and inclination. A model rigid footing was placed on the improved soil and settlement behaviour was observed under constant pressures.

The settlement readings are taken approximately for 3 days. After testing, representative samples are taken under footing and from all other parts of the tested specimen and the mositure contents are determined accordingly.

The laboratory model testing system consists of:

1. Plexiglas Boxes with 20x20x20 cm inside dimensions (*Figure 3.1.a*)
2. Geotextiles for drainage and prevention of drying (*Figure 3.1.b*)
3. Commercial kaolonite type of clay
4. Loading jack for the application of constant (consolidation) pressure (*Fig.3.2*)
5. Brass Nails (the sides of which are rasped for increasing frictional capacity) (*Figure 3.20*)
6. 30x30x10 mm, 50x50x10 mm (*Figure 3.5*) Aluminium Footings
7. Timber Templates for insertion of nails (*Figure 3.6*)
8. Load Hangers (*Figures 3.3, 3.29,3.33*)
9. Displacement Transducers, displacement dials (*Fig. 3.28.b, 3.29, 3.30*)

10. Data acquisition system consisting of:

    a. A computer (PC)

    b. 16-channel data logger (ADU)

    c. A software (DADU) arranging and recording the readings taken from transducers.

3.1.1 Testing Box Assembly

A Plexiglas testing box system, which was manufactured by *Kul (2003)*, was used for the purpose of loading the footing and measuring the settlements.

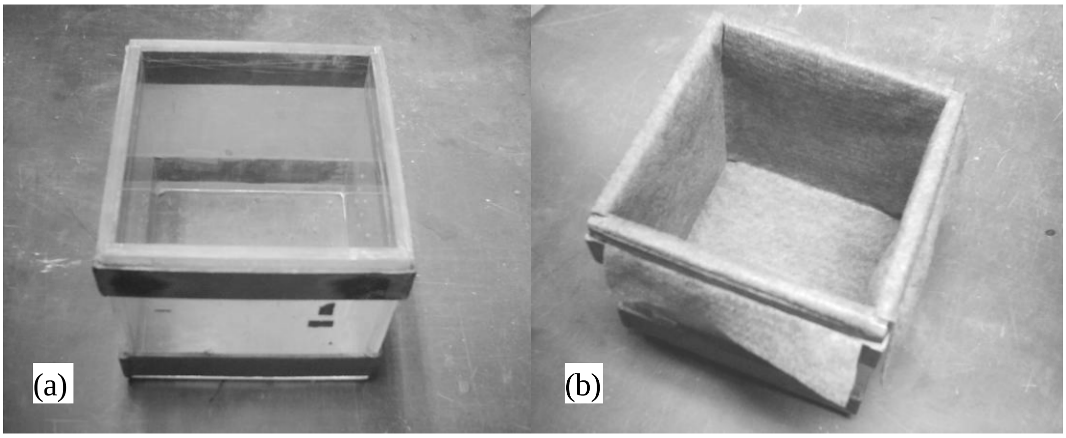


Figure 3.1 (a) Plexiglas Box Empty, (b) After Geotextile Sheets Placed

The box has the inside dimensions of 200 x 200 x 200 mm and 10 mm wall thickness. Two steel clusters were attached in order to strengthen the box for lateral straining.

It is theoretically known that vertical effective pressure distributions extend down to 2-3 B. Therefore, the internal dimensions were chosen 200 mm to form a medium like an elastic half space. A jack having 370 kgf (3.7 kN) lift capacity, with an equipped loading frame, was used to apply the consolidation pressure. A loading hanger was used to apply constant consolidation load during the test.

3.1.2 Loading Frame and Loading Jack

Jacks with loading capacities of 375 kgf (3.75 kN), was connected to the loading frames consisting of four steel rods and U steel plates. The piston rod of the loading jacks pressed the test specimens as seen in (*Figure 3.2*). Desired amount of consolidation pressure was applied by means of the jack.

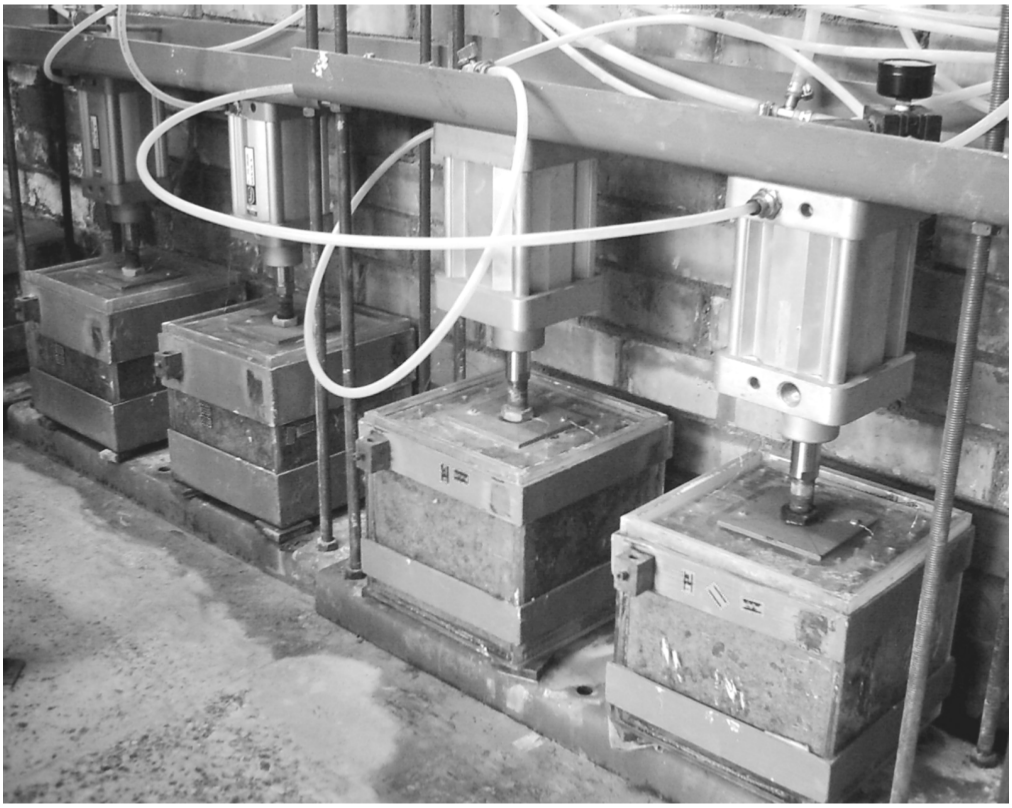
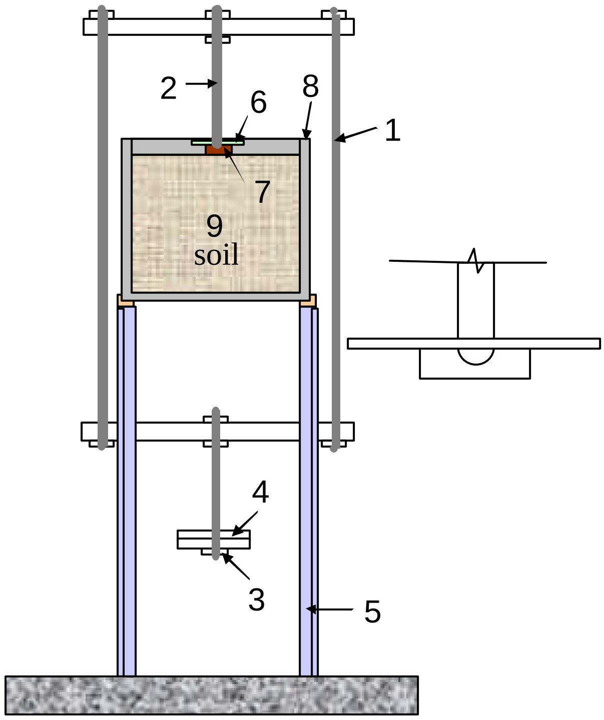


Figure 3.2 Loading Frame and Jack System



1. Loading frame
2. Loading rod
3. Hanger for weights
4. Load
5. Box support
6. Platform for transducers
7. Model footing
8. Plexiglas test box
9. Soil sample

Figure 3.3 Components of the Testing Assembly (After *Kul, 2003*)

3.1.3 Commercial Type of Kaolin Clay

In this experimental study commercial type of kaolin clay, which has low plasticity and low activity, was used. This type of clay is usually preferred in model testing in avoidance of potential complications in behaviour due to swelling, shrinkage...etc.

Kaolin powder was obtained by grinding the oven-dried kaolin samples. Soil specimens used in the tests were prepared by mixing the kaolin powder with water in desired consistencies. The same procedure was adopted in the remoulding process for each time to minimize the variations on the results of tests. The prepared soil is left for maturation in the humidity room for at least 2 days. It is then placed in the Plexiglas boxes for consolidation to obtain a standard soil specimen for testing. The consolidation duration was determined by *Kul (2003)* with Asoaka Method (*Figure 3.4*).

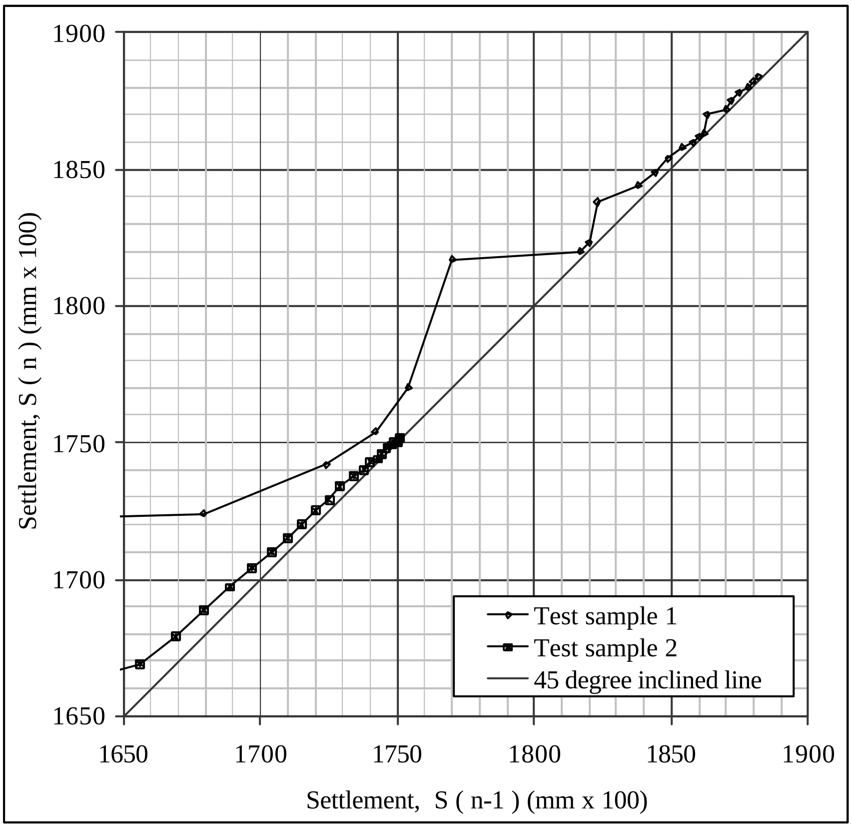


Figure 3.4 Asoaka Consolidation Curves for Two Samples (After *Kul, 2003*)

3.1.4 Aluminum Model Footing

Aluminium model footings are used for the application of footing pressures. In the first series of tests, model footings of dimensions 30 x 30 x 10 mm, and for the rest of the testing program, 50 x 50 x 15 mm (*Figure 3.5*) footings were used. The thicknesses of the aluminium footings were chosen such that they act as rigid and also much more than settlement amounts for a correct observation of settlement amounts (i.e. the platform fixed on top of the footing made for transducers and dials, should not touch the surface of the soil during settlement period).

Figure 3.5 Placement of 50x50x15 mm Model Footing with a Rigid Steel Platform Fixed on Top on the Soil Surface Freely Before Testing

3.1.5 Brass Nails

The model nails used in this model study is made of brass with a diameter of 2mm. The length of nails 1, 1.33, 2.4, 3 times the width B of the footing, namely, 30mm, 40mm, 72mm and 90mm respectively were used for the first part of the testing program where the model footing was 30x30x10 mm. In the rest of the testing program the nail lengths of 1.5 and 2 B were used, where 50x50x15 mm model footings were used. The elastic modulus of the brass is $1x10^8$ kN/m^2 (*Kul, 2003*). The surface of the brass nails was rasped in order to obtain a rough surface to increase frictional capacity. Nails prepared in desired lengths were inserted into soil sample vertically or inclined manner in different patterns by hand (*Figure 3.20*).

3.1.6 Timber Templates

In order to insert nails in desired inclinations, timber templates having groves of 2 mm width such that a nail just fits on were used. Leaning on the grove, nails were driven into soil in correct angle of inclinations (*Figure 3.6*).

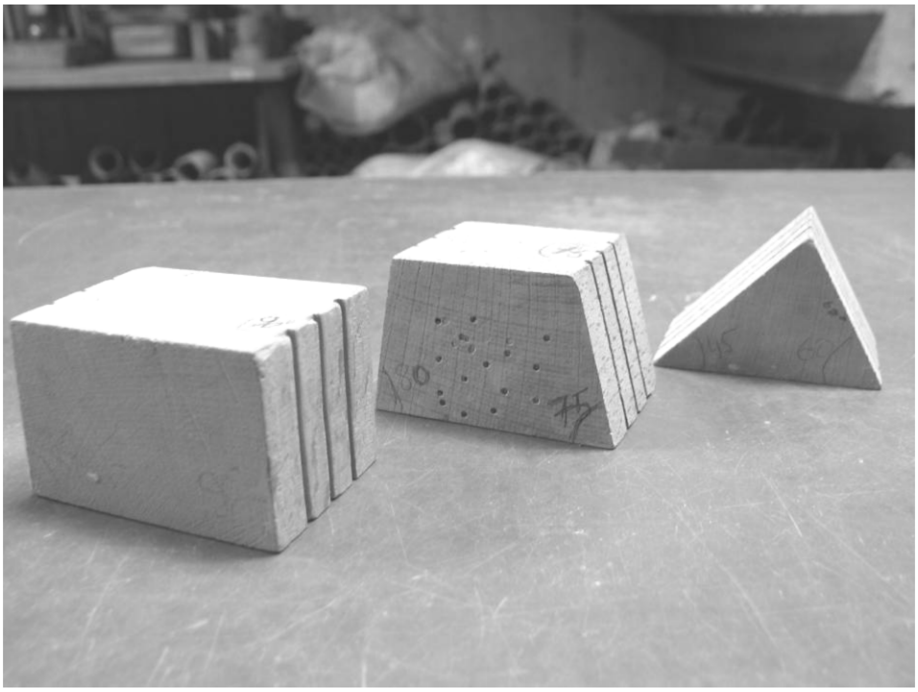


Figure 3.6 Timber Templates of Different Inclinations

3.2 Laboratory Testing

The following index properties were determined by *Kul (2003)* after performing standard laboratory tests in accordance with TS 1900.

3.2.1 Atterberg Limits

The Atterberg limits tests were performed on the prepared soil, which was left for five days for it become mature and homogeneous batch. Liquid and plastic limits are given in *Table 3.1*.

Table 3.1 Atterberg Limits of the Kaolin Clay (After *Kul, 2003*)

LL (%)	PL (%)	PI (%)	Classification
49	25,5	23,5	CL

3.2.2 Grain Size Distribution

The particle size distribution of the soil sample was determined with the standard method for fine-grained soils (Hydrometer test). The soil fractions with their percentages and the grain size distribution curve are given in *Table 3.2* and *Figure 3.7* respectively.

Table 3.2 Soil Fractions and Their Percentage (After *Kul, 2003*)

Soil Fraction	Grain Size Range (μm)	Material Passing, %
Fine Sand	200 - 60	99
Coarse Silt	60 - 20	97
Medium Silt	20 – 6	91
Fine Silt	6 – 2	70
Clay	<2	45

Therefore the percentage of fine sand, silt and clay in the kaolin is 1, 54 and 45, respectively. According to SI soil classification the soil is silty–clay with low plasticity. The specific gravity (G_s) of the kaolin clay was determined as 2.70.

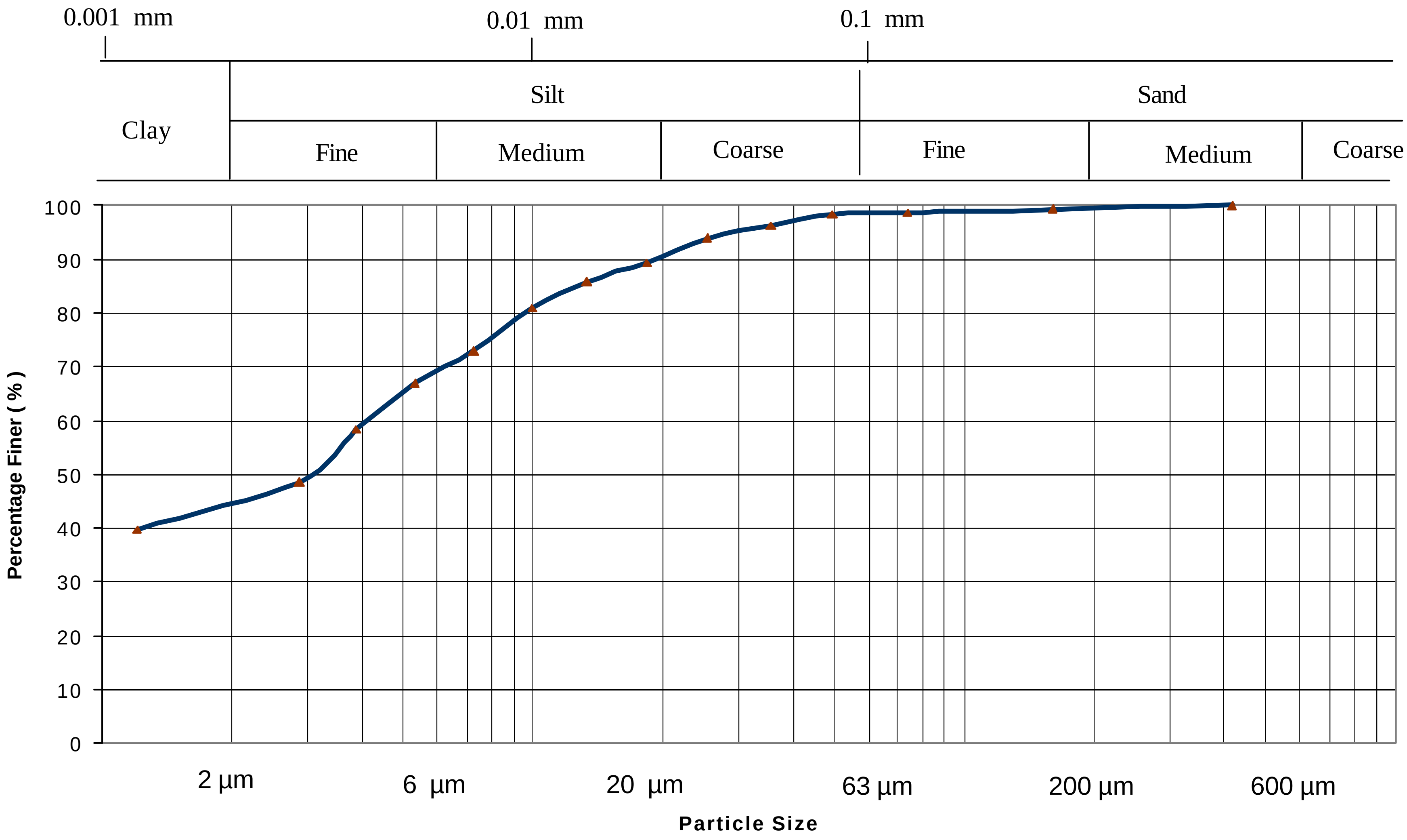


Figure 3.7 Grain Size Distribution Curve (After *Kul, 2003*)

3.3 Testing

3.3.1 Nail Number and Length Relationship

The aim of this section of the testing program is to determine the most effective nail pattern keeping the total nail length. In this part of the model study, five series of tests were performed with different nail numbers (*Figure 3.8*), hence different individual nail lengths, including the tests with no nail, which are summarized in *Table 3.3*.

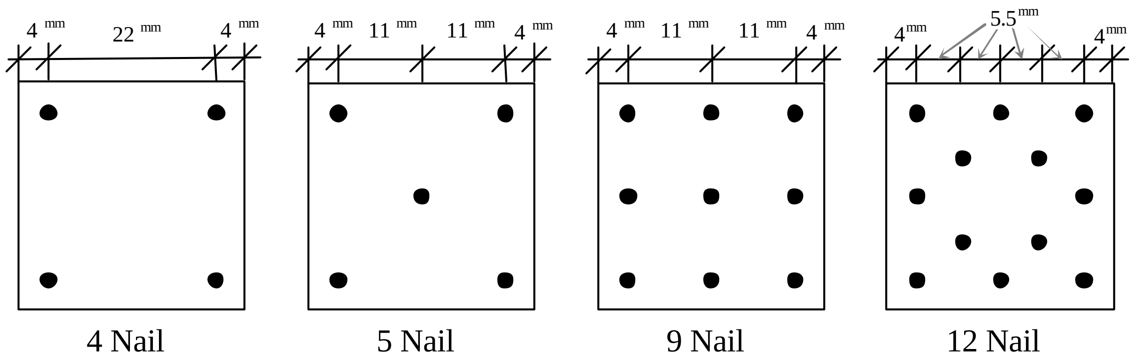


Figure 3.8 The Nailing Patterns for the Study of Nail Number-Length Relationship (B=30 mm)

Table 3.3 Summary of the First Section of the Testing Program

Nail Length	Number of Nail	Number of tests performed
3 B	4	4
2.4 B	5	4
1.33 B	9	4
B	12	4
No Nail	-	2

3.3.2 Effect of Nail Inclination

In this section of the experimental study, the effect of inclination of soil nails in terms of settlement reduction is investigated. In the first part of this section, the inclination effect was investigated by four nails having lengths of 2B were driven vertically and in 15^0 inclination ($B=30^{mm}$, σ_f =50 kPa). In the second part, firstly, the effect of nail inclination is investigated by four nails that were inserted vertically, and 15^0 and 30^0 inclinations in a series of tests ($B= 50^{mm}$, σ_f =50 kPa). After this effect was analysed, it was checked with a series of tests by six nails of 2B length, which were also driven vertically, and 15^0 and 30^0 inclinations ($B= 50^{mm}$, σ_f =50 kPa) (*Figure 3.9* for the sketches of nailing patterns).

In this section, ten series of tests, including no nail case, were performed. The nail length and numbers used are summarized in *Table 3.4*.

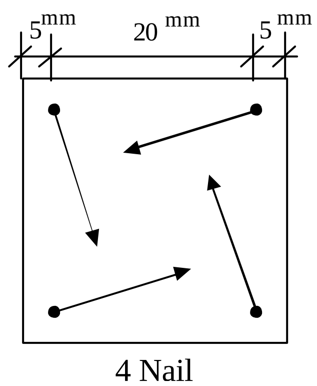


Figure 3.9 The Nailing Patterns for the Study of Nail Inclination Effect (*First Part*; B=30 mm)

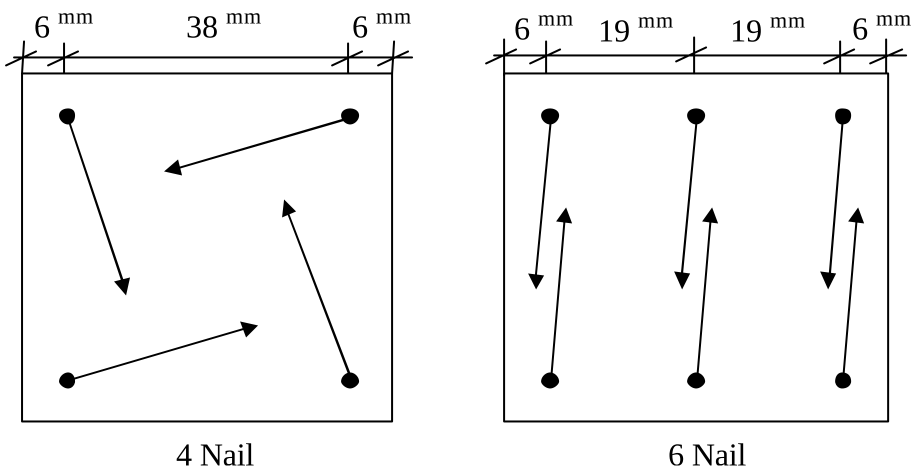


Figure 3.10 The Nailing Patterns for the Study of Nail Inclination Effect (*Second Part*; B=50 mm)

Table 3.4 Summary of the Testing Program for the Investigation of Inclination Effect

B	Nail Length	Number of Nail	Number of tests performed			No Nail Case
			Vertical	15^0 Inclined	30^0 Inclined	
30 mm	2 B	4	6	6	-	4
50 mm	1.5 B	4	3	5	6	7
	2 B	4	3	7	4	
		6	3	3	3	

3.3.3 Testing Procedure

The testing procedure is as follows:

a. The raw kaolin was dried and then powdered by means of a grinder.
b. The kaolin powder mixed with water to achieve a water content of 38% and left for rest at least 2 days.
c. Inside the Plexiglas boxes, geotextile sheets, which are cut in appropriate width and length, were placed for proper drainage and also for prevention of drying of the soil during consolidation period (*Fig.3.11*).
d. The soil was then placed in the Plexiglas boxes layer by layer manually, and then surface was smoothened with the aid of a straightedge (*Figures 3.12*, *3.13* and *3.14*).
e. The top portion is covered with the geotextile sheets (*Fig. 3.15*) and the Plexiglas cover, which has small holes for drainage, was placed (*Fig. 3.16*).
f. The Plexiglas box was placed under the loading jack (*Fig. 3.2*).
g. 50 kPa consolidation pressures were applied for a consolidation time of three weeks at least.
h. The samples were wetted from top periodically to prevent drying during the consolidation period.
i. After consolidation period, the sample was taken and the upper 30 mm portion of the soil was removed in thin layers - not to cause any disturbance- with the aid of another straightedge specifically designed for this operation

(*Figure 3.17, 3.18, 3.19*). This is a kind of remedy for the minimization of the effect of inevitable surface drying and also possible suction on top portion just after the removal of consolidation pressure. During this operation, samples were taken for water content determination.

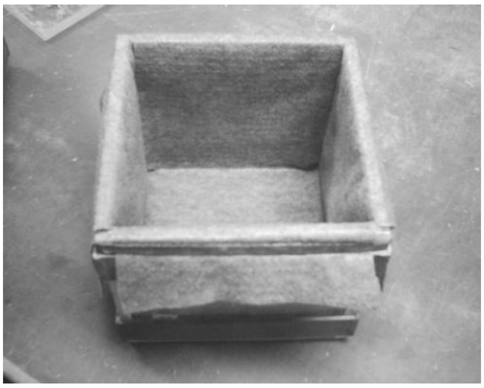


Figure 3.11 Empty Box in which Geotextile Sheets Placed

Figure 3.12 Placement of Prepared Soil Manually

Figure 3.13 Representative Samples Taken for Water Content Determination

Figure 3.14 Surface Finishing with the Aid of a Special Straightedge

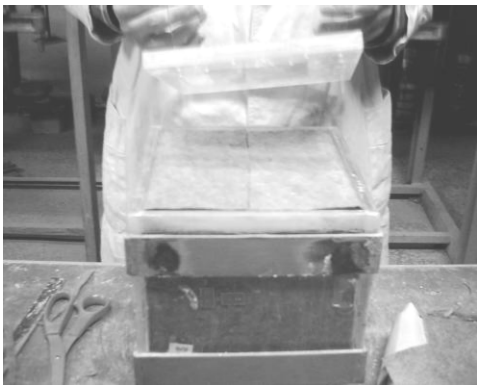


Figure 3.15 Placement of the Plexiglas Cover

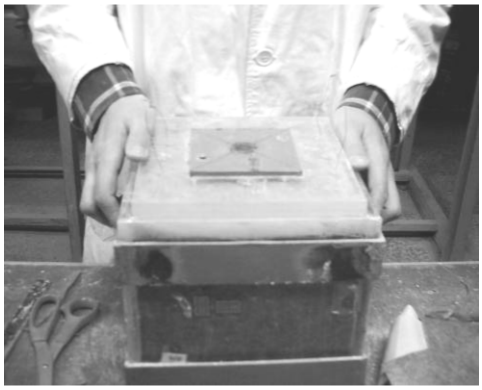


Figure 3.16 Specimen Ready to be Consolidated under Constant Stress

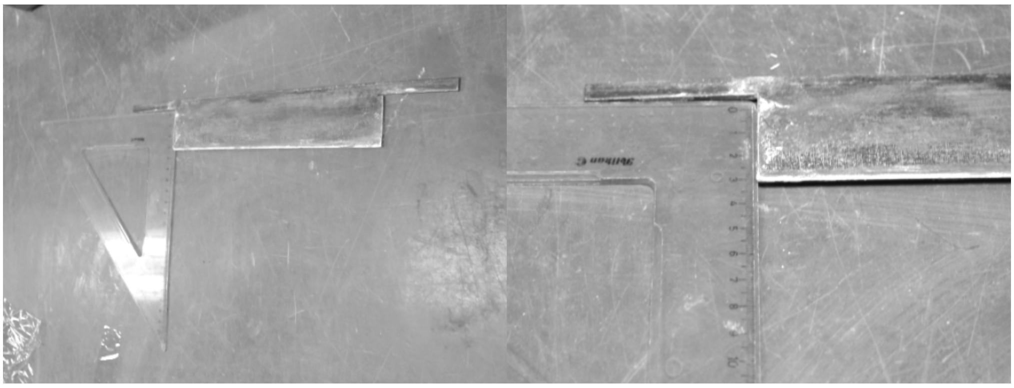


Figure 3.17 Steel Straightedge Designed for Removal of 3 cm Top Portion of Consolidated Soil

The surface is smoothened with spatula. The related procedure after this point is explained in the following parts in details for the two cases of the testing programme respectively.

3.3.3.1 Testing Procedure for Nail Number and Length Relationship

- A thin nylon sheet, with a square shaped opening just bigger than the size of the model footing is placed (*Figure 3.22*).
- A template sheet prepared showing where and in which directions the nails will be driven (*Figure 3.23*) is placed. On this sheet there are openings left through which the nails will be driven.

- The model brass nails of 2 mm diameter were inserted in determined number and pattern through this indicative sheet, with the aid of a supporting timber template, which was grooved for the nails to be driven in desired manner (vertical or inclined to the soil surface, *Figure 3.26*).
- The model footing is placed (*Figure 3.27*) on this reinforced soil such that it does not cause any eccentricity in guidance of light traces left on soil surface tangent to the outer nail heads with the edge of a spatula (*Figure 3.24-3.25*). Remaining open parts are also covered with nylon sheet (*Figure 3.28* and *3.29*).
- Remaining portions of geotextile sheets are soaked, rolled and placed at the edges as described in *Figure 3.30.*
- After levelling of the footing (*Figure 3.31*) on top a nylon sheet is covered (*Figure 3.32*) and small holes are opened for the placement of LVDT and/or displacement dials and for load langer (*Figure 3.33*).
- The transducers (LVDT's), which are fixed on an independent refence frame attached on sides of the Plexiglas boxes, are placed on two opposite sides on a rigid platform attached to the footing (*Fig. 3.33.b, 3.34, 3.35*)
- The prepared specimen is left to heave for about 12 hours.
- The load hanger is levelled (*Figure 3.*36).
- After the critical heaving period, which actually keep on continuing at lesser rate, the load hanger that cause 40 kPa stress, is placed on the footing. At the same time the data acquisition system (ADU) start to take settlement values.
- The settlement readings were taken with a data acquisition system consisting of a PC, 16 – channel data logger and software (DADU), which monitor and store the test data at desired time intervals.

3.3.3.2 Testing Procedure for Effect of Nail Inclination

- A nylon sheet is placed on the smoothened surface to avoid drying of the sample surface (*Figure 3.19*).
- The remaining parts of the geotextile are folded over it and covered with a sheet of nylon (*Figure 3.20*).
- The sample is left to free heave for about 23 hours (*Figure 3.21*).
- The heaved portion is removed with the straightedge having 30 mm edge and smoothened.
- The insertion of nails and the other detailed works for sample preparation is carried out as done in the Part 3.2.3.1, but the settlement readings are taken with dials and the footing pressure is increased to 50 kPa.

j. Testing period was continued for about 72 hours.

k. After testing representative samples were taken under footing and from all other parts of the tested sample for water content determination.

The test results were interpreted in the following chapter.

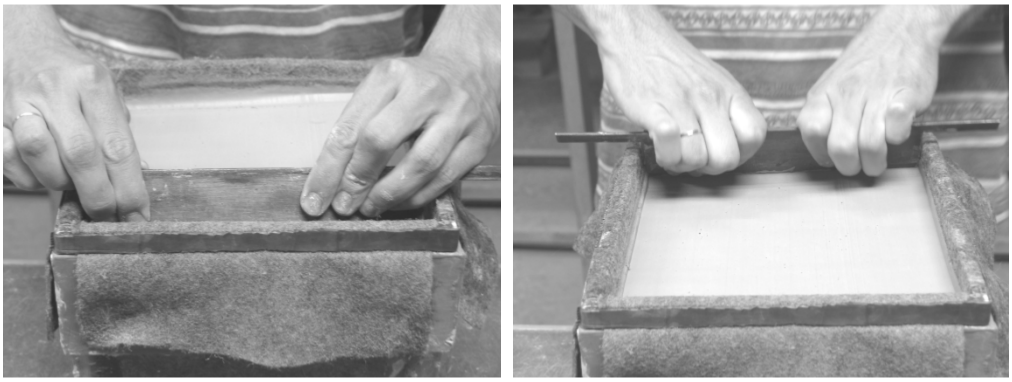


Figure 3.18 Removal of Top 30 mm Portion after Consolidation Period

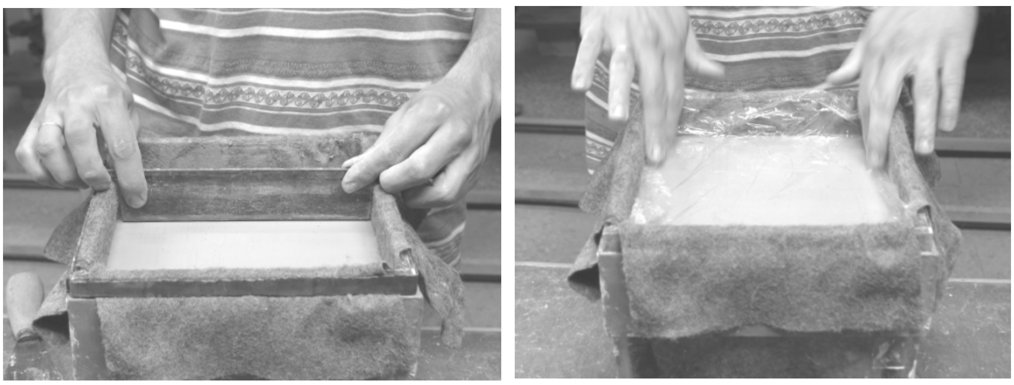


Figure 3.19 Finalization of Removal of 30 mm Top Portion and Nylon Sheet is Placed

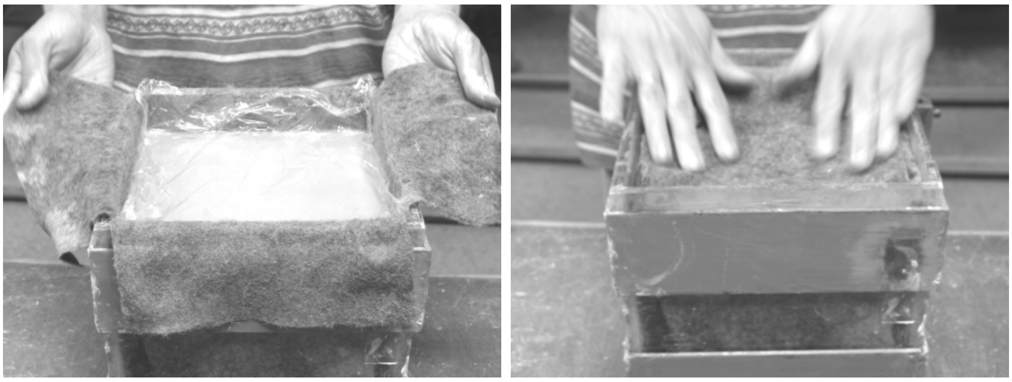


Figure 3.20 Folding over the Remaining Portions of Geotextile Sheets

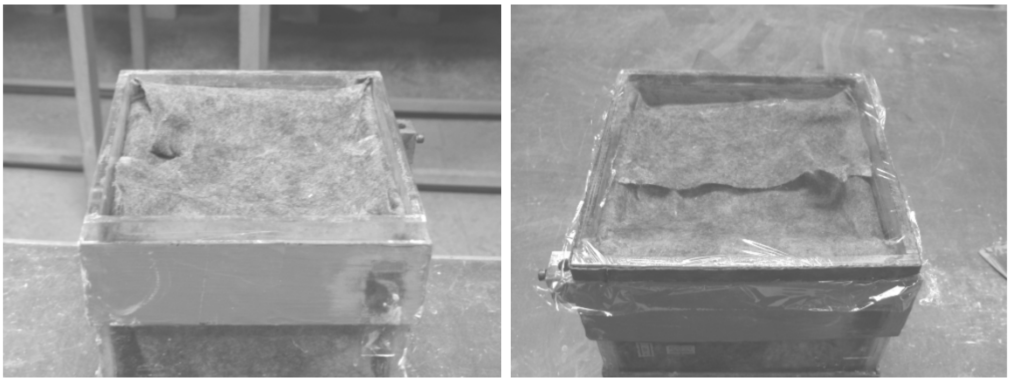


Figure 3.21 Final View of Sample Left for Free Heave

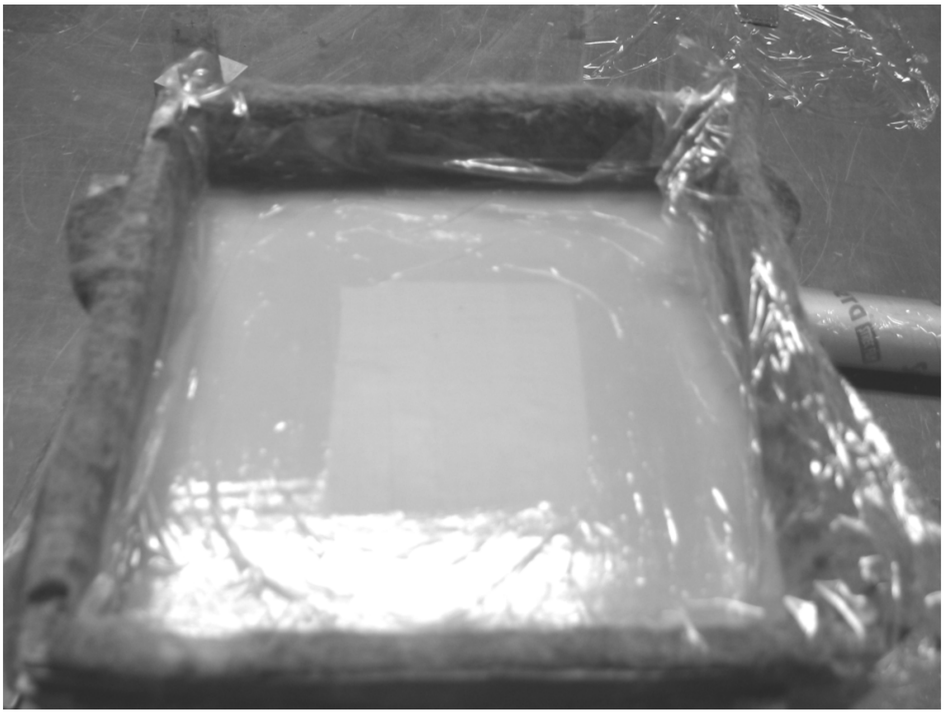


Figure 3.22 Prepared Specimen just before the Nails were Inserted (A Nylon Sheet with a Square Opening at Centre for Improvement Operations)

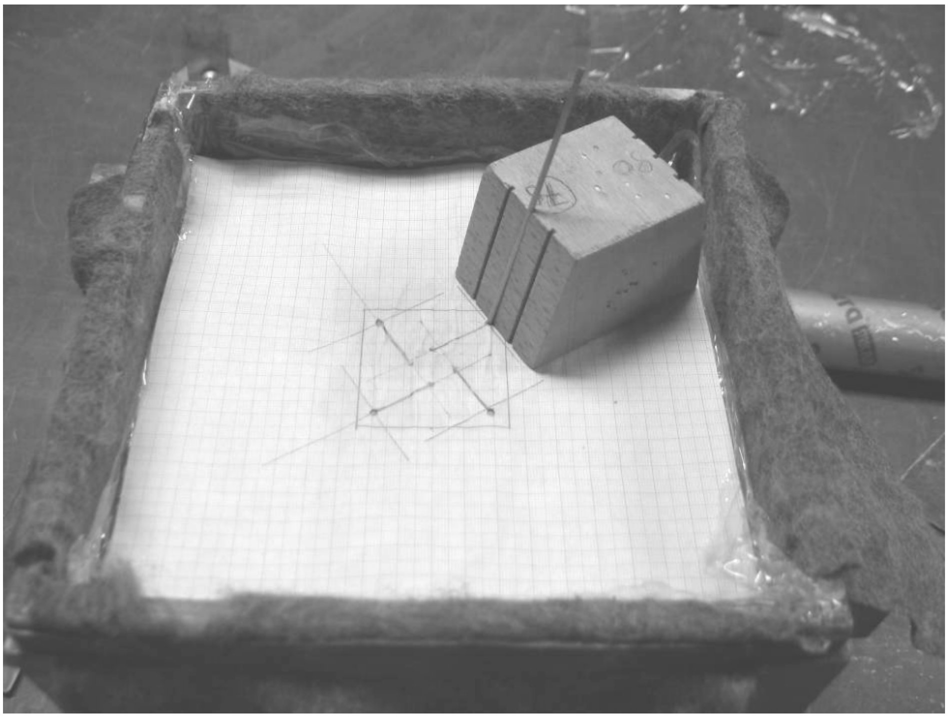


Figure 3.23 Nail to be Driven (15 Degrees from Vertical) Leaned on the Groove of Timber Temp late Support.

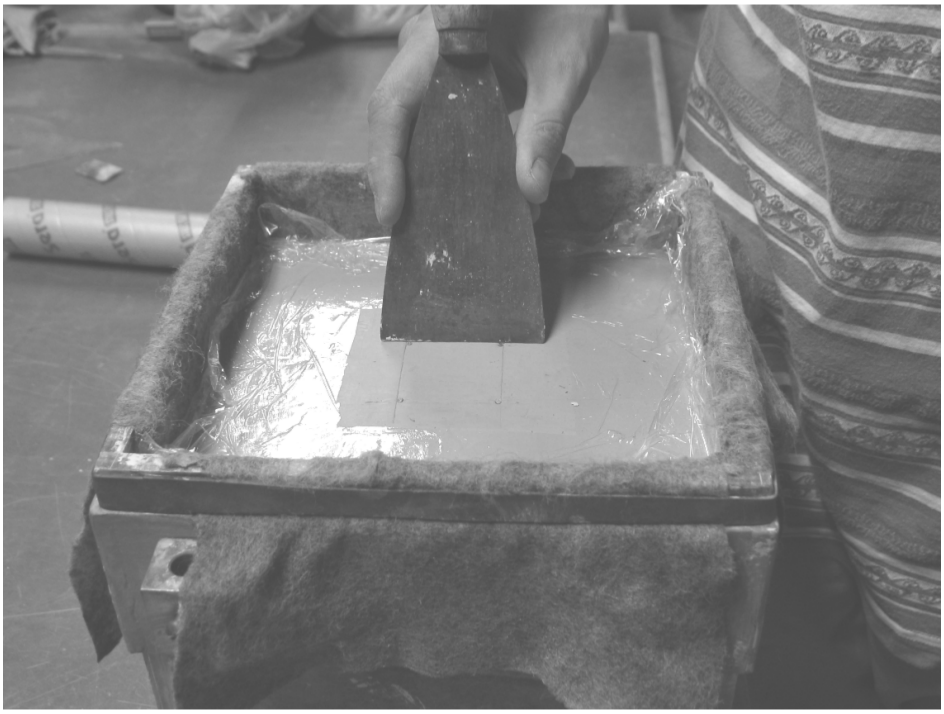


Figure 3.24 Leaving Traces with Spatula after the Insertion of Nails

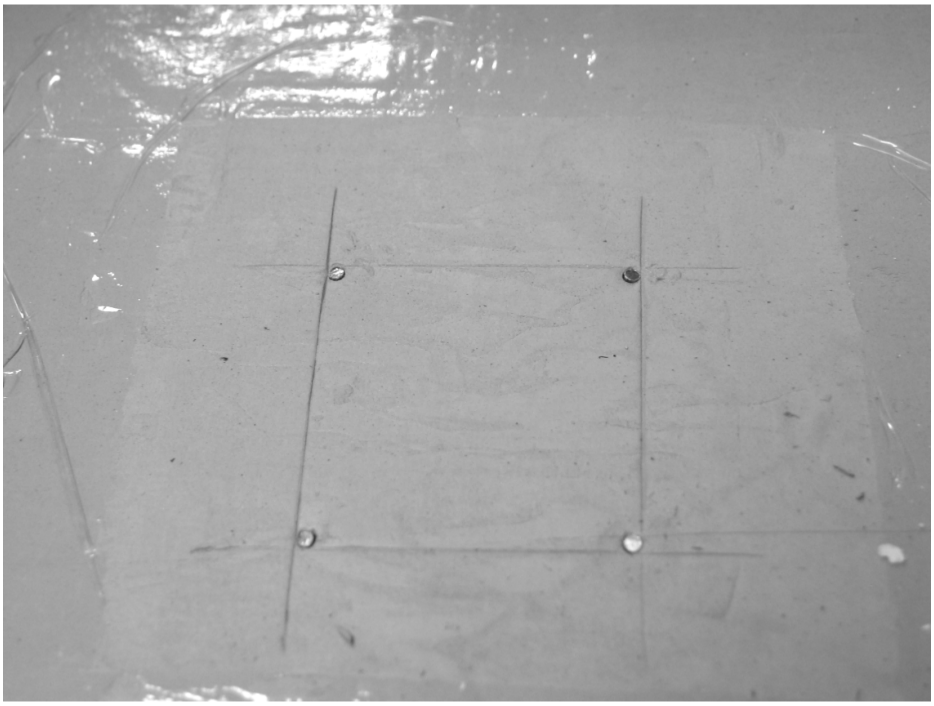


Figure 3.25 Left Traces for the Guidance of Placing the Footing Concentrically

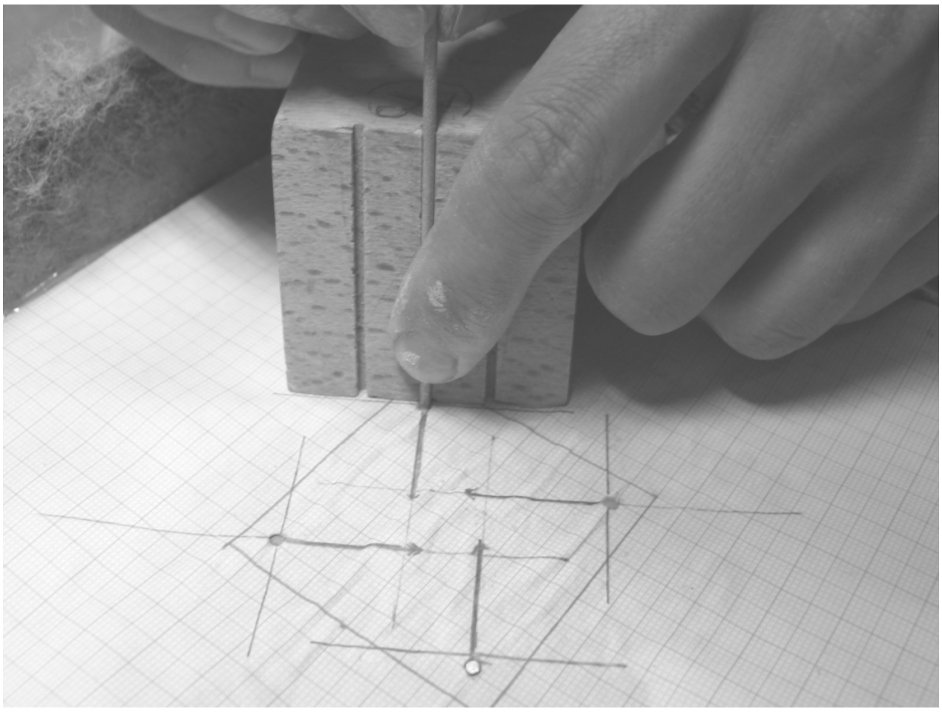


Figure 3.26 Insertion of Nail by Hand in Guidance of the Timber Template

Figure 3.27 Placement of the Footing in Guidance of the Traces Left by Spatula

Figure 3.28 Placement of Nylon Sheet around the Footing

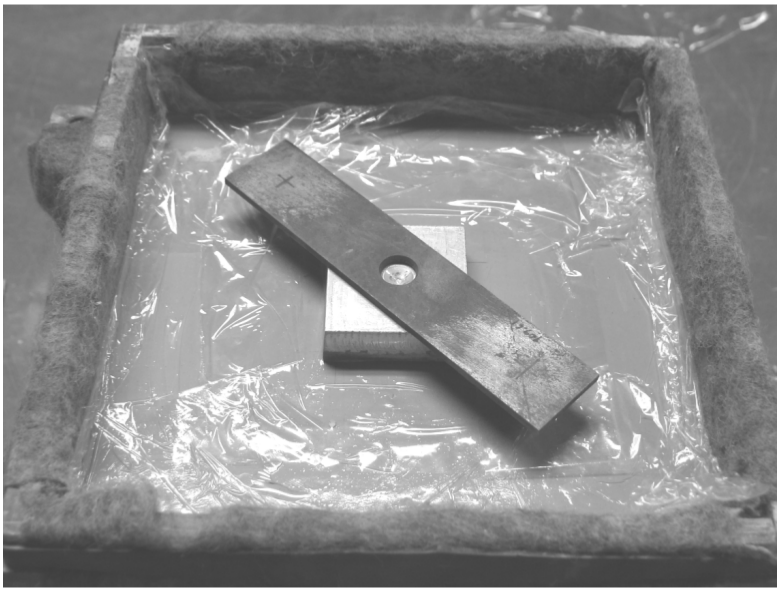
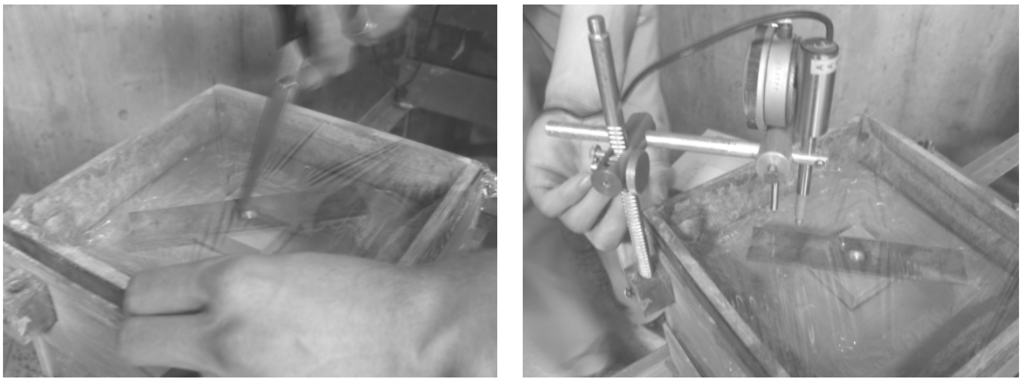


Figure 3.29 After the Placement of the Footing (Improvement Complete)

Figure 3.30 Wetting and Rolling of Remaining Portions of Geotextile

Figure 3.31 Leveling of Footing

Figure 3.32 Top Covered with Nylon Sheet



a b

Figure 3.33 **a)** Opening Holes for Transducers, Dials and Load Hanger
b) Fixing LVDT & Dial

Figure 3.34 Placement of the Load Hanger on the Steel Support
(Load is not Applied on the Footing Yet)

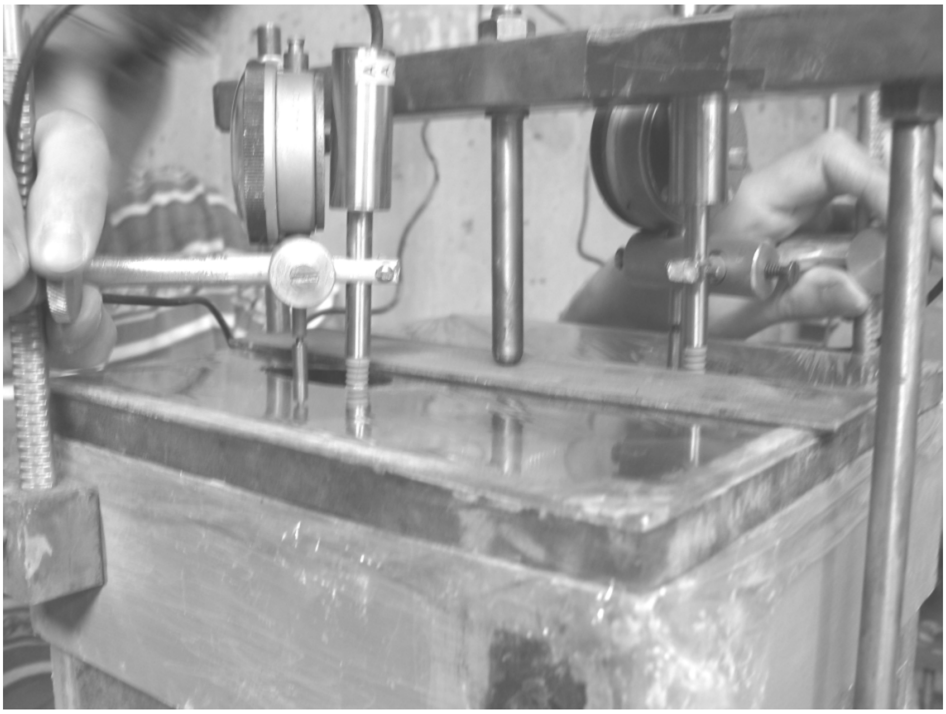
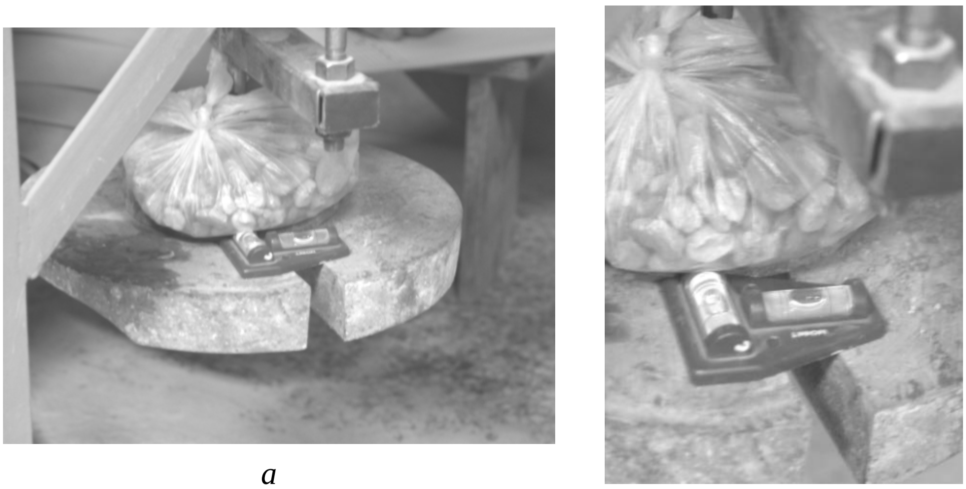


Figure 3.35 Adjustments of LVDT's and Displacement Dials



a

b

Figure 3.36 **a)** Levelling of the Load Hanger **b)** Close View

Figure 3.37 After the Load is Applied (Testing Stage)

4 TEST RESULTS

4.1 General

In this model study, the effect of soil nails applied in various patterns under rigid footing as settlement reducers was investigated.

The soil samples used in the tests were lightly overconsolidated (OCR=1.25) and normally consolidated kaolin type clay for the first and second part of the testing program, respectively. After insertion of the nails and placement of footing over, vertical displacement transducers were placed on a platform fixed to the footing. Finally, a constant vertical load (P_f) was applied concentrically and corresponding settlements were recorded with a data acquisition system in the first section of the testing program. In the second section, settlement readings were taken with displacement dials, which have 0.001-inch accuracy.

In the first part of this chapter, settlement curves of footings (B = 30mm, σ_f =40 kPa) on improved soil with nailing patterns of 4, 5, 9 and 12-nails having same total length are presented (*Figures 4.1-4.5*).

In the first part of the second section (where; B=30mm, σ_f =50 kPa), settlement curves of 4 nails of 2B length (4 x 2B) for vertical and 15^0 inclined nails, and also no nail case are presented (*Figures 4.6-4.8*). In the second part (where; B=50mm, σ_f =50 kPa), settlement curves of nailing patterns of 4 and 6-nails that are normal to the ground, 15^0 and 30^0 inclined from the normal of the ground and also no nail case are presented (*Figures 4.9-4.18*).

The average curves with their error bars are plotted for each case on the corresponding graphs. The average curves are shown in dashed lines with error bars, which define the ranges for each nailing case. Upper and lower ranges of the error bars are determined from the standard deviations of each settlement data point of the corresponding delayed time for all of the test series of same nailing patterns. For example, the error bar ranges for the delay time of 10 second, is the standard deviation of the settlement values of all tested specimens for the delay time of 10 second.

After the determination of the ranges of settlement curves for each nailing pattern separately, the average curves for each pattern were plotted by taking the average of settlement curves of the corresponding pattern of nailing, discarding the ones out of the ranges. Although not being taken into account in the determination of average values of settlement, the out-of-range curves were shown on the corresponding settlement vs. time graphs of each nailing pattern.

4.2 Settlement Measurements of Footings on Nailed Soil

The settlement behaviour of the model tests will be discussed mainly in two headings:

- Nail number and length relationships
- Effect of nail inclination

4.2.1 Nail Number and Length Relationships

In this part of testing program, the nailing patterns (i.e. number of nails) were changed keeping the total length of nails constant. Settlement behaviours of 4, 5, 9 and 12-nails, which have lengths of 3 B, 2.4 B, 1.33 B and B respectively and also no nail case, were investigated. The footing breadth and pressure, B and σ_f were 30mm and 40 kPa, respectively.

For this part of the experimental work a total number of 18 tests performed and the results are presented in the form of tables and figures. The settlement values (initial and consolidation), water contents and dates of testing are summarized in *Table 4.1*.

The settlement amounts recorded under constant load are presented as settlement versus logarithm of time curves for each tested specimen. The settlement curves are presented for different nailing patterns separately. The settlement behaviours of 4 nails having 3 B length (shown as 4 x 3B), 5-nails having 2.4 B length (5 x 2.4 B), 9-nails having 1.33 B length (9 x 1.33 B) and 12-nails having B length (12 x B) are presented in *Figures 4.1, 4.2, 4.3, 4.4* and *4.5*, respectively.

The settlement amounts, and also settlement reduction percentages for increasing length of nails are presented and discussed in *Chapter 5*.

4.2.2 Effect of Nail Inclination

In this part of the testing program, effect of nail inclination was investigated through a series of tests in which the inclination of nails were changed for the same length. The total number of tests carried out in this section of the testing program is 60 (15 tests in the first part, 45 tests in the second part).

In the first section, the effect of nail inclination for 4-nails of 2 B length was investigated. Vertical, 15^0 inclined nails of B=30mm and no nail case were tested for σ_f= 40 kPa. The test results for this part are summarized in *Table 4.2*. The settlement vs. logarithm of time curves are presented in *Figures 4.6,.4.7* and *4.8* for vertical, 15^0 inclined nails and no nail case, respectively.

In the second section, initially, the inclination effect was investigated for 4-nails of B=30mm, for 1.5 B and 2 B cases for σ_f= 40 kPa. The settlement versus logarithm time curves are given in *Figures 4.9, 4.10, 4.11, 4.12, 4.13* and *4.14* for vertical, 15^0, 30^0 inclined cases, respectively.

Finally, to verify the results and conclusions of the effect of nail inclinations for 4-nail cases, patterns of 6-nail cases were investigated. The settlement versus logarithm of time curves are given in *Figures 4.15, 4.16* and *4.17* for vertical, 15^0, 30^0 inclined cases, respectively. Results obtained for No Nail Case for this part is given in *Figure 4.18.*

The summary of test results is shown in *Table 4.1* for the first section and *Table 4.2* and *4.3* for the first and second parts of the second section, respectively.

Table 4.1 Summary of Test Results for the First Section of the Testing Program.

B= 30 mm s_f= 40kPa	S_T (mm)	S_i (mm)	S_c (mm)	w (%)
4 x 3B	0,870	0,330	0,540	----
	0,826	0,254	0,572	----
	0,823	0,320	0,503	34,68
	0,738	0,293	0,445	33,39
Average	0,814	0,299	0,515	34,04
St. Deviation	0,055	0,034	0,055	0,91
5 x 2.4B	1,406	0,777	0,629	33,98
	1,039	0,457	0,582	33,21
	0,712	0,264	0,448	34,00
	1,017	0,461	0,556	33,97
Average	1,028	0,459	0,569	33,79
St. Deviation	0,284	0,212	0,077	0,39
9 x 1.33B	1,386	0,377	1,009	----
	1,239	0,428	0,811	----
	1,037	0,452	0,585	34,18
	0,975	0,370	0,605	32,99
Average	1,084	0,417	0,667	33,59
St. Deviation	0,189	0,040	0,199	0,84
12 x B	0,955	0,476	0,479	32,79
	1,224	0,702	0,522	33,58
	1,132	0,526	0,606	33,70
	1,135	0,578	0,557	33,74
Average	1,134	0,552	0,582	33,45
St. Deviation	0,113	0,097	0,054	0,45
No Nail	1,552	0,783	0,769	34,03
	1,786	0,922	0,864	34,26
Average	1,669	0,853	0,817	34,15
St. Deviation	0,165	0,098	0,067	0,16

NOTE: Settlements values written in orange ink are not taken into account when calculating the average values

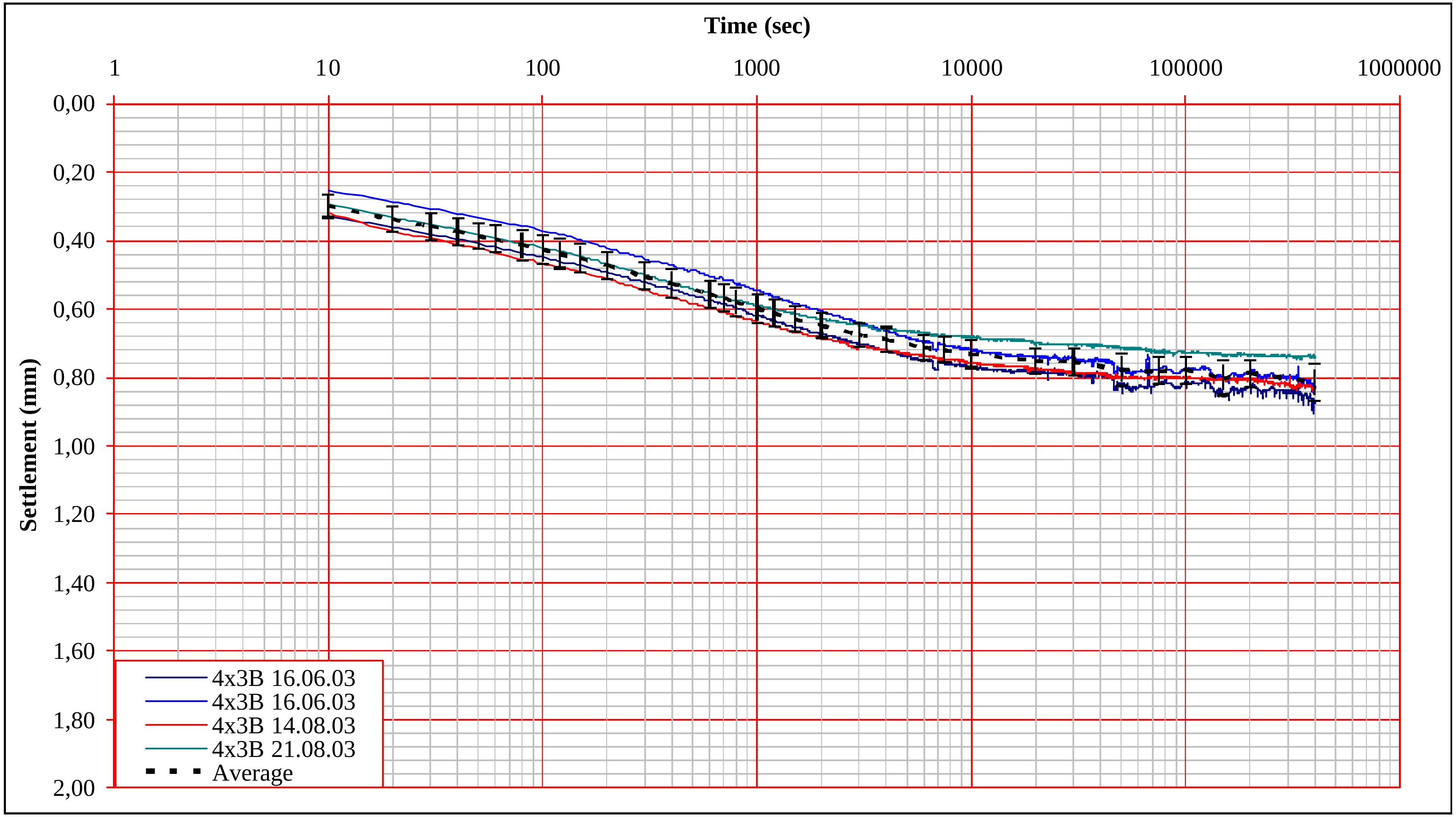


Figure 4.1 4 x 3 B (*B= 30mm*, s_f = *40 kPa*) Nail Group Settlement vs. Time Relations

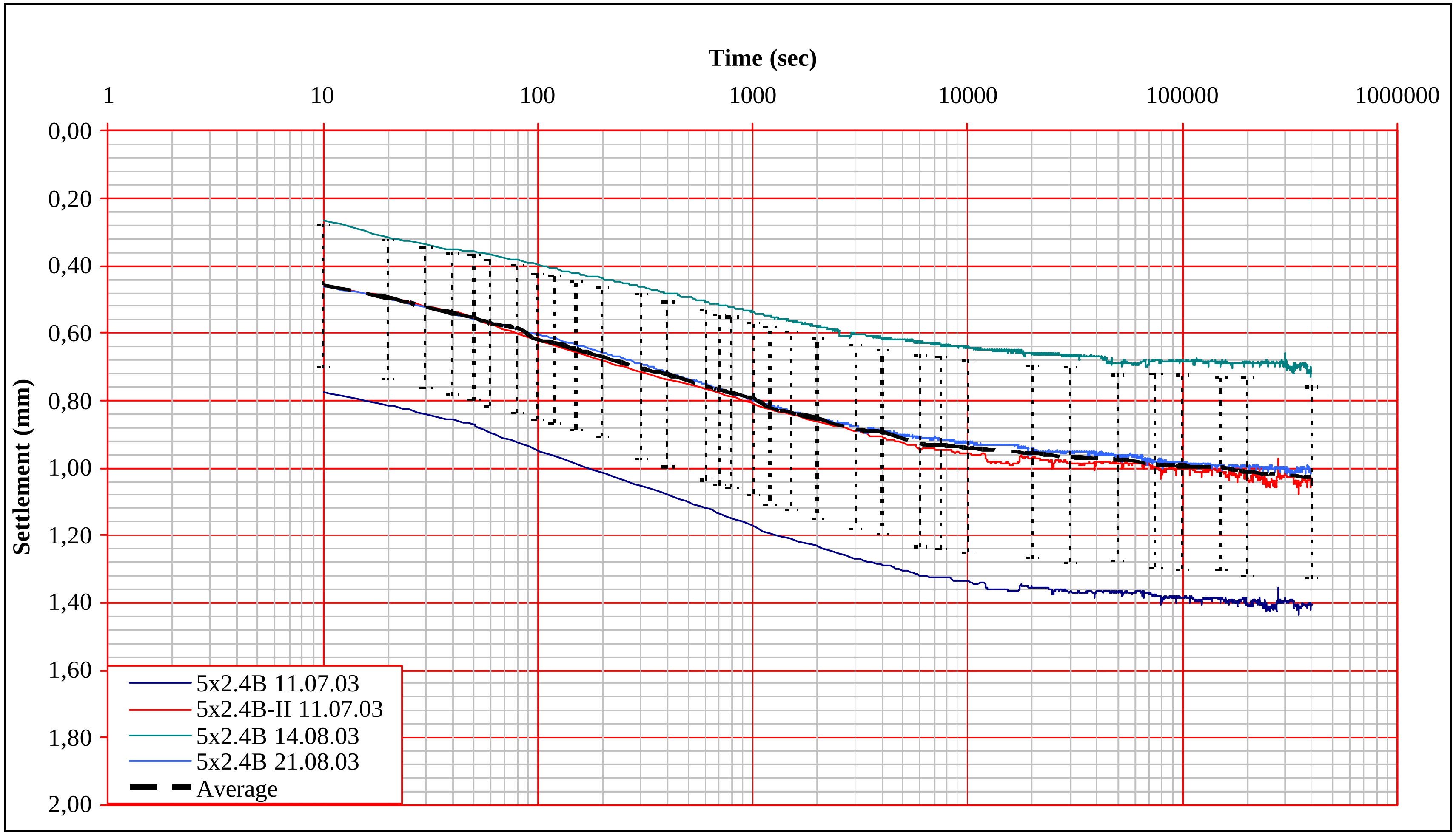


Figure 4.2 5 x 2.4 B (*B= 30mm*, s_f *= 40 kPa*) Nail Group Settlement vs. Time Relations

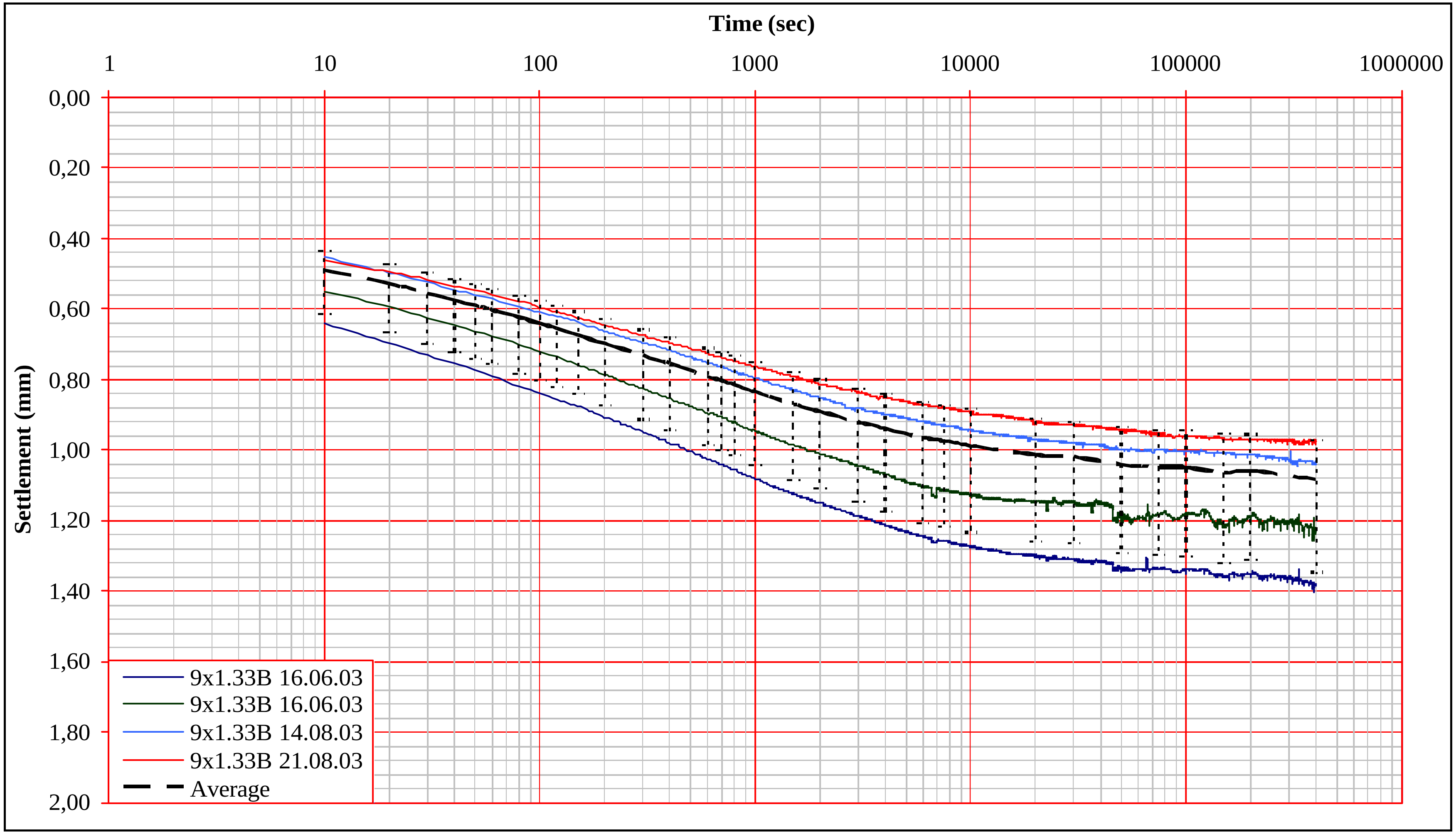


Figure 4.3 9 x 1.33 B (*B*= *30mm*, s_f = *40 kPa*) Nail Group Settlement vs. Time Relations

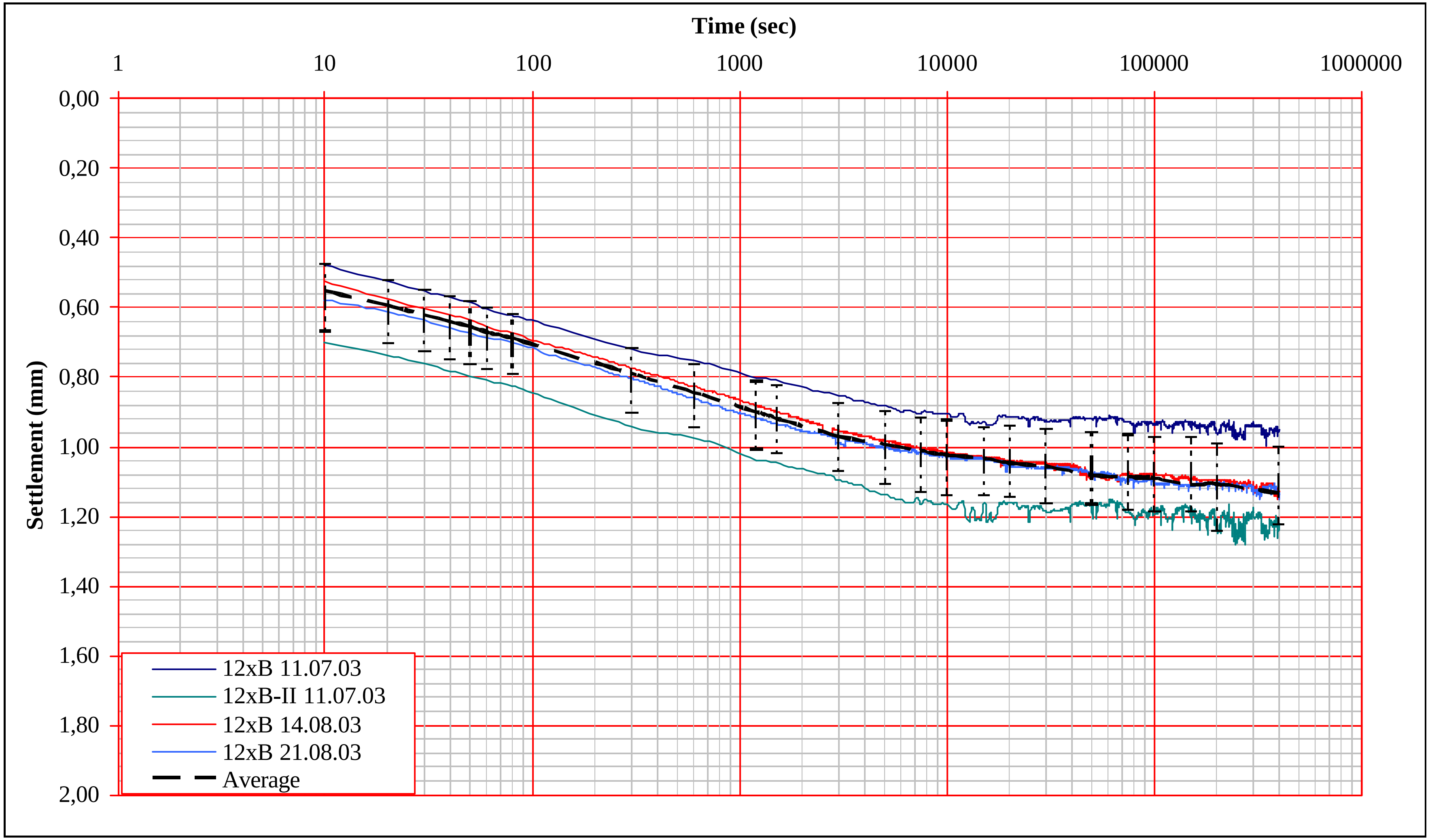


Figure 4.4 12 x B (*B= 30mm*, s_f *= 40 kPa*) Nail Group Settlement vs. Time Relations

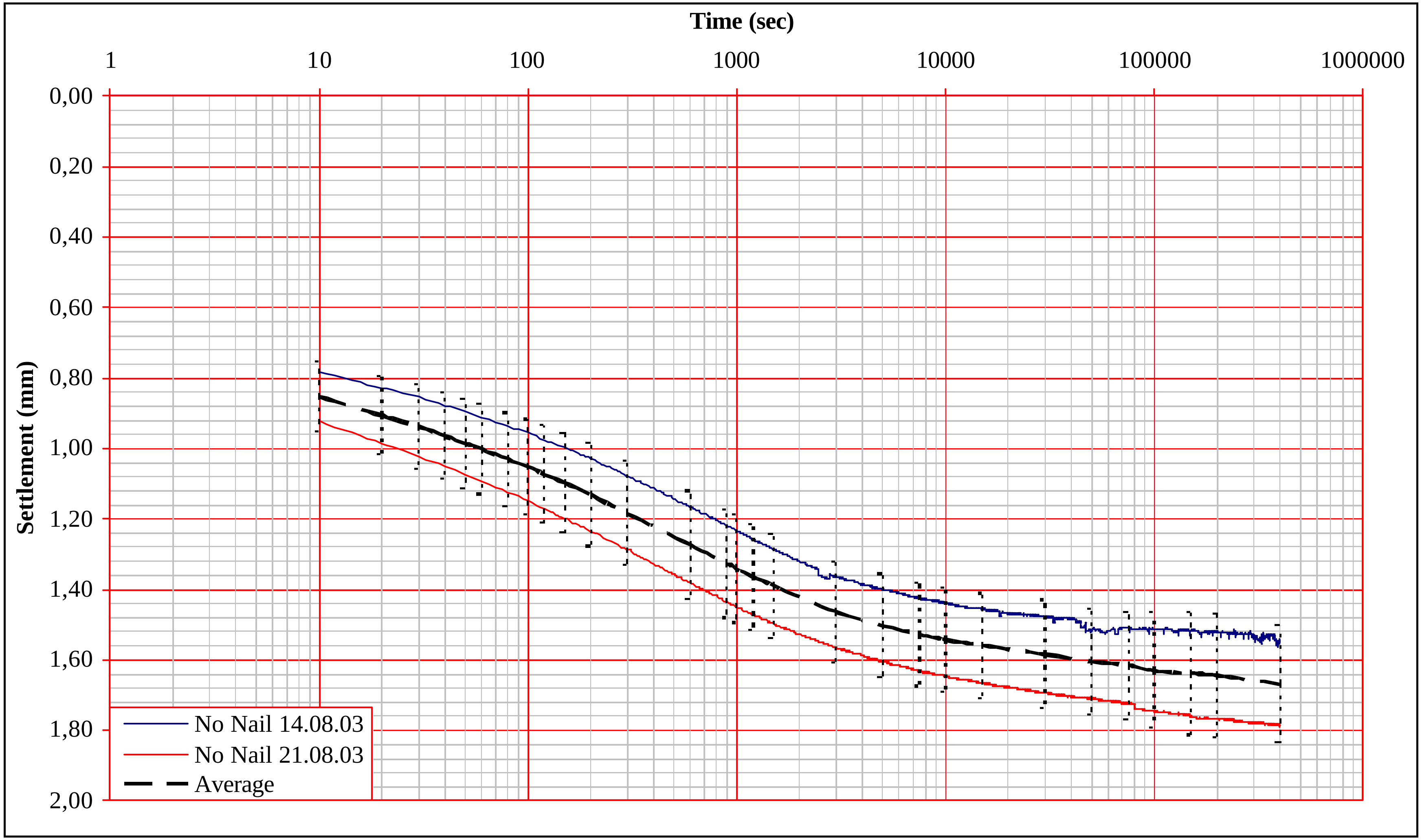


Figure 4.5 No Nail Case (B= *30mm*, s_f = *40 kPa*) Settlement vs. Time Relations

Table 4.2 Summary of test results for the investigation of nail inclination

B= 30 mm sf= 50kPa	S_T (mm)	S_i (mm)	S_c (mm)	w (%)
4 x 2B	0,930	0,350	0,580	33,61
	1,250	0,540	0,710	34,37
	1,100	0,510	0,590	33,89
	1,475	0,685	0,790	34,09
	1,530	0,720	0,810	33,74
	1,280	0,700	0,580	33,50
Average	**1,327**	**0,631**	**0,696**	**33,87**
St. Dev.	**0,225**	**0,144**	**0,108**	**0,32**
4 x 2B - 15^0 Inclined	1,195	0,600	0,595	34,45
	1,160	0,530	0,630	33,24
	1,120	0,555	0,565	34,10
	1,290	0,615	0,675	34,10
	1,720	0,800	0,920	34,05
	1,540	0,780	0,760	33,98
Average	**1,261**	**0,616**	**0,645**	**33,97**
St. Dev.	**0,240**	**0,115**	**0,131**	**0,40**
No Nail	1,840	0,920	0,920	33,63
	2,120	0,965	1,155	34,21
	2,360	1,290	1,070	33,96
Average	**2,107**	**1,058**	**1,048**	**33,93**
St. Dev.	**0,260**	**0,202**	**0,119**	**0,29**

NOTE: Settlements values written in orange ink are not taken into account when calculating the average values

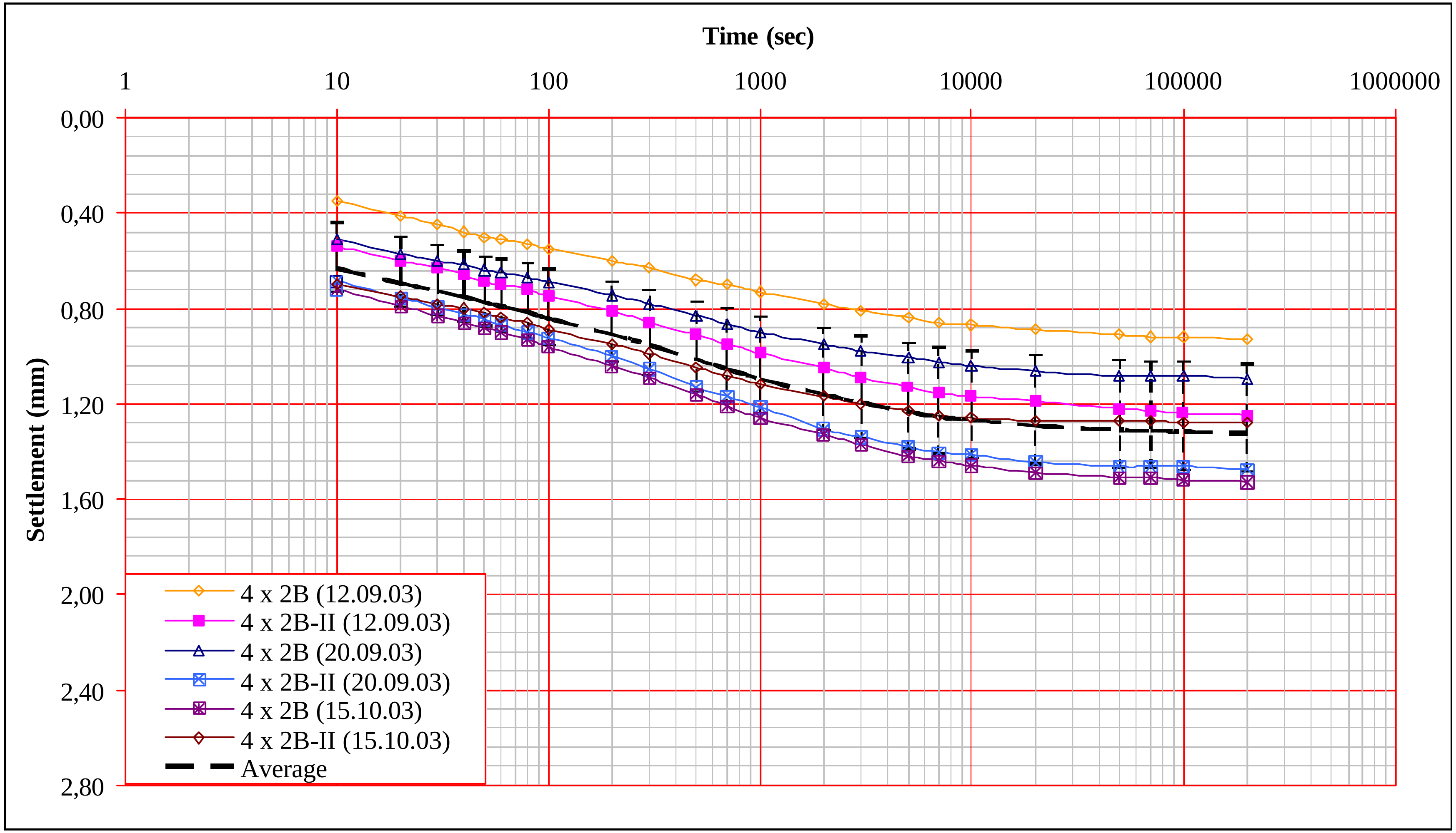


Figure 4.6 4x 2 B (B= $30mm$, s_f = $50\ kPa$) Nail Group Settlement vs. Time Relations

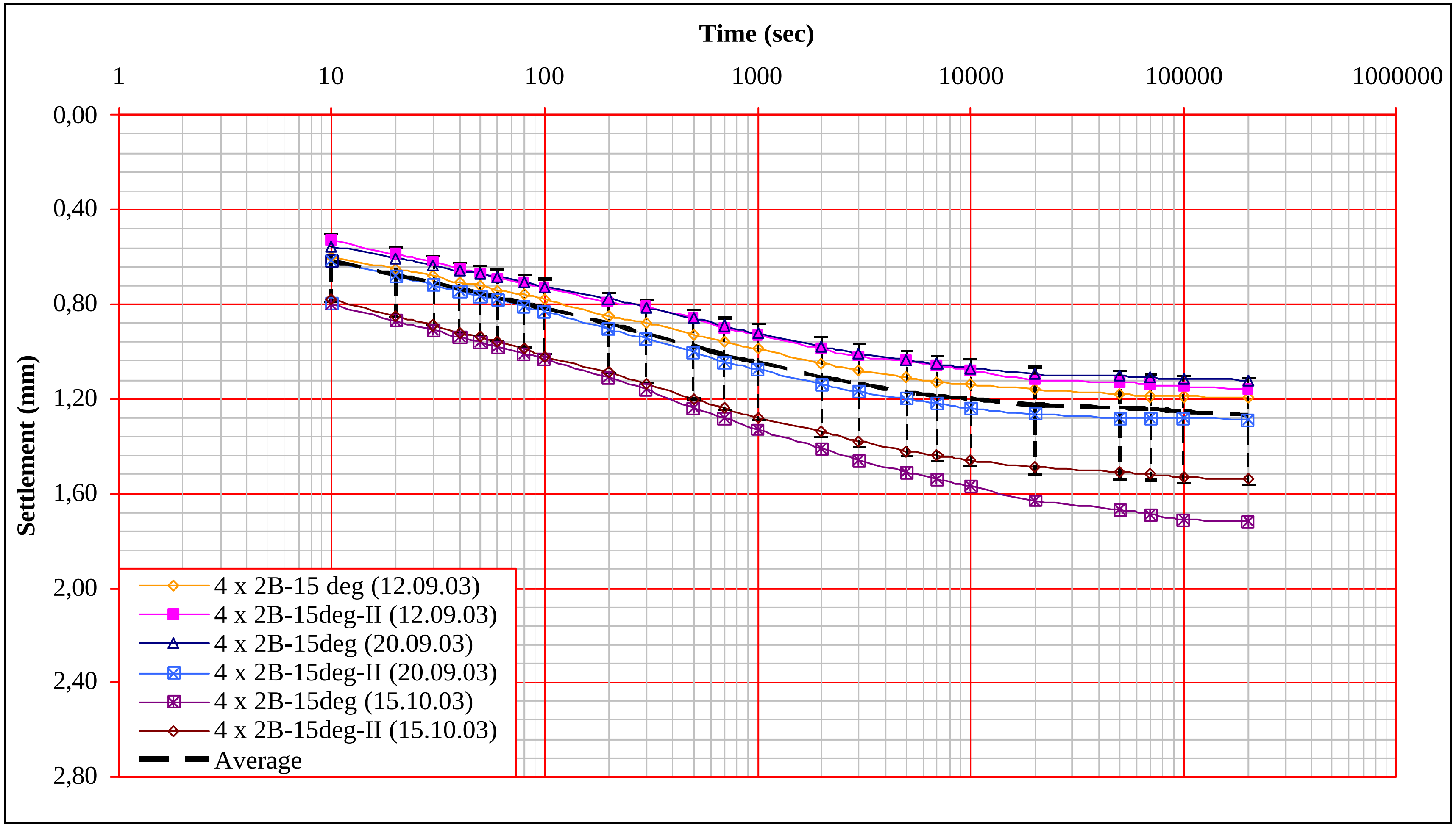


Figure 4.7 4x 2 B – 15^0 Inclined ($B= 30mm,\ s_f = 50\ kPa$) Nail Group Settlement vs. Time Relations

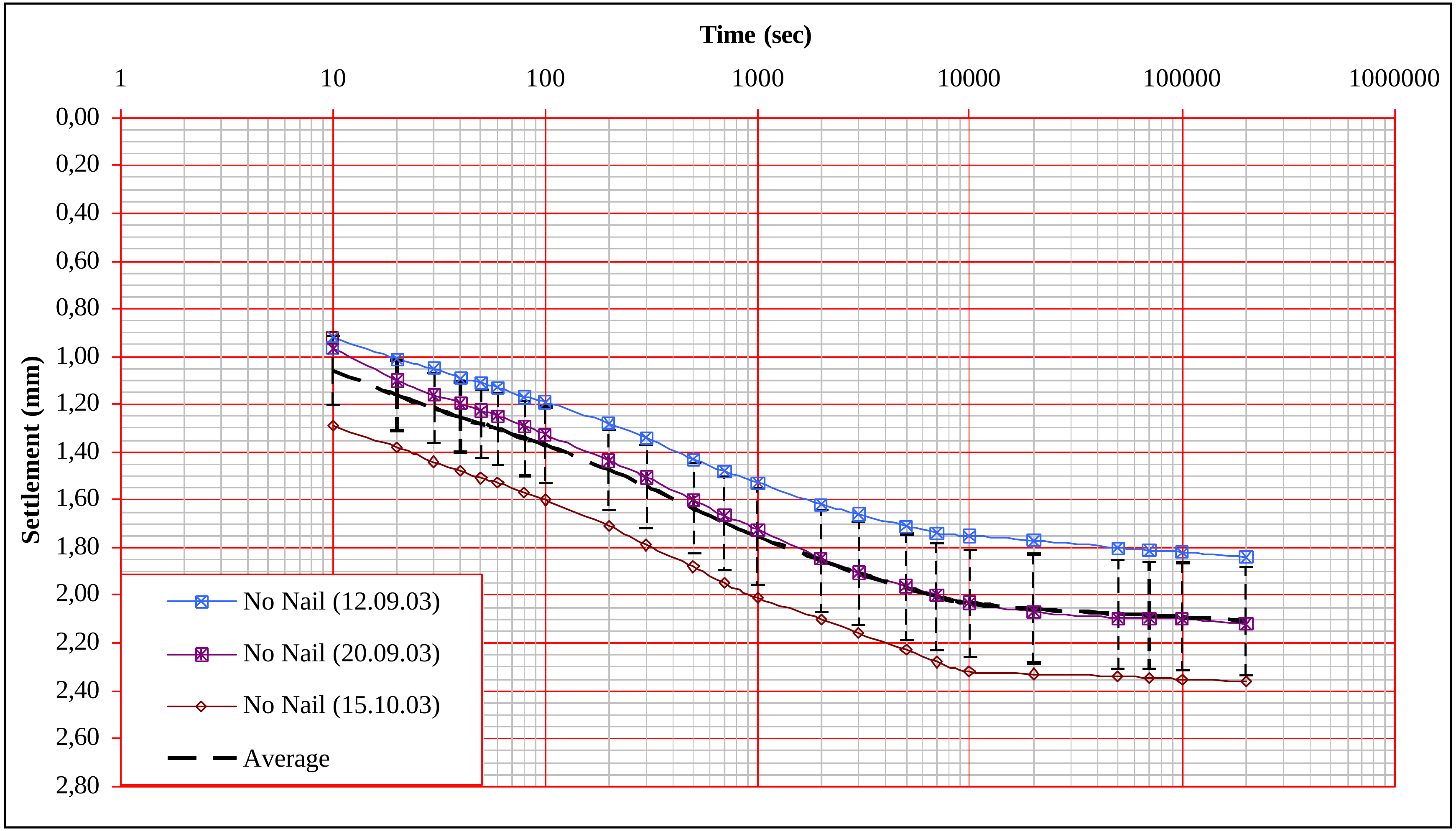


Figure 4.8 No Nail Case ($B = 30mm$, $s_f = 50\ kPa$) Settlement vs. Time Relations

Table 4.3 Summary of Test Results for the Investigation of Inclination Effect

B= 50 mm s_f= 50kPa	S_T (mm)	S_i (mm)	S_c (mm)	w_{bc}(%)	w_{ac}(%)	w_{at}(%)
4 x 1.5B	10,105	3,327	6,778	37,67	33,93	-------
	7,795	3,188	4,607	37,23	34,24	33,39
	9,150	3,696	5,454	37,58	33,18	33,77
	8,405	3,120	5,285	37,49	33,38	33,65
Average	8,450	3,335	5,115	37,49	33,68	33,60
St. Dev.	0,996	0,257	0,908	0,19	0,49	0,19
4 x 1.5B - 15^0 Inclined	10,320	3,418	6,902	37,42	34,35	32,10
	6,550	2,743	3,807	37,96	33,33	33,19
	7,780	3,146	4,634	37,30	34,89	33,75
	6,600	2,865	3,735	37,48	33,67	33,62
Average	6,977	2,918	4,059	37,54	34,06	33,17
St. Dev.	1,766	0,302	1,479	0,29	0,70	0,75
4 x 1.5B - 30^0 Inclined	7,795	3,993	3,802	37,75	33,85	31,79
	8,150	3,661	4,489	37,14	34,31	33,39
	8,195	2,978	5,217	37,57	34,87	32,91
	5,990	2,593	3,397	37,77	33,37	30,95
	6,581	2,715	3,866	38,15	33,41	33,73
	8,075	2,934	5,141	37,55	34,42	33,72
Average	7,759	3,256	4,503	37,66	34,04	32,75
St. Dev.	0,942	0,556	0,753	0,33	0,60	1,14
4 x 2B	6,870	2,521	4,349	37,55	33,69	33,74
	7,350	2,728	4,622	38,77	34,75	33,03
	6,760	2,903	3,857	38,28	34,73	31,92
Average	6,993	2,717	4,276	38,20	34,39	32,90
St. Dev.	0,314	0,191	0,388	0,61	0,61	0,92
4 x 2B - 15^0 Inclined	6,460	2,209	4,251	-----	34,17	33,64
	6,710	2,915	3,795	-----	33,45	33,28
	7,580	3,223	4,357	-----	34,08	34,00
	5,926	2,836	3,090	39,18	34,73	34,08
	6,530	2,731	3,799	38,51	34,84	34,52
	7,235	3,175	4,060	39,10	35,05	31,39
	8,560	3,684	4,876	38,63	34,58	32,86
Average	6,903	2,851	4,052	38,86	34,41	33,40
St. Dev.	0,873	0,461	0,557	0,33	0,55	1,04
4 x 2B - 30^0 Inclined	7,470	2,841	4,629	-----	33,72	33,72
	6,620	2,788	3,832	-----	34,17	33,64
	7,100	3,199	3,901	-----	34,13	33,65
	7,930	3,035	4,895	39,78	35,05	33,77
Average	7,285	3,020	4,265	39,78	34,27	33,70
St. Dev.	0,556	0,188	0,529		0,56	0,06

Table 4. 3 Continued

B= 50 mm σf= 50kPa	S_T (mm)	S_i (mm)	S_c (mm)	w_{bc}(%)	w_{ac}(%)	w_{at}(%)
6 x 2B	4,965	1,822	3,143	38,12	33,40	32,62
	6,250	1,962	4,288	37,49	33,44	31,92
	6,850	2,705	4,145	37,42	34,35	32,10
Average	6,022	2,163	3,859	37,68	33,73	32,21
St. Dev.	0,963	0,475	0,624	0,39	0,54	0,36
6 x 2B - 15⁰ Inclined	7,175	2,934	4,241	37,73	33,77	----
	6,440	2,465	3,975	37,86	33,28	32,03
	5,090	2,070	3,020	37,48	32,71	31,12
	5,590	2,400	3,190	37,51	33,65	32,93
Average	6,015	2,433	3,583	37,62	33,35	32,03
St. Dev.	0,922	0,356	0,593	0,21	0,48	0,91
6 x 2B - 30⁰ Inclined	6,880	2,369	4,511	38,08	34,14	31,76
	6,600	2,921	3,679	37,90	34,42	32,27
	6,870	2,777	4,093	37,56	34,75	33,20
Average	6,735	2,689	4,094	37,85	34,44	32,41
St. Dev.	0,159	0,286	0,416	0,26	0,31	0,73
No Nail	7,195	2,932	4,263	38,79	33,52	-----
	10,560	4,303	6,257	39,78	35,34	32,88
	11,020	4,382	6,638	38,31	34,58	32,43
	6,860	3,392	3,468	-----	32,85	32,75
	7,630	3,829	3,801	39,12	34,31	34,06
	8,727	3,563	5,164	38,00	33,76	33,76
	11,400	4,398	7,002	37,51	34,10	33,45
Average	9,484	4,019	5,465	38,59	34,07	33,22
St. Dev.	1,917	0,566	1,430	0,82	0,80	0,63

NOTE: Settlements values written in orange ink are not taken into account when calculating the average values.

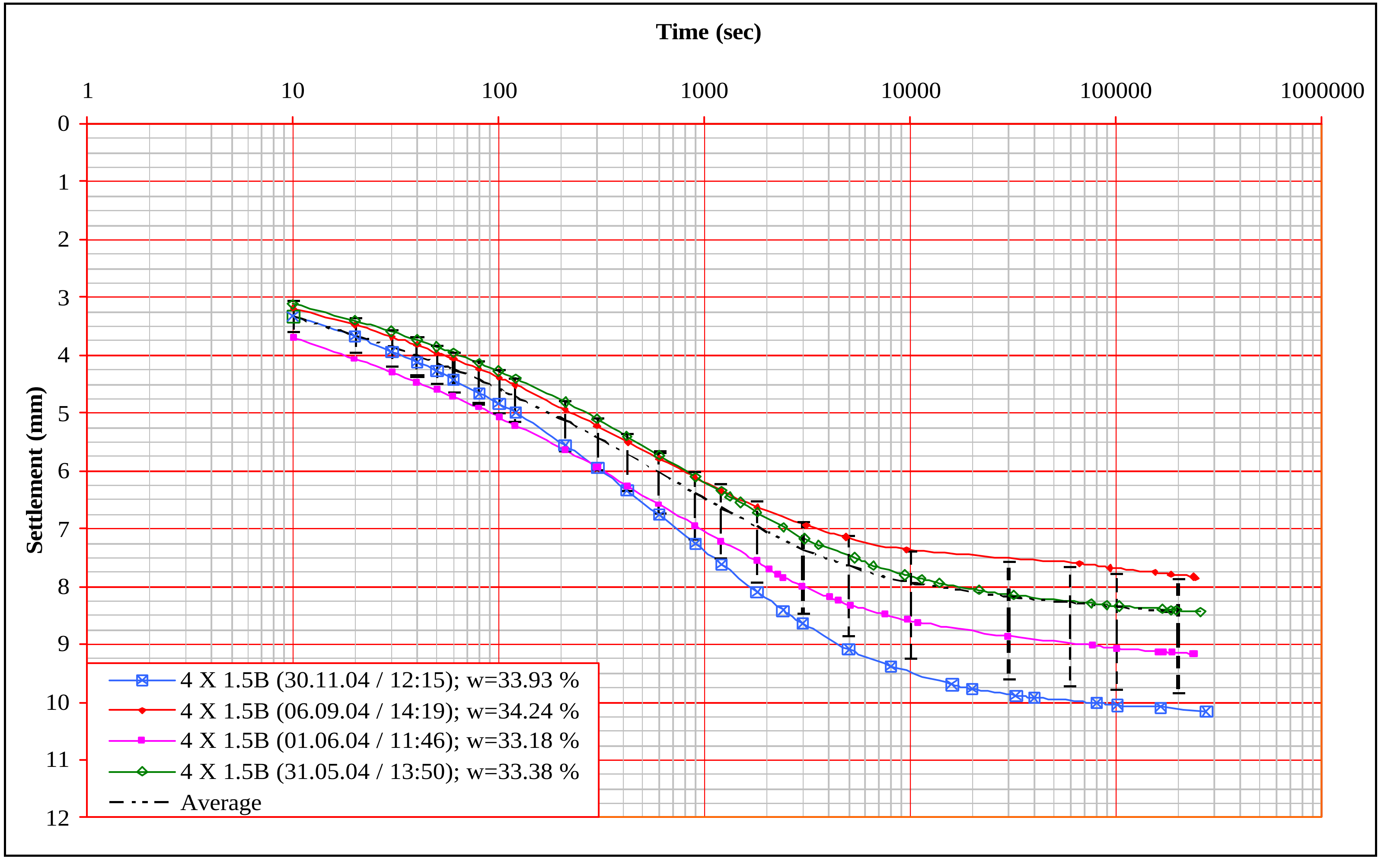


Figure 4.9 4 x 1.5 B (*B= 50 mm*, s_f= *50 kPa*) Nail Group Settlement vs. Time Relations

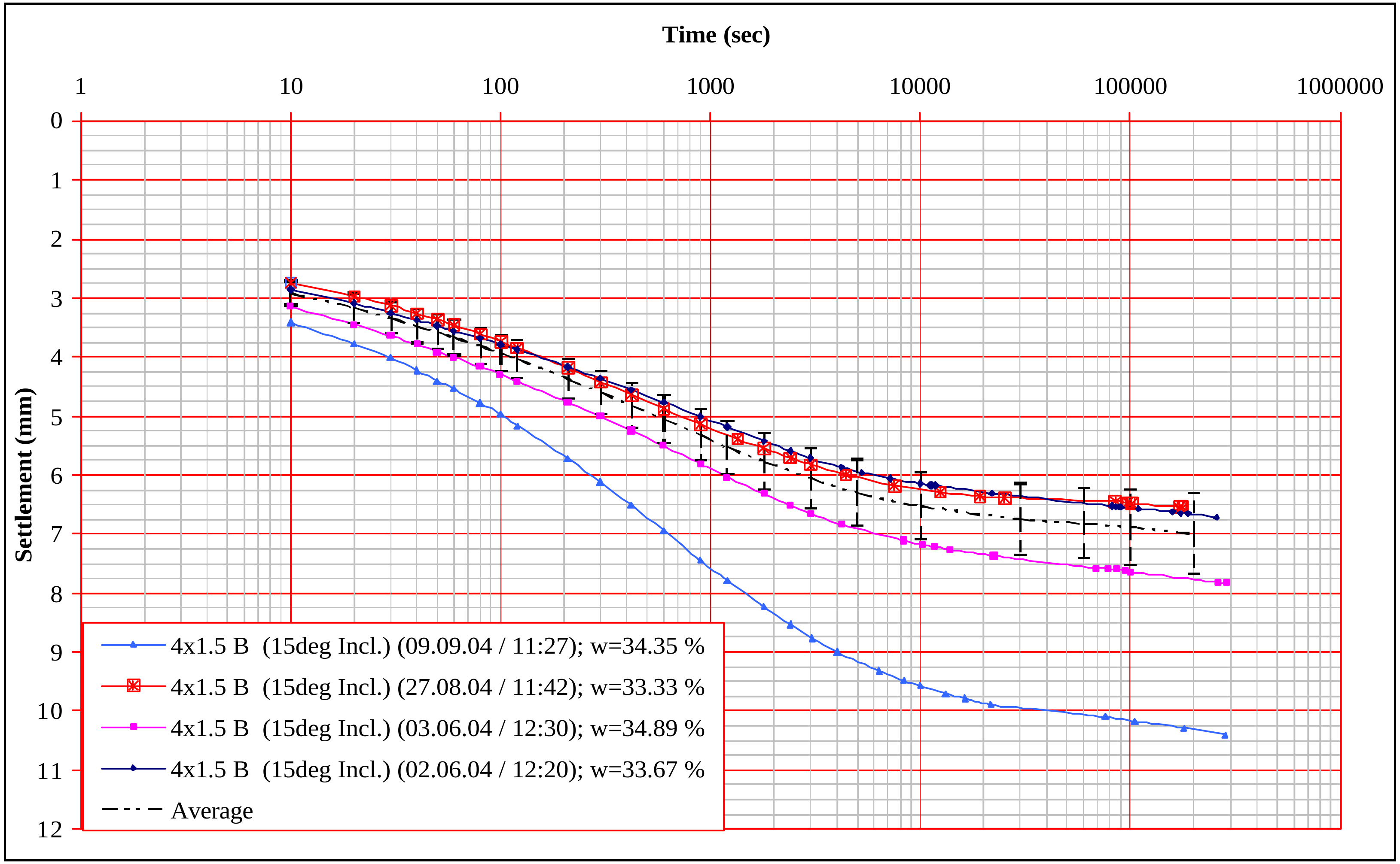


Figure 4.10 4 x 1.5 B - 15^0 Inclined (*B= 50 mm, sf= 50 kPa*) Nail Group Settlement vs. Time Relations

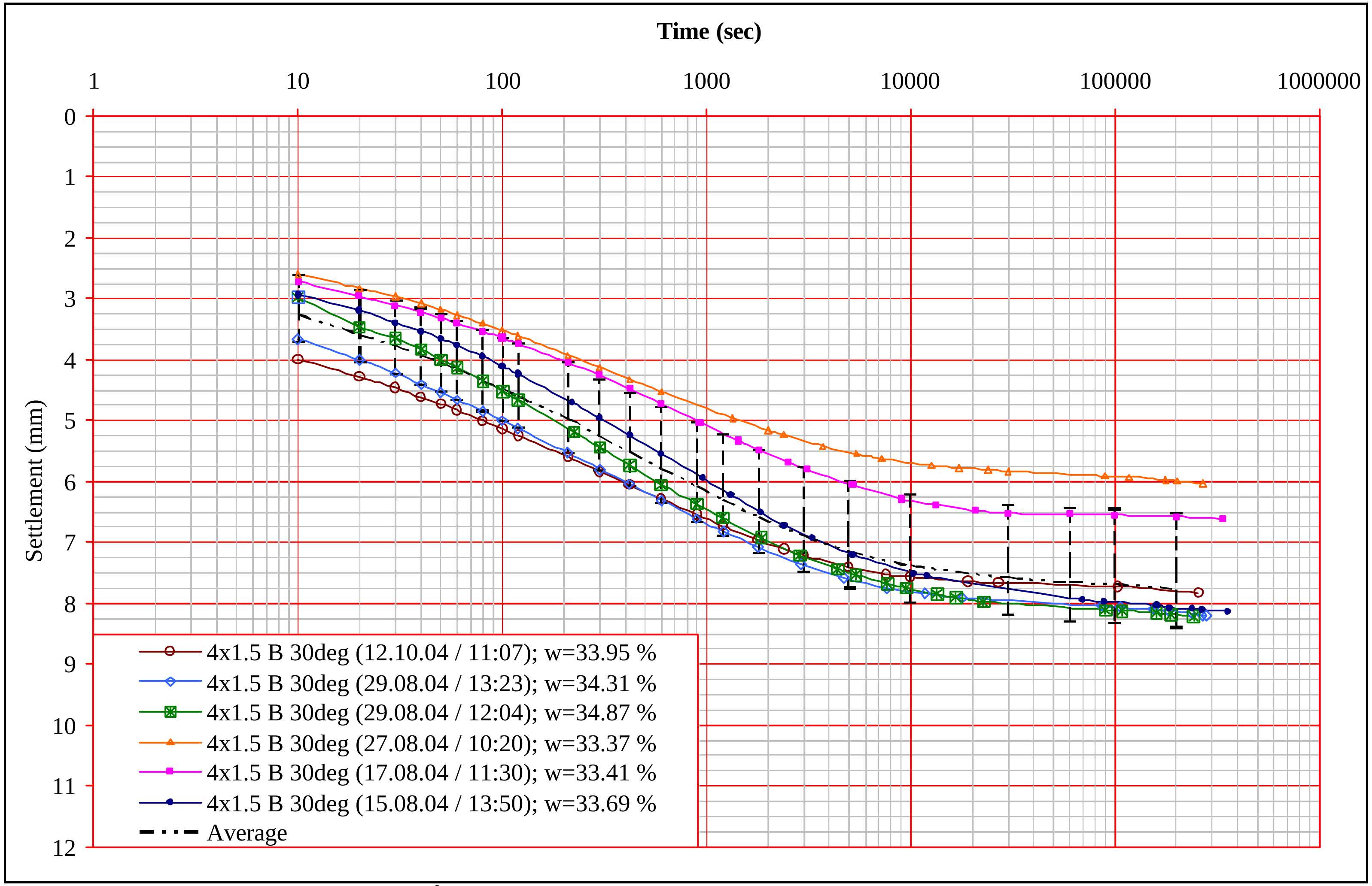


Figure 4.11 4 x 1.5 B - 30^0 Inclined (B= *50 mm*, s_f= *50 kPa*) Nail Group Settlement vs. Time Relations

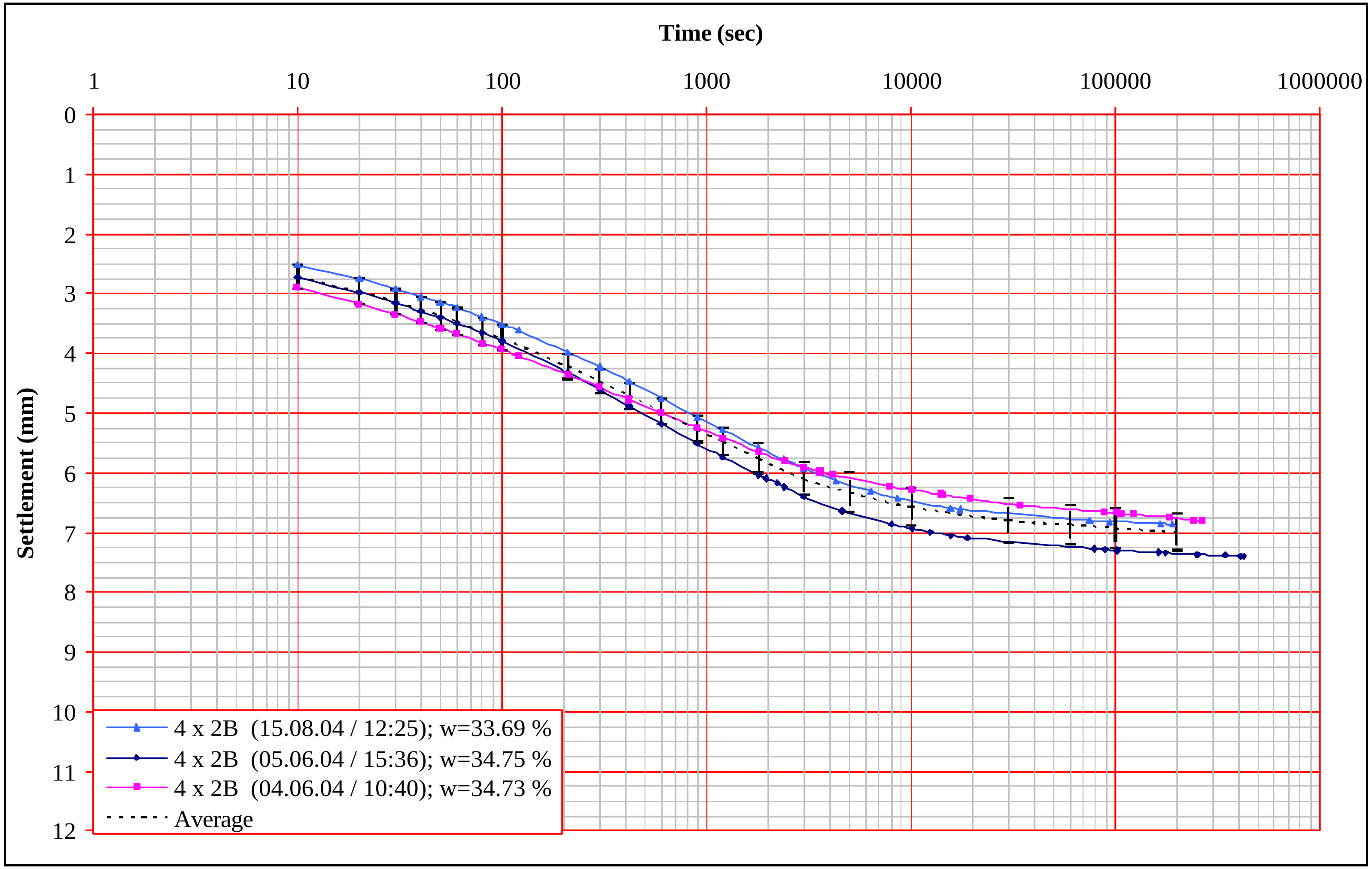


Figure 4.12 4 x 2 B (B= *50 mm*, s_f= *50 kPa*) Nail Group Settlement vs. Time Relations

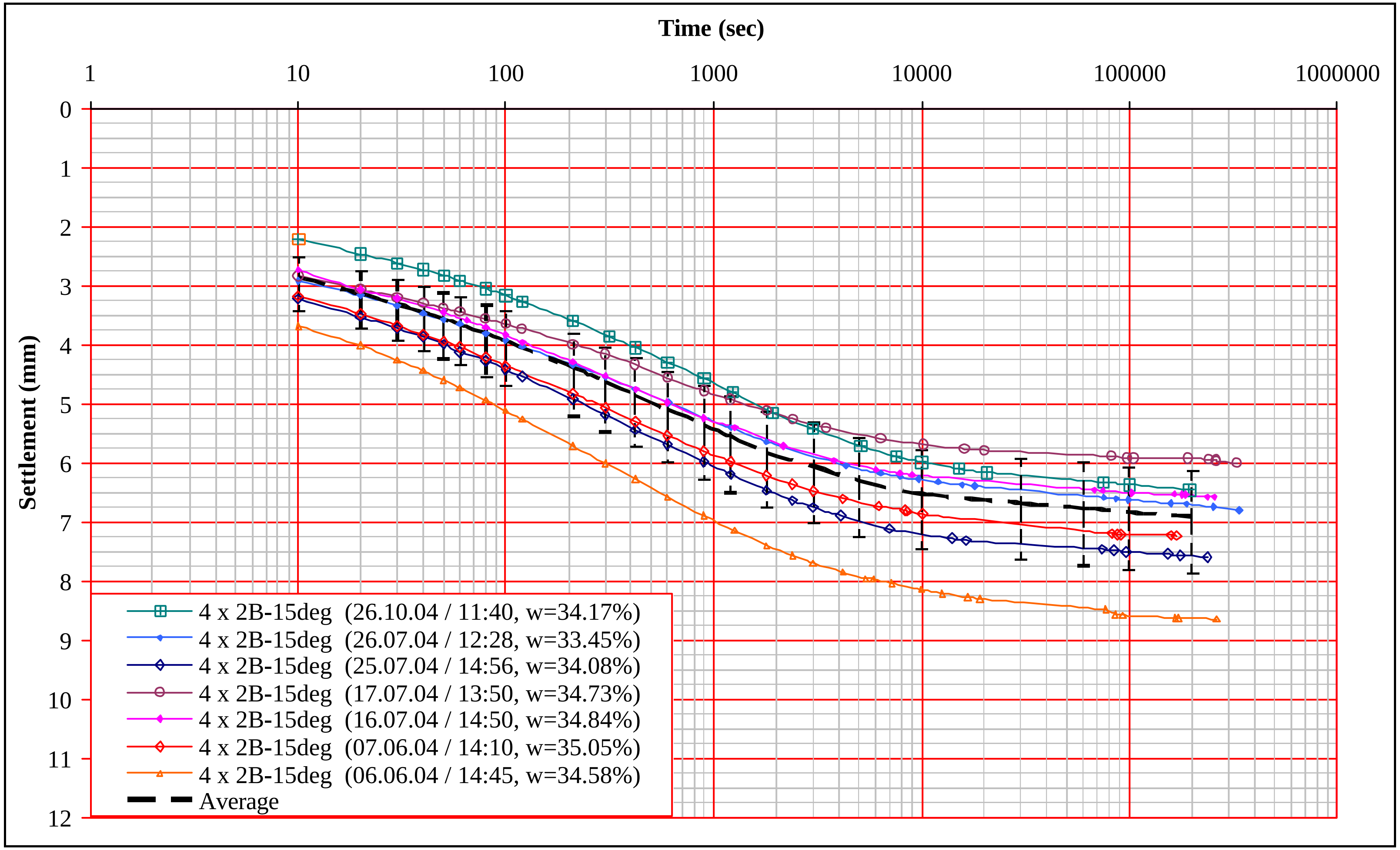


Figure 4.13 4 x 2 B - 15^0 Inclined (B= *50 mm*, s_f= *50 kPa*) Nail Group Settlement vs. Time Relations

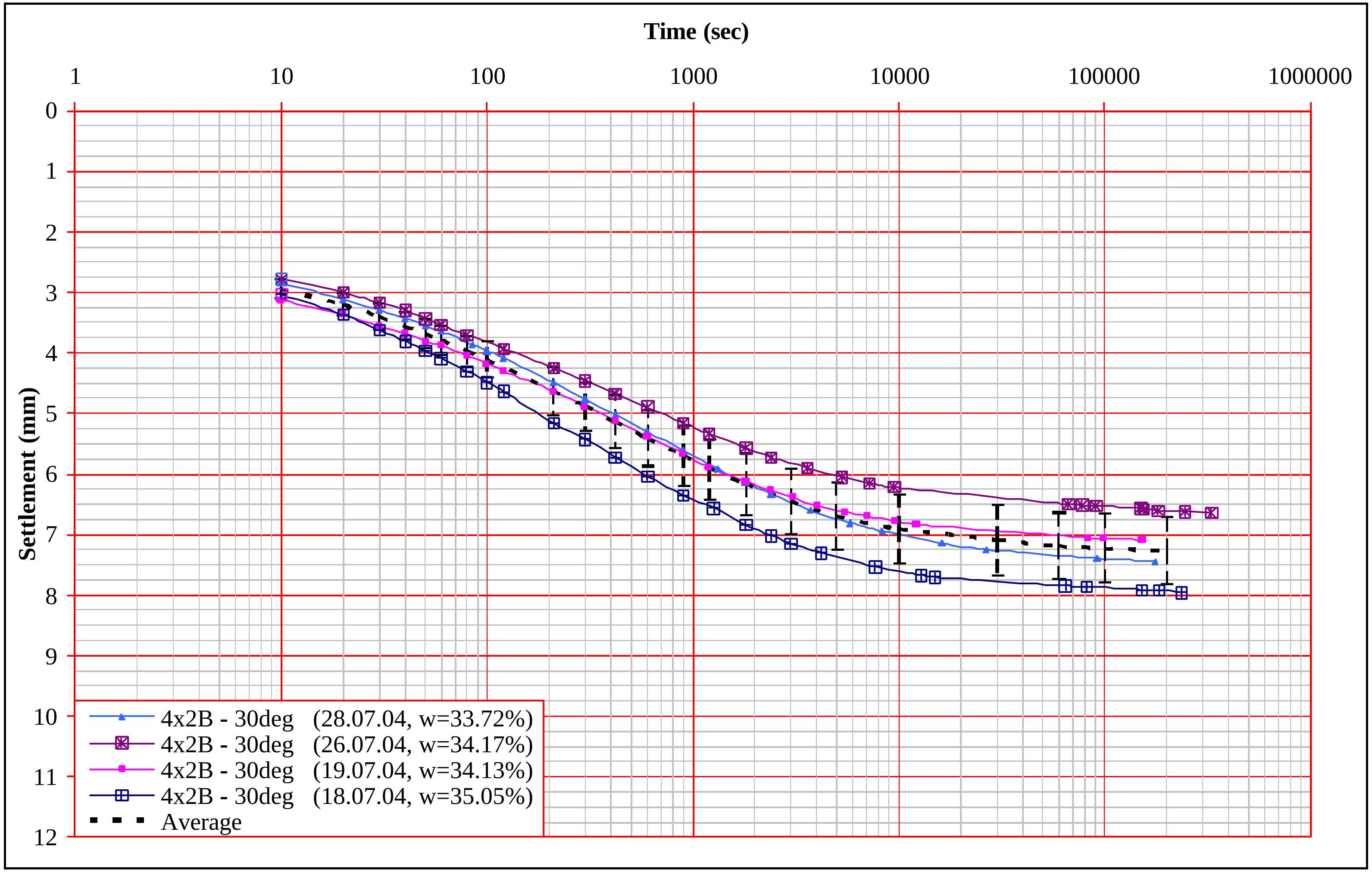


Figure 4.14 4 x 1.5 B - 30^0 Inclined (B= *50 mm*, s_f= *50 kPa*) Nail Group Settlement vs. Time Relations

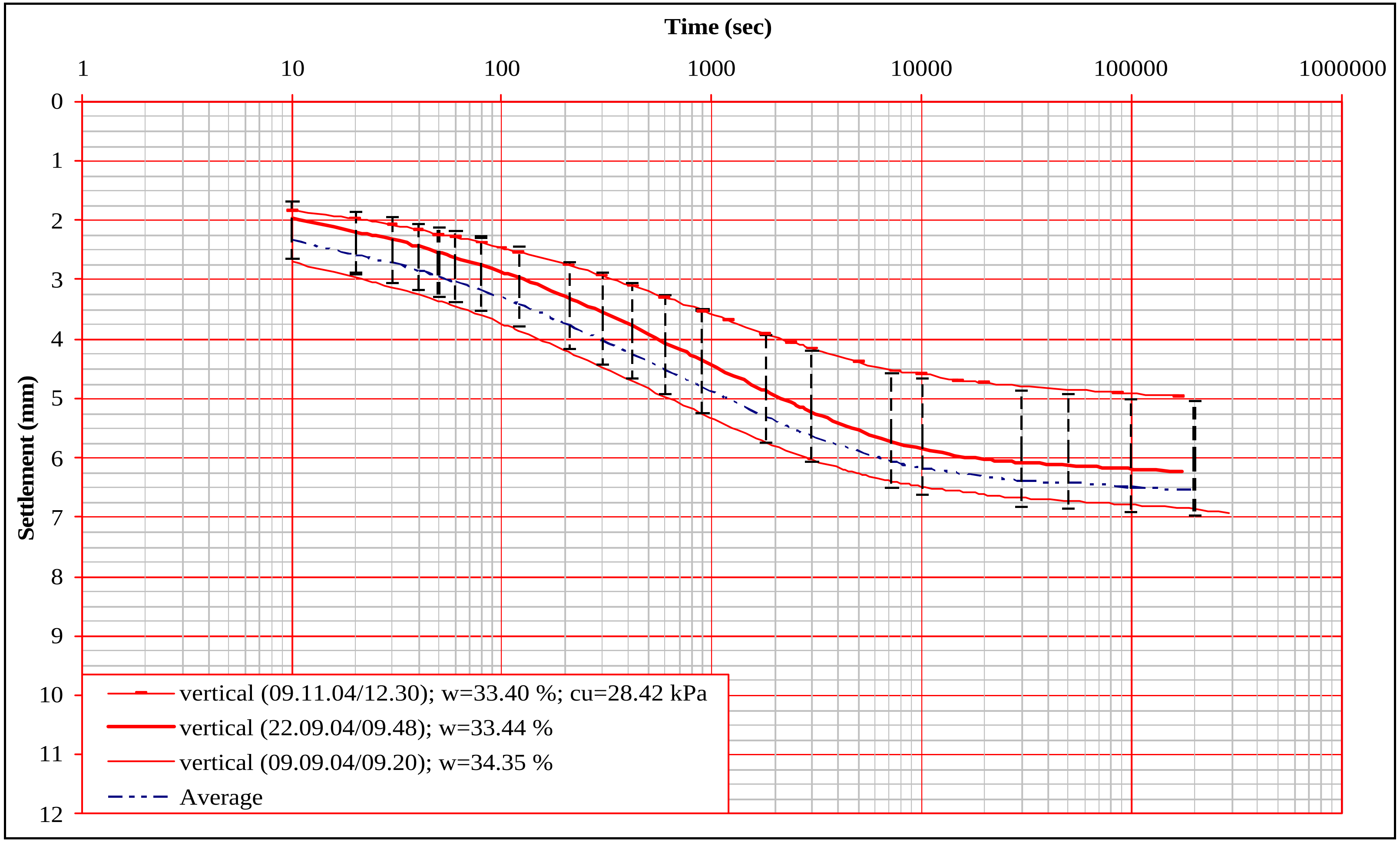


Figure 4.15 6 x 2 B (*B= 50 mm,* s_f*= 50 kPa*) Nail Group Settlement vs. Time Relations

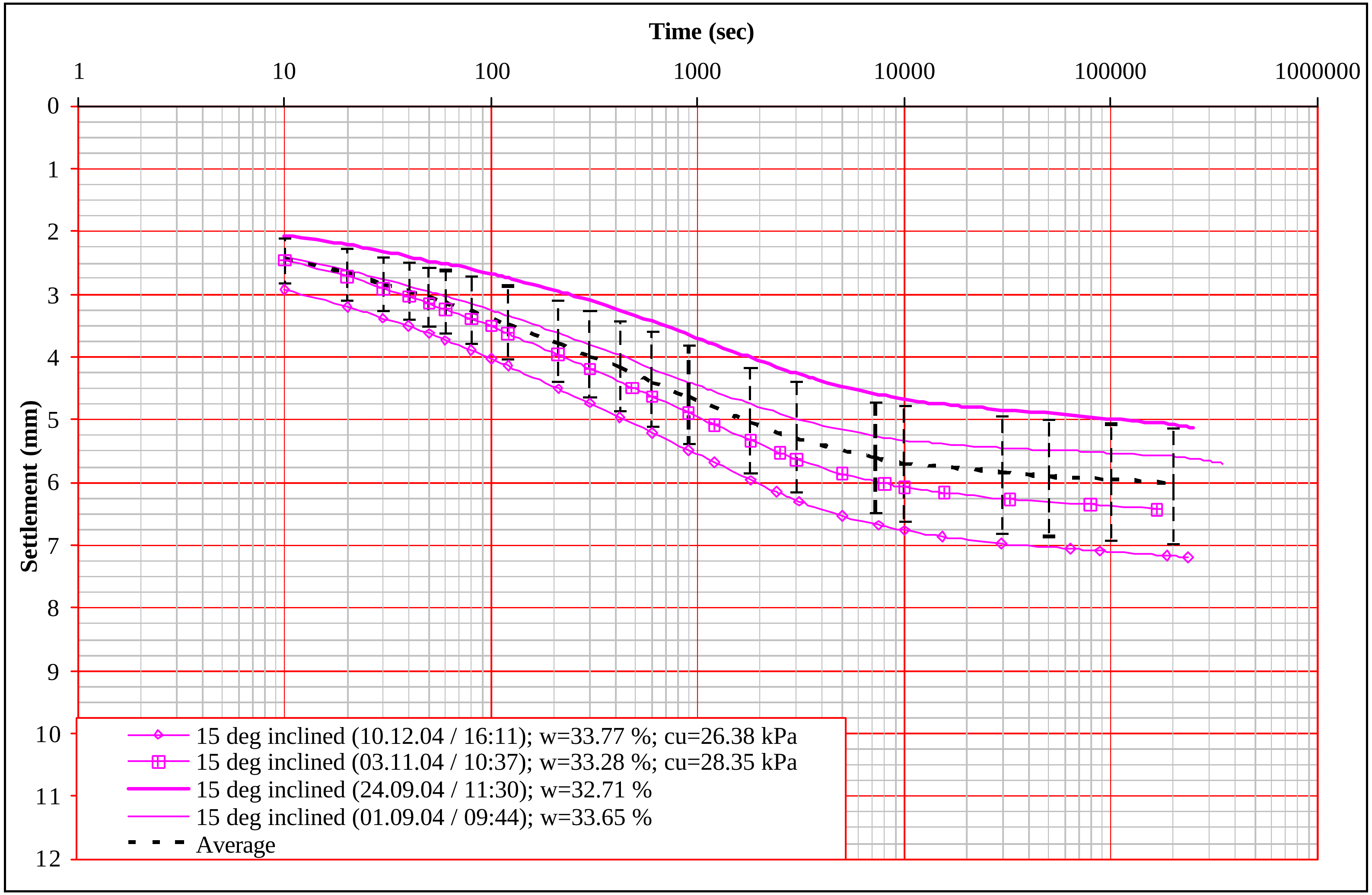


Figure 4.16 6 x 2 B - 15^0 Inclined (*B= 50 mm*, s_f*= 50 kPa*) Nail Group Settlement vs. Time Relations

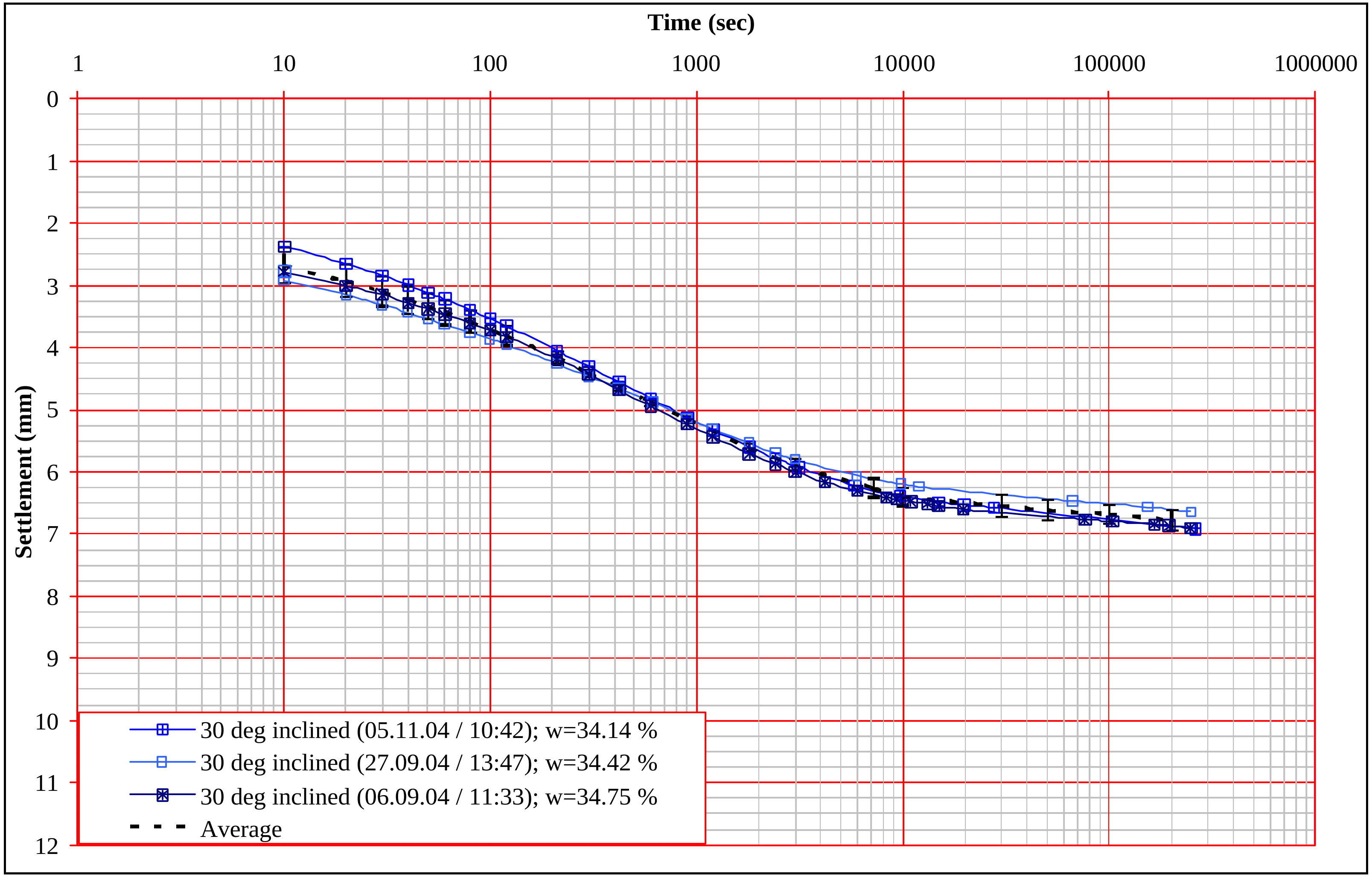


Figure 4.17 6 x 2 B -30^0 Inclined (B= *50 mm*, s_f= *50 kPa*) Nail Group Settlement vs. Time Relations

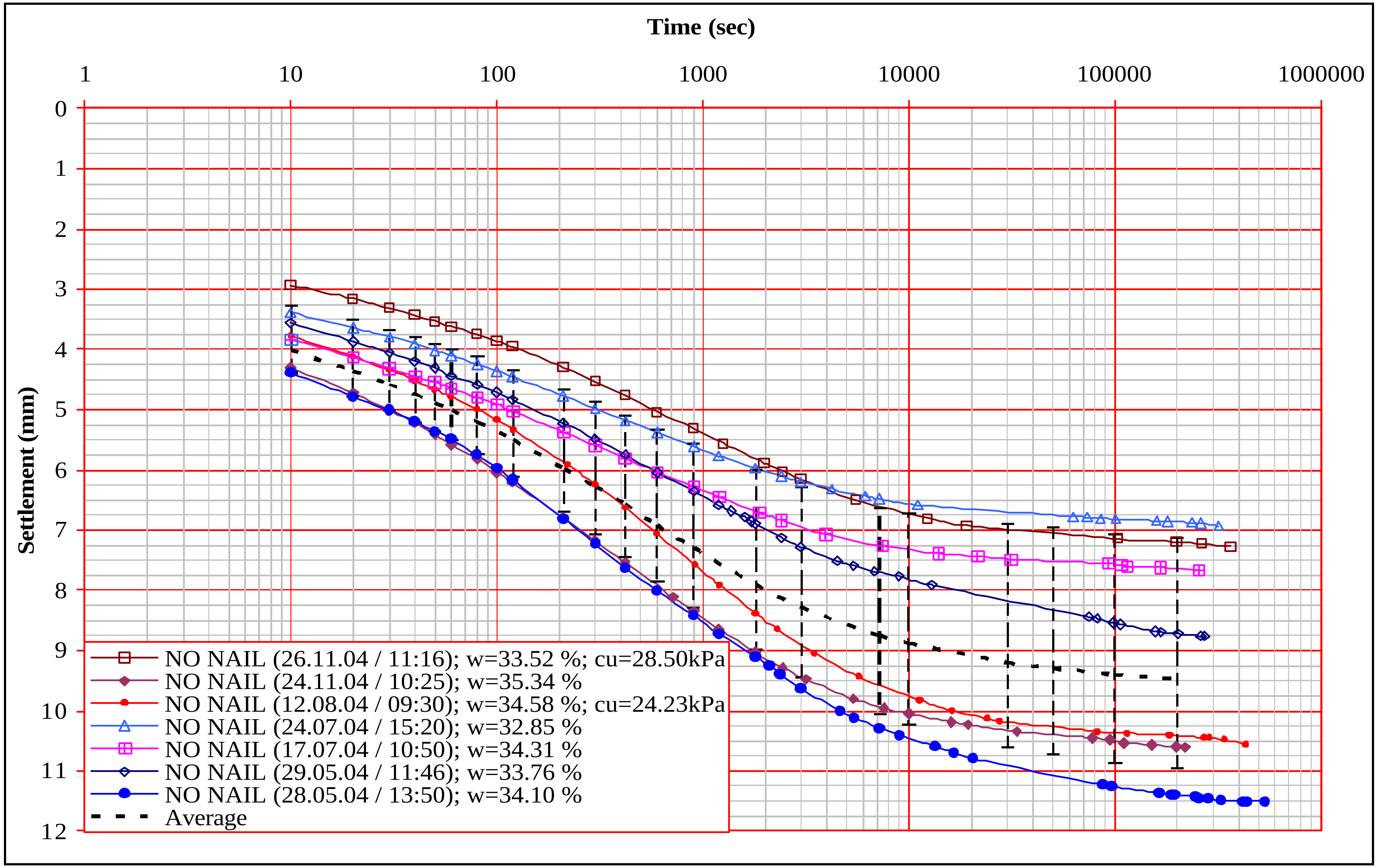


Figure 4.18 No Nail Case (*B= 50 mm*, s_f*= 50 kPa*) Settlement vs. Time Relations

5 DISCUSSION OF TEST RESULTS

5.1 General

The effect of soil nails applied in various patterns under rigid footing as settlement reducers was investigated through laboratory model tests by recording the settlement of model footing in the testing box. Settlement versus logarithm of time graphs are presented for different patterns of nails and no nail case and the summary of the test results are presented in tabular form in the previous chapter.

The settlements observed for the first part of the testing program, which is aimed to find out the contribution of number of nails to settlement improvement while the total length is being kept constant (B=30mm, σ_f =50 kPa) were in a range of 0.738 mm to 1.786 mm.

The second part of the testing program covers the investigation of the effect of nail inclination. In the first section, in which soil nails of 4 x 2B (B=30mm, σ_f=50 kPa) were tested for vertical and 15^0 inclined cases, the settlements observed was 1.10 mm and 2.36 mm. In the second section, in which soil nails of 4 x 2B, 4 x 1.5B and 6 x 2B (B=30mm, σ_f =50 kPa) were tested for vertical and 15^0 and 30^0 inclined cases, the settlements observed were in a range of 5.590 mm to 11.020 mm.

There are variations in settlement curves in the same series of tests. These may be due to the changes in the homogeneity, isotropy, water content and undrained shear strength of the prepared kaolin specimens and variations in the surface roughness of the soil nails. Problems in the displacement measurements caused some errors since the load is placed on the footing manually by a load hanger different impact forces on the footing and eccentricity due to improperly levelled hanger might result in different initial settlements. Throughout the testing period, in most of the specimens, differential settlements (tilting) had encountered. In order to come up to a conclusion, the test results are presented with average curves with their upper and lower ranges indicated.

In this chapter, the results are going to be discussed in two parts, because the scope of each part is different and also there are small differences in the testing procedure.

5.2 Nail Number and Length Relationships

In this part of testing program, keeping the total nail length constant the numbers thus individual nail lengths were changed. Settlement behaviour of 4, 5, 9 and 12 nails having lengths of 3 B, 2.4 B, 1.33 B and 1B respectively and no nail case, were investigated.

The amount of settlement and reduction in settlement for increasing nail lengths, which are presented in *Figure 5.1* and *Figure 5.2* show that decreasing the nail number, therefore using longer nails seems to decrease the total settlement. In other words, using 4 nails of 3B length resulted in best improvement level in this series of tests. On the contrary, using 12 nails of 1B length gave the largest settlement. *Figure5.1* shows the average settlements obtained for different nail lengths. There is a sudden drop from no nail case to 12 x 1B nail case, as expected. There is a small reduction in settlement while the nail number is reduced to 9 nails. In between 9 and 5 nails, there is also a slight reduction. But when the nail number is decreased from 5 to 4, the total settlement amount had changed drastically.

The observed behaviour above may probably be due to the contact stress distributions under rigid footings in clay (*Figure 5.3*). As explained in detail in Part 2.1, the contact stress distribution under rigid footing is such that it goes to yield stresses at the edges and decreases drastically at the centre (one half of the applied stress). Thus, as the nails are stress-transferring elements to lower strata, if they are placed at these stressed zones (i.e. corners); they may reduce the settlement of the footing more. On the contrary, if they are placed at the less stressed zones (i.e. centre), they will be less effective to reduce the settlement of the footing. Therefore, the longer the nails are placed at corners the less the settlements are expected to be.

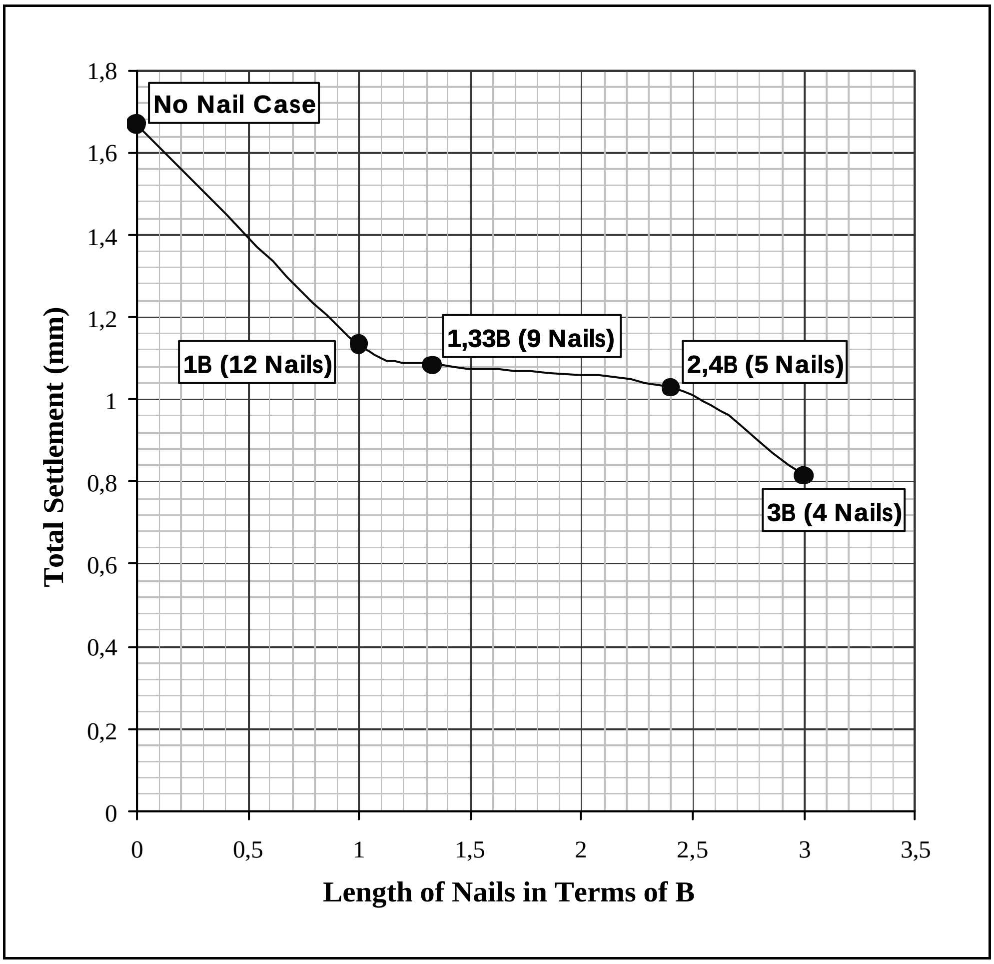


Figure 5.1 Total Settlement vs. Nail Length Relations (Total Nail Length is Constant in Each Group)

The transmission of the load to low stressed zones by increasing the length of nail could surely causes a reduction in settlement. Because, when the nail lengths are increased, the footing pressure is transmitted to deeper levels. Therefore, the thickness of the highly stressed zone, thus the amount of settlement is reduced more when longer nails are used. In other words, as the nails are long enough to reach beyond the highly stressed zones, they become more effective in improving the soil (i.e. reducing total settlements).

In the *Figure 5.2*, the percent reduction in settlement of the nailing patterns is presented. The improvement in settlement reduction, as explained in the previous paragraphs, has an increasing trend with nail length; hence decrease in number of nails. Improvement ranges are calculated by dividing the reduction in settlement to unimproved soil settlement. As discussed up to here, the reduction in settlement increases with the increase in nail length. Fig. 5.2 shows more than 50% reduction in settlement is obtained by using 4 x 3B nails and minimum 32% settlement reduction is obtained using 12 x B nails. Settlement reduction in the improved soil may be expressed by a ratio (R_s):

$$R_s = 100 \cdot \frac{S_{NoNail} - S_{Nailed}}{S_{NoNail}} (\%)$$

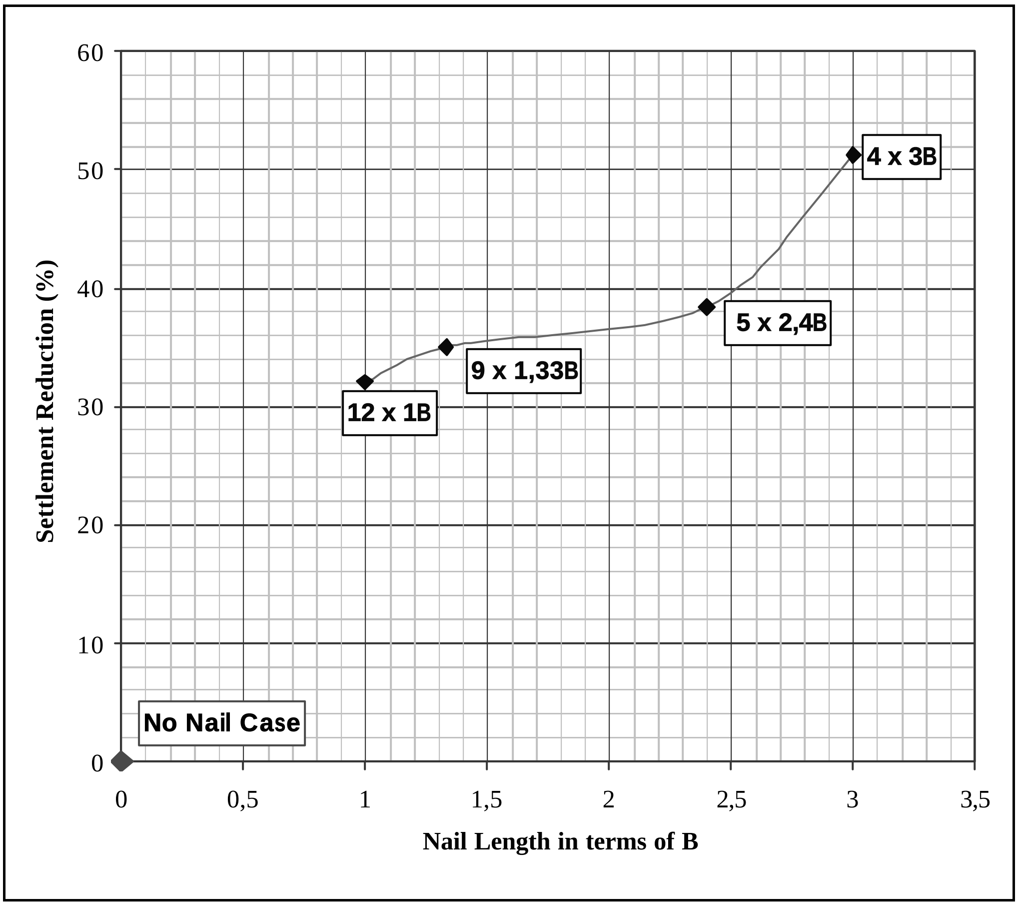


Figure 5.2 Percent Settlement Reduction of Nail Groups-Nail Length Relations (Total Nail Length is Constant in Each Group)

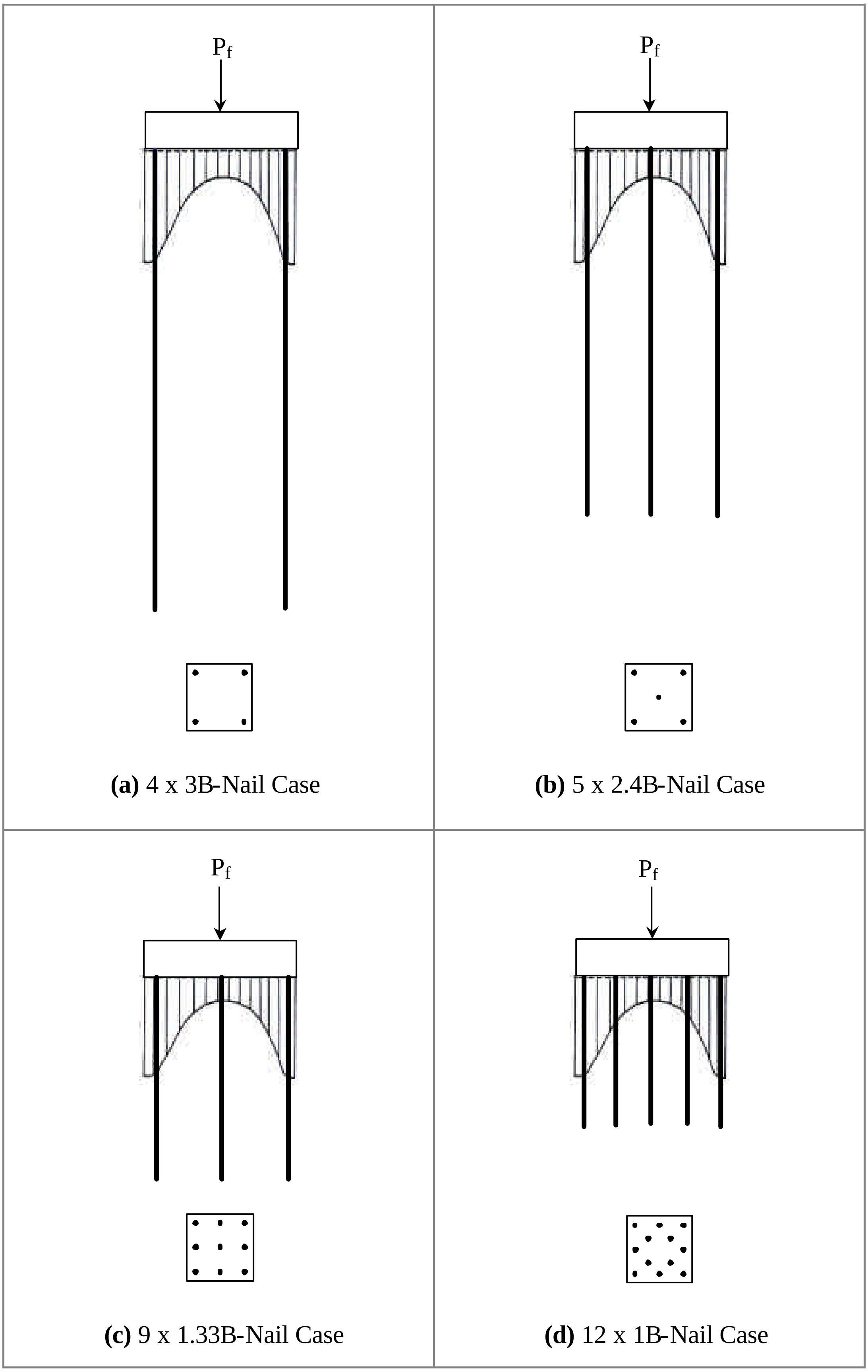


Figure 5.3 Sketch of Nailing Patterns and the Theoretical Contact Stress Distribution

5.3 Effect of Nail Inclination

In this section of the testing program, the effect of nail inclination was investigated through a series of tests.

In the first part of this section a comparative study had done between vertical and 15^0 inclined nails of 4 x 2 B (60mm) length. According to results obtained in this series of test, 15^0 inclined nails came out to be more effective in terms of settlement reduction (*Figure 5.5*).

In the second part (B=50mm), 4 nails and 6 nails, which were driven into soil vertically, 15^0 and 30^0 with vertical, and settlement of the groups were measured to clarify the effect of nail inclination. For the 4 nail cases, nail lengths of 1.5 B and 2 B were used. For the 6 nail cases only nails of 2 B length were used.

15^0 inclined nails, especially 4 x 1.5B Nail group yields the most effective improvement in terms of settlement reduction (*Figure 5.5*). The settlement and settlement reduction for increasing nail inclinations show that the total settlement seems to decrease at an optimum inclination angle of 15^0. But the reduction for the nailing patterns with nail lengths of 2B was not nearly the same for vertical and 15^0 inclined nail groups. For all of the nailing patterns, 30^0 inclined nails seem to be the least efficient pattern for the nail length of 2 B. For 4 x 1.5 B nail case, the least effective pattern was the vertical case. (*Figure 5.6*)

The settlement ratio variations of the vertical and inclined nails are given in *Figure 5.5*. Settlement ratio is defined as:

$$s/B = 100 \cdot \frac{S_{NoNail} - S_{Nailed}}{S_{NoNail}} \cdots (\%)$$

Kul (2003) results for the inclined nails in settlement ratio are given in *Figure5.4*. It is observed that the 1.5 B and 2B long nails are similarly affected. 3B long nails are also affected from different inclinations (In these series B = 30 mm, σ_f =66 kPa).

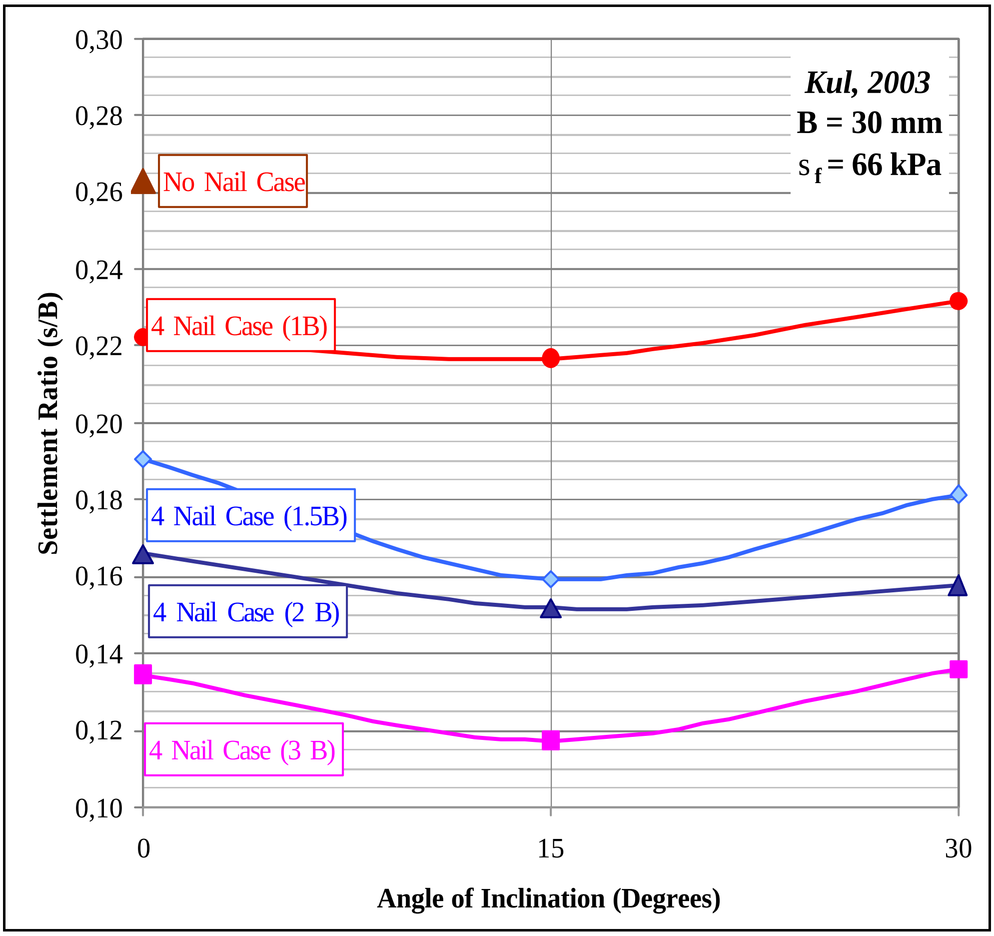


Figure 5.4 Settlement Ratio Variation with Angle of Inclination (After *Kul, 2003*)

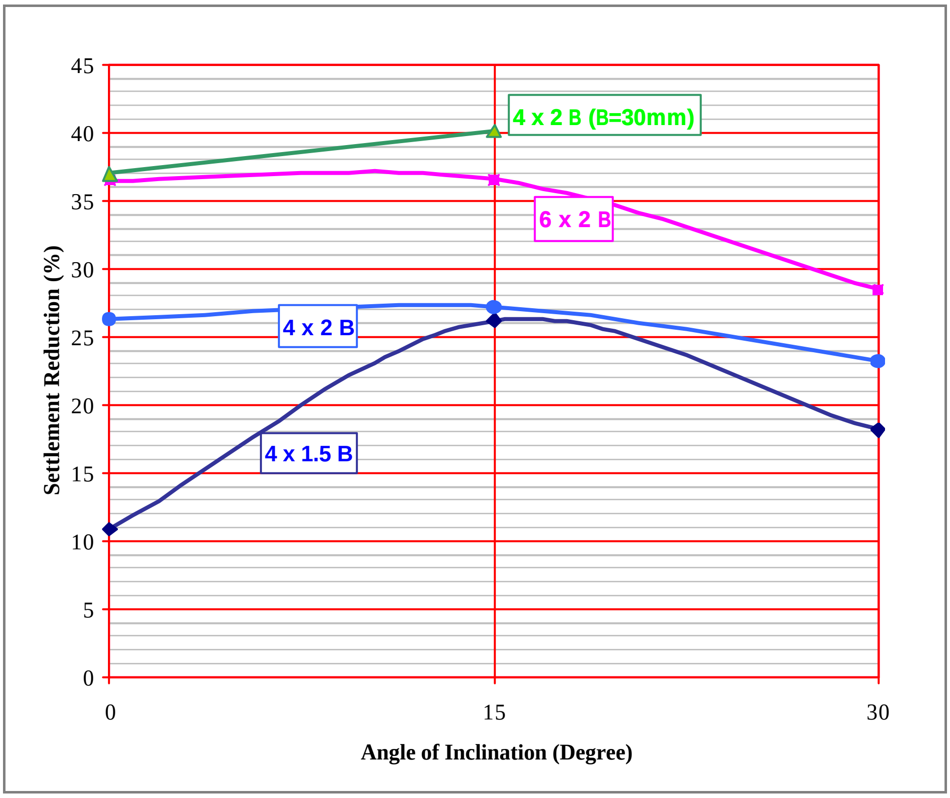


Figure 5.5 Settlement Reduction vs. Angle of Inclination

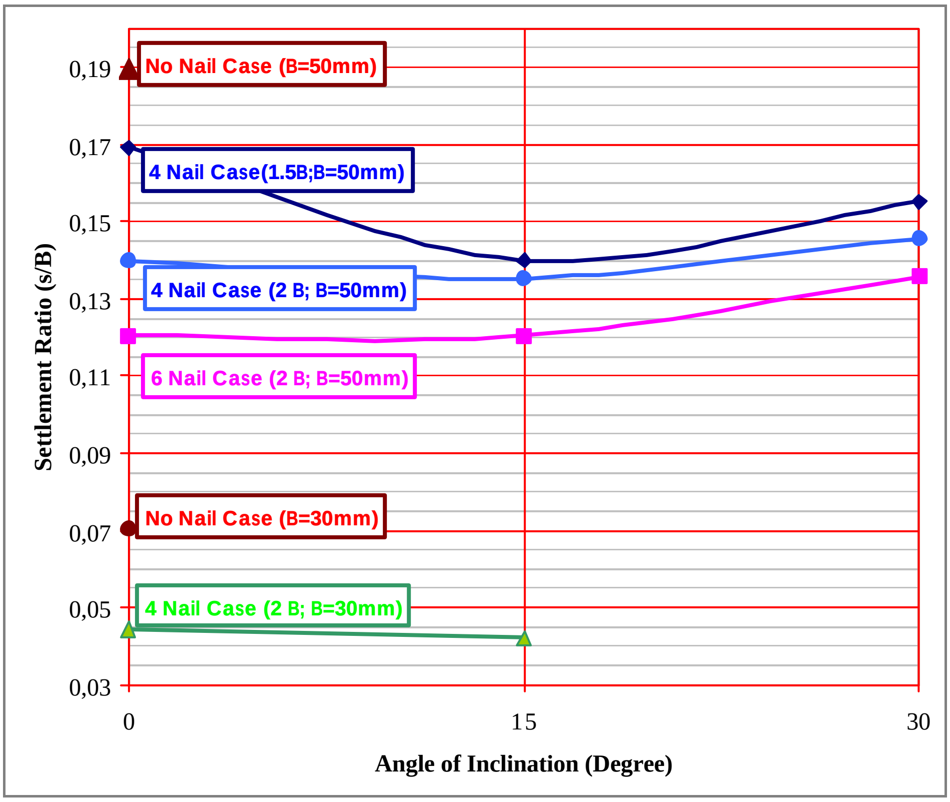


Figure 5.6 Settlement Ratio Variations with Angle of Inclination

The behaviour of settlement of nail-supported footings is complex while settlements take place. The behaviour of a nailed footing can be assumed to be affected from two mechanisms:

- Confinement effect of the nails
- Transfer of loads to deep strata

5.3.1 Confinement Effect of nails

Soft clay under footing loading when compressed it tries to deform radially. There is a tendency of soil flow similar to that of bearing failure mechanism, such that, soil tries to escape from the load and flow to the minimum stressed zone in the possible shortest path. This behaviour is accompanied with the surface heave observed at the edges of footing during testing, just as the load was applied. It seems that 15^0 inclination gives a better confinement most probably due to a more advantageous direction of displacement vectors under the footing. So, nails confine the stressed soil under the footing, while reducing the settlement by skin friction. As the soil is not allowed to escape from under the footing, settlement is reduced by prevention of small-scale subsidence.

In *Figure 5.6*, it can be concluded that 30^0 inclined nails gives the largest settlements except for the nails of 1.5 B lengths. In 4 x 1.5 B nailing cases, where the stresses could not be transmitted to low stress zones effectively.

In *Figure 5.7* possible yielded zones under footings are shown for nails of 1.5 B lengths. As soon as the load is applied, the coloured zones would be additionally stressed and foundation system reforms together with the footing and nails. Therefore the more this newly formed foundation system reaches deeper, the less settlement would be observed.

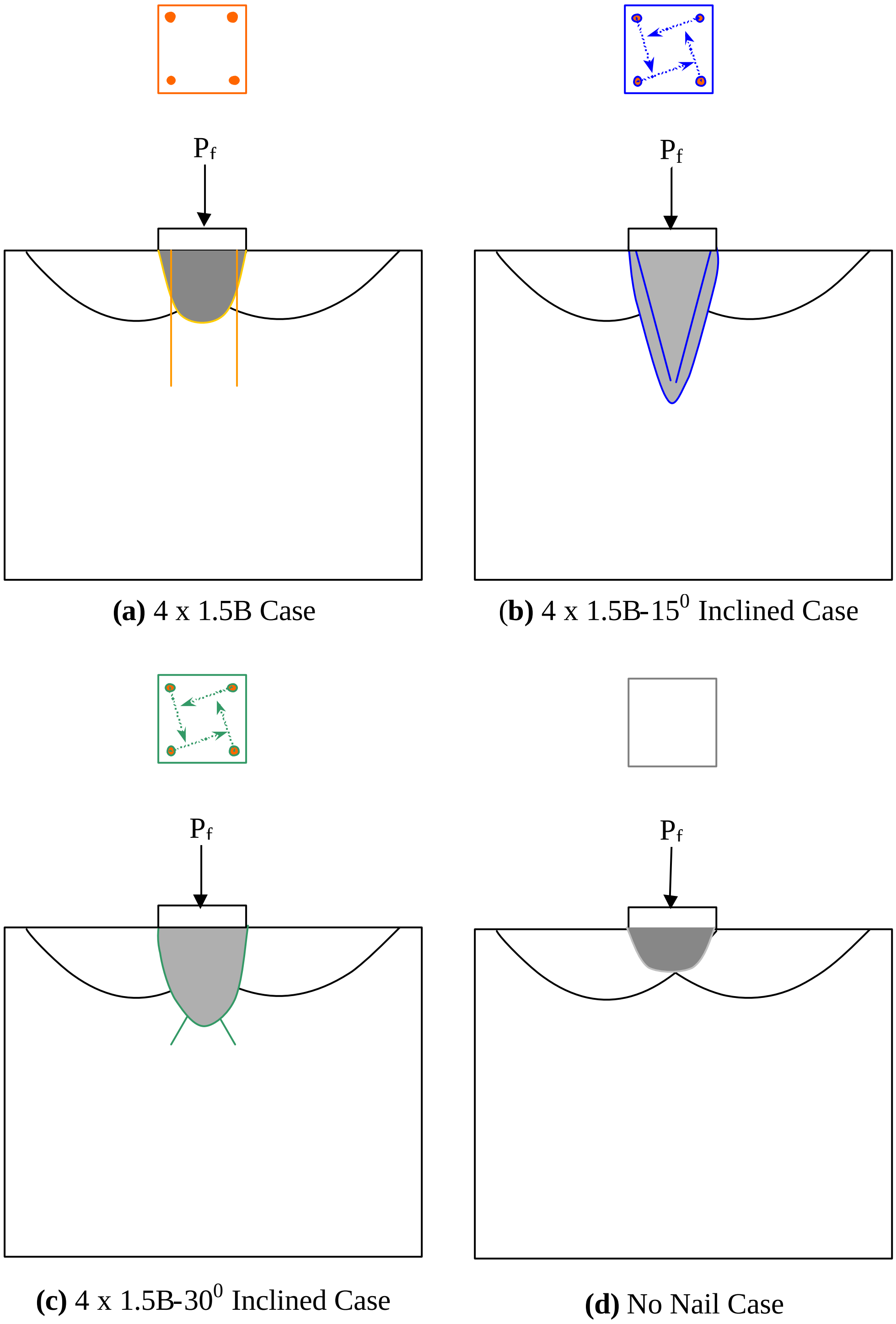


Figure 5.7 Possible Remoulded Zones of Nailed and Unreinforced Footing

5.3.2 Transfer of Loads to Deeper Strata

The nails also reduce settlement with skin friction. The skin friction is fully mobilized, because with a displacement less than half millimetres skin friction is mostly mobilised (*Figure 5.8*). Therefore, nails follow the settlement of footing and resist the movement with a fully mobilized frictional resistance develop on the nail surface.

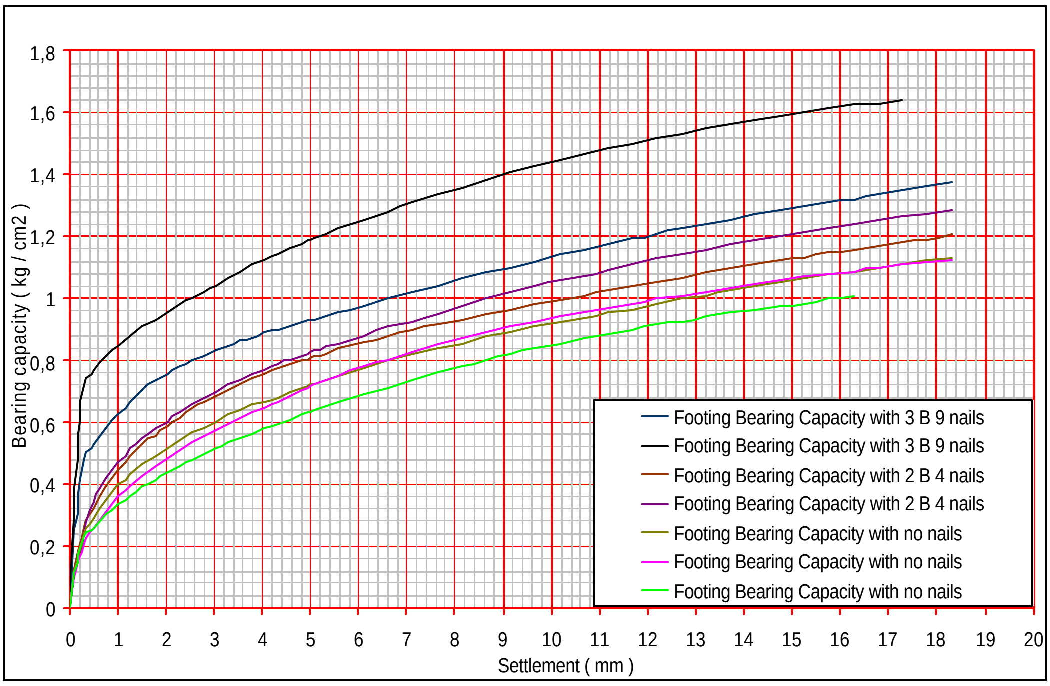


Figure 5.8 Bearing Capacity vs. Settlement Relations (After *Kul, 2003*)

There is also a limiting value for the inclination angle. Because, as the inclination angle increases its effectiveness decreases. In *Figure 5.9*the sketches of the two cases of inclined nailing patterns and the frictional forces acting on them are shown. The vertical resultant forces of nail skin friction are shown for two different inclinations of nails as, $F_{Rv15}{}^{o}$ and $F_{Rv30}{}^{o}$, respectively. For 15^0

inclined nails, $F_{Rv15}{}^o$ is large enough to keep nail movement without bending out of its axis. On the contrary, for 30^0 inclined nails, $F_{Rv30}{}^o$ is not large enough to prevent nail bending out of its axis and rather than a movement on its axis or a rotational movement may take place, which may cause more settlement.

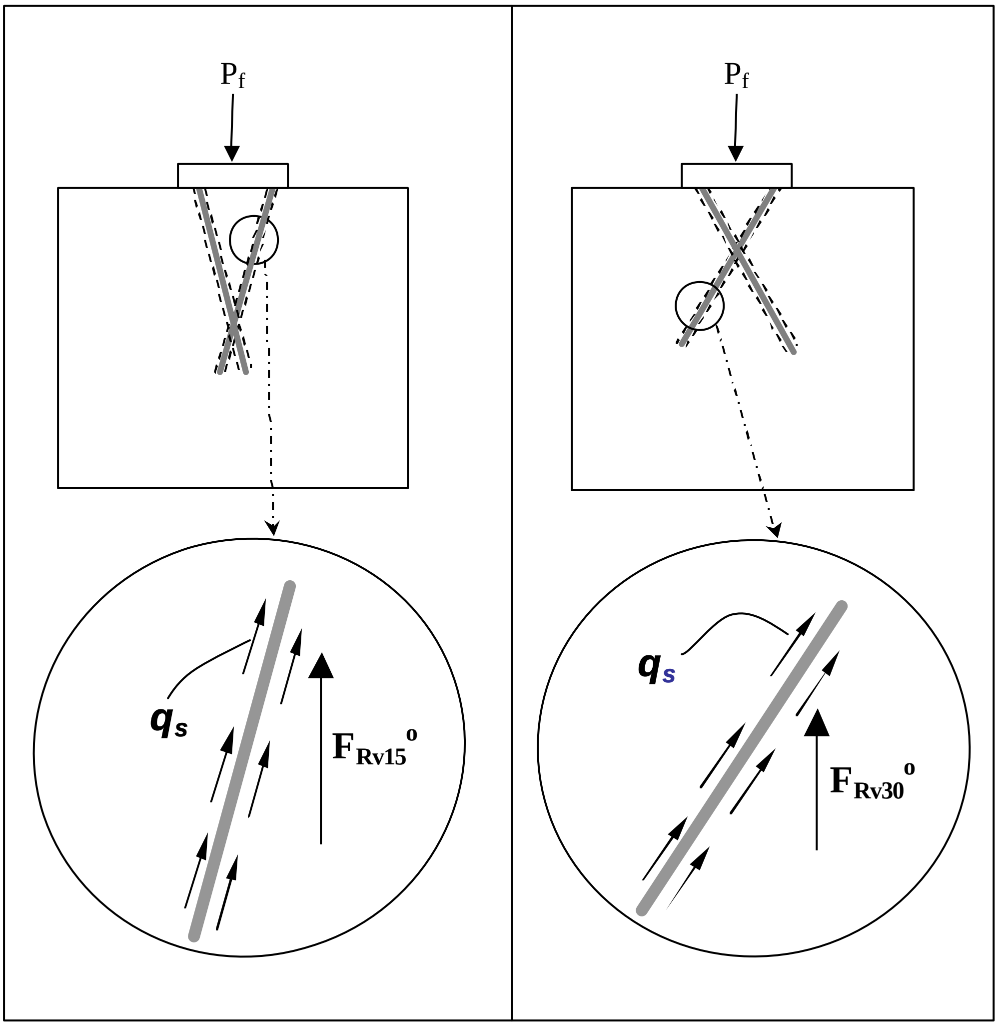


Figure 5.9 Sketches of Skin Friction Mobilized on the Inclined Nails

As the nail length was increased, the entrapment effect diminished and the bending and/or rotation problem dominated the behaviour of 30^0 inclined nails. This effect was explained in the previous paragraph and in Figure 5.8. On the other hand, vertical nails had become more advantageous compared to 30^0 inclined nails, when longer nails were used.

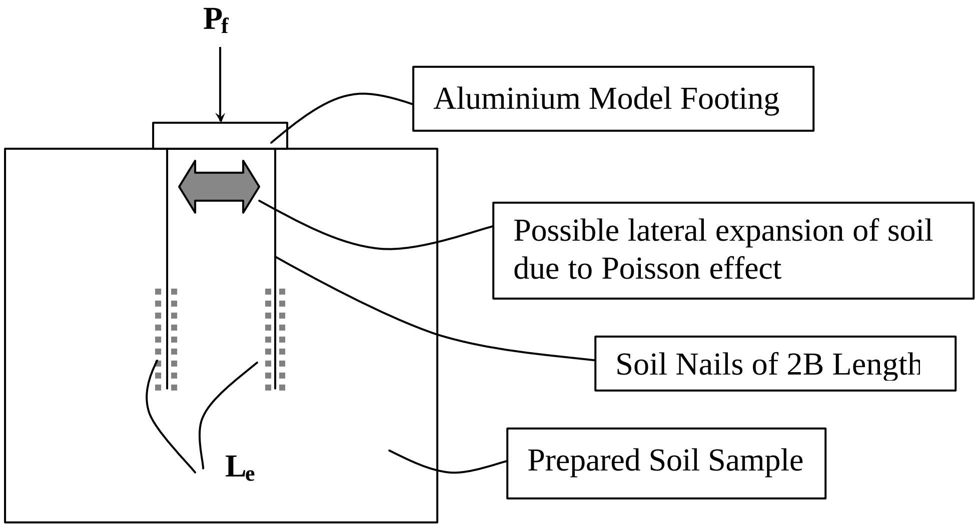


Figure 5.10 Sketch of Poisson Behaviour of Soil and Embedment Effect

Considering the bending of nails out of their axis, the embedment effect also comes into scene. The embedment effect is the increase in supporting soil length, L_e (effective embedded length) around the end portions of nails that reach the low stressed zones (*Figure 5.10*). So, as L_e increases the lateral soil expansion under the footing can be reduced more effectively.

6 CONCLUSIONS

In this model study, settlement behavior of nailed footings was investigated through 1g physical model in laboratory conditions. Several tests were made using nails under footings with different lengths, numbers and inclinations.

In the first section of the testing program, which was for the investigation of the most effective (i.e. giving the least settlement) nail length-to-number relationship keeping the total nail length constant, rate of decrease in settlements with respect to nail lengths varied non-linearly. Keeping the total nail length constant, using less number thus longer individual nails give less total settlements. Namely, improvement in settlement reduction was as much as 50% and the maximum reduction in settlement was obtained for the pattern of 4 nails (4 x 3B).

In the second section of the testing program for the investigation of the effect of nail inclination, nails with 15^0 inclinations with the normal of the ground give the least total settlements. In other words, the most effective pattern in terms of settlement reduction is the 15^0 inclined nail cases (for both of the 4 and 6 nail cases).

For the ease of applicability and traditional use in practice is considered, the use of vertical nails would be advisable.

It is clearly observed in this model study that use of nails under footings as settlement-reducing members has a great potential in shallow foundations, since the major criteria is the satisfaction of allowable settlements. Using adequate number and length of nails before placing the footing can reduce and keep the settlements in allowable limits. Therefore, use of nails under footings can be an effective and cheaper way to reduce settlement, since nails are ordinary construction steel and the application procedure does not require a formwork, concrete work… etc.

Recommendations for future research:

- Full scale field-testing and case records are required before establishing the design rules.
- In order to reduce the variations in settlements the model size should be increased.
- For a better representation of field conditions and construction period, the footing loading on the improved soil media should be increased gradually.
- Although there are studies for improvement of bearing capacity of existing footings on sand, the behaviour of soil nails under footings in various soil types can be investigated in order to check the improvement in terms of bearing capacity criteria.

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