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M.Sc. in Civil Engineering

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UNIVERSITY OF GAZİANTEP

GRADUATE SCHOOL OF

NATURAL & APPLIED SCIENCES

INDEPENDENT COMPARISON BETWEEN PLAXIS 2D AND DEEPXCAV FOR THE DEEP EXCAVATION PROJECT IN A CASE STUDY

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For The Deep Excavation Project In A Case Study

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

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I hereby declare that all information in this document has been obtained and Presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Nebahat YILMAZ

ABSTRACT

INDEPENDENT COMPARISON BETWEEN PLAXIS 2D AND DEEPXCAV FOR THE DEEP EXCAVATION PROJECT IN A CASE STUDY

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The purpose of a deep excavation support system is to provide lateral support for the soil around an excavation to limit the wall deflections and ground movements. If the wall deflections and ground movements are excessive during the excavation process, severe damages to surrounding buildings, roads and infrastructures may occur and in the worst case, collapse of the excavation system itself. In this thesis, in order to make analysis for a case study in deep excavation two common softwares were used. One of them is Plaxis 2D and the other one is DeepXcav. Both of them are very beneficial and common programs to make design and analysis for geotechnical works. When they are compared with each other, there are some differences among them. Making an analysis with Plaxis 2D is easy, reasonable and reliable, on the other hand for some cases DeepXcav is more remarkable than Plaxis 2D. Therefore, the comparison results that were obtained from Plaxis 2D and DeepXcav were observed and presented in this study.

Keywords: Deep excavation, Plaxis 2D, DeepXcav, Retaining wall, Case study.

ÖZET

BİR SAHA ÇALIŞMASI OLAN DERİN KAZI PROJESİ İÇİN PLAXİS 2D İLE DEEPXCAV ARASINDA BAĞIMSIZ BİR KARŞILAŞTIRMA

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Derin kazı destek sisteminin amacı, zemin kaymasını ve iksa sisteminin hareketini kazı boyunca engelleyerek yatay bir destek sağlamaktır. Kazı esnasında zemindeki kayma ve duvardaki hareket aşırı seviyede olursa, yolların, binaların ve büyük yapıların etrafında önemli zararlar oluşabilir. Hatta, bu yapıların çökmesine devrilmesine sebep olabilir. Bu tez kapsamında, gerçek bir saha uygulaması üzerinde çok yaygın kullanılan iki geoteknik mühendisliği yazılım programı, geoteknik analizleri yapmak için kullanılmıştır. Bu programlaradan biri Plaxis 2D, diğeri ise DeepXcav programlarıdır. Bu programların her ikiside geoteknik projelerde analiz ve tasarım yapımında kullanılan çok faydalı ve dünyada yaygın bir kullanıcı kitlesine sahip iki programdır. Bu programları birbiriyle karşılaştırdığımızda, aralarında bazı farklar vardır. Plaxis 2D ile analiz yapmak çok kolay, gerekli ve güvenilirken, bazı durumlar için DeepXcav programını kullanmak daha dikkate değerdir. Bundan dolayı, Plaxis 2D ve DeepXcav programlarıyla yapılmış analiz sonuçları bu tez kapsamında gözlemlenmiş ve birbirileriyle karşılaştırılarak tartışılmış ve sunulmuştur.

Anahtar Kelimeler: Derin kazı, Plaxis 2D, DeepXcav, İksa sistemi, Saha çalışması

Dedication

To

My parents

Who supported & encouraged me from the beginning of my life up to now

All my teacher

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CHAPTER 1

INTRODUCTION

1.1. Background

Performance of a deep excavation 1is related to both stability and deformation. Deep excavations are designed to be stable and to limit deformations to acceptable levels. A stable deep excavation is an excavation whose walls do not collapse, and whose base does not heave uncontrollably. Ground deformations around excavations can damage adjacent buildings, streets, and utilities. The severity and extent of damage depends on the magnitude and pattern of ground movements around the excavation.

Stability and deformation are related with each other. If the factor of safety against collapse is large, strains in the soil around the excavation will be small, and ground movements will be small. On the other hand, if the factor of safety against collapse is close to one, strains in the soil around the excavation will be large, and ground movements can be large.

Thus, prediction of deep excavation performance involves analysis of both stability and deformation. Experience has shown that stability can be evaluated with sufficient accuracy using simple limit equilibrium calculations. Deformations, however, are significantly more difficult to predict, and finite element analyses are often used for this purpose when ground movements are particularly important.

The excavation of soil from a deep excavation has two main effects. The first is that the removal of the weight of the excavated soil results in a decrease in the vertical stress in the soil beneath the excavation. The second is that the removal of the soil in the excavation results in a loss of lateral support for the soil around the excavation. The purpose of a deep excavation support system is to provide lateral support for the soil around an excavation and to limit movement of the surrounding soil.

Support systems for deep excavations consist of two main components. The first is a retaining wall. The second component is the support provided for the retaining wall. Many types of walls and supports have been used in deep excavations. The principal types of walls are diaphragm (structural slurry), sheet pile, soldier piles and lagging, tangent piles, contiguous piles, and deep soil mixed walls. The principal types of supports are struts (braces), rakers, and tieback anchors.

A major problem pertaining to deep excavations is the potential damage to the surrounding buildings and foundations as a result of excessive deflection in the retaining wall and associated ground movement. Therefore, deep excavations in densely built-up areas require stringent control measures to protect against such damage. To minimize the damage to surrounding buildings and facilities, a normal approach often involves usage of a stiff support system including a strong retaining wall together with a stiff bracing system above the final formation level.

In cases where the excavation is underlain by a thick layer of soft soil and supported by the normal bracing system, the maximum wall deflection often occurs at a location below the excavation level. As the maximum wall deflection occurs below the excavation level, even a stiff supporting system above the excavation level may not be effective enough to control the maximum wall deflection.

To prevent excessive soil and wall movement, one effective way is to improve the soft soil layer where the maximum wall deflection is expected to occur. Deep cement mixing method (DCM) and jet grouting method (JG) are two frequently used techniques in stabilizing the soil below the formation level to reduce the maximum wall movement. If the entire layer of soil from one wall to the opposite wall is improved, it behaves like a strut and is referred to as an improved soil raft. On the other hand, if one end of the improved soil contacts with the wall while the other end rests in the soft soil.

The effectiveness of ground improvement techniques in stabilizing excavation has been proven in many successful projects worldwide and verified in numerical studies and experimental studies. Gaba (1990), Newman et al. (1992), Tanaka (1993), Shun (1996) and Ho et al. (1998) reported the effectiveness of such soil stabilization through case studies where excavations were stabilized with fully treated layers. Lee et al. (1991), Yong (1998) and Xie et al. (2000) studied the behaviour of excavations stabilized by soil improvement techniques using numerical simulation. It was found that the ground and wall deformations for stabilized excavations were less than those without any improvement. Liao (1993) conducted a series of 1g laboratory experiments and found that for partially improved excavation, the mobilized shear resistance and end bearing are the two main contributory factors of the improvement effect.

An embedded retaining wall is one that penetrates the ground at its base and obtains some lateral support from it. The wall may also be supported by structural members such as props, berms, ground anchors and slabs.

The need for design and construction of embedded retaining walls increased in the last decades as the need for underground structures, such as deep basements and subway systems, increased. Any embedded wall project must be designed to provide suitable protection against ultimate limit states and serviceability limit states. Ultimate limit states are those associated with collapse or with other similar forms of structural failure. They are concerned with the safety of people and the safety of the structure. Serviceability limit states correspond to conditions beyond which specific service performance requirements are no longer met, for example predefined limits on the amount of water seepage, wall deflections.

The design of the embedded walls include the determination of penetration depth of the wall (wall toe level), structural forces and the effects on adjacent structures or facilities if any. The wall toe level (or penetration depth of the wall) of any embedded retaining wall should be the deeper of that required to satisfy load bearing capacity, hydraulic cut-off and uplift, global stability and lateral stability.

The wall toe level for overall lateral stability can be determined by limit equilibrium method, subgrade reaction method, pseudo finite element method and finite element&finite difference methods. These various design methods have different capabilities and yield different results which confuse the designers.

1.2. Objective of the Thesis Study

Main objectives of the thesis study are given followings below;

- The first aim of this study is to make an analysis for a deep excavation by using Finite Element Analysis Method.
- In order to make an analysis two software programs (DeepXcav and Plaxis 2D) were used.
- To observe the comparison results between the real outputs and the software analysis results.
- To show the reliabilty of the software analysis methods for doing deep excavation work.
- To define the best analysis techniques for FEM analysis in order to reach correct solution.

1.3. Organization of the Thesis Study

The organization of the thesis study as following;

Chapter 1: The general overview of the study was presented in this chapter. Several deep excavation methods were given in this chapter. Furthermore, general information related with the software programs that were used for making analysis were presented in this chapter.

Chapter 2: The literature review begins with discussing the conventional support system in controlling ground movements and its limitations. Subsequently, attention was paid to evaluate the effectiveness of having an embedded improved soil layer to control ground movements during excavation works in soft ground.

The central idea is to evaluate the fundamental behaviour of an excavation stabilized with an improved soil mass.

Chapter 3: The comparison results that were obtained from Plaxis 2D and DeepXcav were observed and presented.

Chapter 4: The test results obtained from the study were given and the discussions were done with respect to the results in this chapter.

Chapter 5: The conclusions extracted from the thesis study were presented clearly and recommendations related to next study were submitted in this chapter.

CHAPTER 2

LITERATURE REVIEW

2.1. Introduction

The purpose of a deep excavation support system is to provide lateral support for the soil around an excavation to limit the wall deflections and ground movements. If the wall deflections and ground movements are excessive during the excavation process, severe damages to surrounding buildings, roads and infrastructures may occur and in the worst case, collapse of the excavation system itself. In cases where the excavation is underlain by a thick layer of soft soil, the maximum wall deflection often occurs below the excavation level. In order to control the maximum Wall deflection, one commonly used technique is to improve the soft clay below the excavation level using ground improvement techniques like jet grouting (JGP) and deep cement mixing (DCM) to help to reduce the lateral movement of the retaining wall and ground settlement.

In this chapter, the literature review begins with discussing the conventional support system in controlling ground movements and its limitations. Subsequently, attention was paid to evaluate the effectiveness of having an embedded improved soil layer to control ground movements during excavation works in soft ground. The central idea is to evaluate the fundamental behaviour of an excavation stabilized with an improved soil mass.

2.2. Conventional excavation support system and its limitations

In an excavation, there are two main effects from the stress point of view. Then first is that removal of soil causes a reduction in the total vertical stress in the soil beneath the excavation. The second is that the removal of the soil results in the removal of lateral earth pressure on the excavated side, thereby causing a stress imbalance. Thus, the entire system including the soil will move to ensure other forces are mobilized to balance the stress relief in both directions during an excavation. The relief of the vertical stress causes basal heave and the removal of lateral stress leads to the movement of the retaining wall and soil behind wall towards the cut. The basal heave and the inward movement of retained soils are often accompanied by subsidence of the ground near the excavation. If the ground movement is excessive during the excavation process, severe damage may occur to surrounding buildings, roads and infrastructures. The purpose of a deep excavation support system is to provide lateral support for the soil around an excavation to increase the stability of the excavation and consequently to limit movement of the surrounding soil. Stability and ground movements of an excavation are related. If the factor of safety against failure is large, ground movements will be small. On the other hand, if the factor of safety is close to unity, ground movements can be large.

In cases where the excavation is underlain by a thick layer of soft soil, the maximum wall deflection, which is significantly influenced by the properties of soil beneath the excavation level, often occurs below the excavation level. This has been observed in numerous field cases and also predicted in numerical analyses (Lee et al., 1991; Wong and Patron, 1993; Kusakabe, 1996; Chew et al., 1997). This is often the case in Singapore where many of the deepest excavations are in the downtown area, near to the Singapore River, Geylang River and Kallang River where there are thick deposits of soft clay. Unfortunately, using stiffer and stronger bracing struts and increasing the number of layers of struts may not be effective enough to control the maximum wall deflection (Lee et al., 1991; Wong et al., 1998) because the wall is not propped at the most critical level, which is below the final excavation level.

2.3. Soil stabilization in deep excavation

To control the wall deflection in such situations, the soft clay below the excavation level can be improved by ground improvement techniques like jet grouting or deep cement mixing to increase the stiffness of soil below the excavation level, and consequently to reduce the lateral movement of the retaining wall, base heave and ground settlement.

The provision of an embedded improved soil layer in an excavation is essentially an extension of the idea of bracing. The word ‘embedded’ is to underline the fact that this is below the excavation level. As conventional strut cannot be placed below the final excavation level where the maximum wall movement would occur, the alternative is to improve the soft soil at this critical location through using an insitu soil improvement technique such as jet grouting or deep mixing. An added and distinct advantage over conventional support systems is that this soil improvement is normally carried out prior to the excavation and the improved soil mass exerts its effectiveness right from the start of excavation process.

The effectiveness of these ground improvement techniques in controlling ground movements and lateral movement of the retaining wall has been proven by many successful engineering cases (Tanaka, 1993; Yong and Lee, 1995; Byuan et al., 2001; Hu et al., 2003). Jet grouting was used at Dhoby Ghaut MRT Station (Tornaghi et al., 1985), Newton Station (Gaba, 1990) and Clarke Quay Station (Shirlaw et al., 2000). The lime-column soil improvement technique was used at the Bugis and Lavender stations (Hulme et al., 1989) and more recently this method was also used in the construction of the proposed HDB Centre at Toa Payoh (Tan et al., 2001). Two layers of improved soil raft below the excavation level were constructed to control the wall deflection at the Bugis Junction car park basement (Shun et al., 1996). However, though its use is becoming more extensive, the behaviour and mechanisms involved are still not well understood and the present design concept is highly simplified and empirical in nature.

2.4. Review on soil stabilization in deep excavations

Some research work has been reported concerning excavations stabilized with embedded improved soil layers. This review on soil stabilization in deep excavations is divided into three categories, namely field studies, numerical studies and experimental studies.

2.4.1. Field studies

Gaba (1990) reported the use of a 3.5-m thick jet grouted raft immediately below the formation level at Newton station in Singapore marine clay. The author presented measured field results and concluded that the jet grout raft was successful in reducing the retaining wall deflections as compared to the hypothetical situation without it.

Newman et al. (1992) reported the use of a 1.5-m thick jet grouted raft below the formation level of a braced excavation which was designed both to act as a base prop for the retaining wall to restrain the movement and to resist uplift pressure due to the sub-artesian water pressure below the raft. The scheme was successful in limiting the lateral wall deformation to an acceptable value as compared to predictions from FEM analyses for the excavation if no base stabilization was carried out, which showed large inward movement of the wall below formation level, as shown in Figure 2.1.

Liao et al. (1992) reported a case study involving the improvement of soil both inside and outside a 12-m deep excavation, where the surrounding structures would be sensitive to any excessive ground movements. The plan layout of the excavation is shown in Figure 2.2 and the soil stabilization works consisted of three schemes as shown in Figure 2.3. The measured lateral wall deflection profiles are shown in Figure 2.4. It was found that the buttress type grouted panels installed in front of the wall before the excavation were effective in reducing the wall deflection induced by excavation. Though the improved area did not cover the whole excavation base, the effect of such configuration of the improved soil was equivalent to that of a 'strut' because the buttress panel had direct contact with either retaining walls or foundation piles such that both sides of the improved soil mass were constrained.

Tanaka (1993) analyzed the data from field measurements to study the behaviour of a 15 to 21-m deep braced excavation in soft clay stabilized by a combination of deep cement mixing (DCM) and jet grouting. The soil stabilization scheme was in the form of a layer of overlapping DCM columns spanning across the excavation. Untreated soil between this stabilized ground and the retaining wall was then improved by jet grouting, as shown in Figure 2.5. The measured lateral Wall deflection profiles are shown in Figure 2.6. The author reported that the ground stabilized by DCM of multiple soil columns offered a high resistance against lateral forces, but a low resistance against vertical forces. The soil treated by DCM can be considered as a typical brittle material. It was also observed that large basal heave occurred even with soil stabilization below the formation level due to the thick soft clay deposits below the excavation and the great excavation depth, as shown in Figure 2.7. The author also compared the distribution of earth pressure between the treated and non-treated excavations from measured results, as shown in Figure 2.8. It was found that the treated ground beneath the excavated bottom took a considerable share of the earth pressure from the active side and consequently the remaining component sustained by the struts was significantly reduced. The proposed displacement pattern for the base treated soil is shown in Figure 2.9, and a new stability number, N_t was proposed for the base heave failure for excavations with treated soil at the base that can be used to determine the thickness of the treated soil layer.

Liang et al. (1993) reported a canal construction using jet grouting as the retaining system instead of using the conventional method of sheet piling with struts. The purpose of using jet grouting was to control the upheaval of the soft clay when the canal was excavated to a greater depth. The typical geometry of the proposed jet grouted mass is shown in Figure 2.10. This system basically consists of an inverted arch with two long jet grouted piles at each end of the arch. The end piles act as the abutment for the inverted arch.

The rationale behind the use of the arch geometry was that the arch would be able to resist most of the horizontal stresses resulting from the active pressure of the soil when excavation was carried out. Above the arch was a layer of grouted soil with a lower strength than the jet grouted arch. No significant base heave was observed and the measured wall deflection is typical of the cantilever type shown in Figure 2.11.

Shun et al. (1996) reported a project adjacent to Bugis MRT Station which adopted a double layer jet-grouted raft to reduce the wall deflection during excavation, where soft marine clay extended to depths varying from 27 m to 40 m below the ground level. The lateral wall deflection was predicted by two methods, namely the elasto- plastic spring model and FEM analysis. The results showed that the wall deflection would cause displacement of the adjacent tunnels larger than 15 mm without soil improvement. On the other hand, the double layer jet-grouted rafts could limit the movements of the adjacent tunnels to an acceptable value.

Khoo et al. (1997) reported the use of a soil berm improved by jet grouted piles in the UE Square Project to reduce the lateral deflection of the retaining wall. Owing to the large excavation area of about 150 m by 200 m, diaphragm walls were designed to be retained by soil berms and raking struts. As the soil berm consisted of thick soft organic clay and marine clay, it was expected to be ineffective without improvement. Thus, the soil berm was treated by rows of jet grouted piles for the entire organic clay and marine clay layers and keyed into 1 m of very stiff residual soil of the Jurong Formation. The excavation sequence and soil berm details are presented in Figure 2.12. The deflection of the diaphragm wall and surrounding ground movements were monitored during excavation and also predicted by an elastoplastic spring analysis and FEM analysis. The predicted wall deflection and ground movement agreed reasonably well with the measured values. Based on elasto-plastic analysis, a comparison of lateral deformation of the diaphragm wall with and without improvement in soil berm is shown in Figure 2.13. This analysis confirmed that the treated soil berm limited the wall deflection.

Ho et al. (1998) reported a deep basement excavation of the Singapore Post Centre project in thick soft marine clay which was located in close proximity of the Paya Lebar MRT station and viaduct. A jet grout raft of 3-m to 4-m thick was installed between the diaphragm walls beneath the formation level to reduce the deflection of the diaphragm walls during excavation. Furthermore, in order to restrict wall deflections at the early stages of excavation, the jet grout was fanned out to a zone of 10-m long and 9-m thick in front of the diaphragm walls facing the MRT structures. A trial section of the jet grout scheme is shown in Figure 2.14. From the field measurements, the authors concluded that the fan shaped jet grout strut was effective in restricting the wall deflections especially at the early stages of excavation.

O'Rourke et al. (1998) reported an excavation stabilized by deep mixing method (DMM) and jet grouting in deep marine clay with excavation depth from 13.9 m to 19.4 m. The excavation was supported by a soil mixing wall (SMW) with earth anchored tiebacks. Owing to different excavation depths at the east and west sides, two different configurations of soil treatment were introduced. The typical cross sections of DMM and jet grout improvement near the East and West walls are shown in Figure 2.15. The soil treatment along the East wall penetrated into the underlying firm layer, whereas the soil treatment along the West wall just floated within the clay layer. Two different measured lateral deformation profiles were observed. For the DMM zone that had penetrated into a firm layer, the treated mass acted as a shear beam with lateral deformation distributed along its depth. However, for the floating DMM mass, heave and upward deformations below the treated soil tried to lift the DMM mass. Typical observed incremental lateral deformation profiles and the proposed deformation patterns for the treated mass are shown in Figure 2.16.

From the above review of case studies, both the embedded improved soil raft and embedded improved soil berm were used in practice and the effectiveness of the soil improvement technique in reducing wall deflection has been demonstrated. However, most of the above cases only reported the wall deflection. There was no reported ground movements on the excavated side and therefore short of information on the mechanisms involved for the embedded improved soil to mobilize its resistance to reduce the wall deflection.

2.4.2. Numerical studies

Lee and Yong (1991) reported two projects using jet grouting as the soil stabilization below the formation level to minimize the ground movements. In both projects, the authors analyzed the ground and retaining wall movements by FEM method. In Project A as shown in Figure 2.17, a 2-m thick layer of soil below the formation level was grouted to act like a 'strut' in the marine clay and to transfer the forces to the sides of the retaining wall. In Project B as shown in Figure 2.18, double layer jet- grouted rafts were installed to reduce the wall deflection. It was found that the ground and wall movements were excessive without soil stabilization and increasing the stiffness of lateral supports to reduce ground movement was not as cost-effective as improving the soft clay just below the formation level at the elevation of maximum wall deflection.

Ou et al. (1996) described three typical patterns of treated soil mass, namely block type, column type and wall type as shown in Figure 2.19. For the wall type of soil treatment, the lateral force caused by the inward movement of the retaining Wall acts directly on the counterfort wall, in which the side friction and end bearing provide the resistance. For the case of column type, the lateral force acts on the untreated soil, which in turn transmits the force to the treated soil. The block type of soil treatment has the advantages of both the wall type and column type. The authors reported the study of grouted column type of soil improvement for deep excavations to reduce ground movements. They employed 3-D and 2-D plane strain finite element analyses to back analyse the observations from a case study. The primary objective of the study was to propose a method for evaluating the overall material properties of the treated soil mass whereby the treated area of soil could be replaced by a single material during the 3-D FEM analysis. This method could be used in 2-D plane strain analysis after slight modification to reduce computational resources. However, this study concentrated on the composite properties of the treated soil mass to simplify the analysis.

Yong et al. (1998) reported the 2-D and 3-D numerical analysis of a hypothetical excavation supported by sheet pile wall with a 3-m thick treated soil block or raft below the final excavation level. In order to study the influence of the thickness of the grouted layer, excavations with 1.5-m and 3-m thick grouted layers were simulated by numerical method. It was observed that a 3 m grouted layer was needed to control the deflection of the relatively flexible sheet pile wall. Compared to the cases without any treatment, there was a significant reduction of the maximum wall deflection of about 45% and 38% for the 2-D and 3-D analyses respectively, as shown in Figure 2.20. The results showed the effectiveness of an improved raft to reduce the wall deflection. However, there was no report on the behaviour of an embedded improved berm in an excavation.

Wong et al. (1998) studied the optimization of jet grout configuration for a braced excavation in soft clay by 2-D FEM analysis. This paper presented the results of a series of parametric studies to examine the influence of the jet grouting raft on the behaviour of a braced excavation in soft clay. It was shown that provision of an embedded jet grout raft could reduce wall deflection, ground movements, strut forces and wall bending moment. The effectiveness increased with increasing grout thickness or increasing the number of grout layers. This study provided an overall perspective of the improvement with different configurations. However, this study concentrated on embedded improved soil raft instead of embedded improved soil berm.

Lim (1999) conducted a parametric study using 3-D FEM analysis of excavations in thick soft clay stabilized by different configurations of improved soil. The purpose of this study was focused on the adequacy of lateral and vertical resistance against basal heave provided by the treated soil mass. The author first studied a baseline model with double layers of treated rafts. It was found that this type of stabilization was effective in reducing the wall deflection compared to the case without the treated layers as shown in Figure 2.21. The author also studied three other configurations of the treated soil mass, namely ‘Single Layer’ scheme, ‘Wall Grid’ Scheme I and ‘Wall Grid’ Scheme II.

The calculated wall deflections of these schemes are also presented in Figure 2.21. It was observed that the deflection profiles of these three models were quite similar and the deflections were smaller than that of the baseline mode, which meant that the thicker single layer of treated raft was more effective to reduce the lateral wall deflection and provide sufficient resistance against global base heave when the retaining wall was relatively short and terminated in the soft clay. However, this study again concentrated on the behaviour of excavations stabilized with an embedded improved soil raft.

Xie et al. (2000) reported the behaviour of cantilever and single braced excavations with various widths and depths of improved soil on the passive side using FEM analysis. The schematic layout and the finite element mesh of the cantilever excavation analysis are shown in Figure 2.22. The results showed that enlarging the treated width was more effective than increasing the treated depth in reducing the Wall deflection, ground settlement, base heave and strut forces. The authors suggested that the treated depth should not exceed 50% of the excavation depth for the cantilever excavation and 60% for the single propped excavation. However, the modulus of the improved soil for analysis was selected as 40 MPa, which is relatively low for the treated soil.

2.4.3. Experimental studies

Liao et al. (1993) studied the passive resistance of partially improved soft soil in a 1g laboratory test program. The schematic diagram of the test setup is shown in Figure
2.23. The study involved two types of soil improvement patterns, namely the column type and the buttress type, as shown in Figure 2.24. In their study, a horizontal wall was used to load the reinforced soil specimens to failure. The passive resisting force of the improved soil, the surface heave and the deformation of this Wall were monitored throughout the test.

The authors presented the test results of the column type with different improvement ranges. It was found that the existence of the improved columns would influence the displacement patterns on the passive side. The surface heave for the column type of soil improvement started at the rear portion of the reinforced soil block and reached a peak value at a distance further away from that of the specimen without improvement. However, the surface heave started almost from the contact surface between the retaining wall and the soil for those without improvement. The ultimate passive resistance of treated soil increases with the increase of the improvement range as shown in Figure 2.25. The ultimate passive resistance of the treated soil increased almost linearly with the increase of the contact area between the grouted columns and the soil, as shown in Figure 2.26.

The authors also studied the buttress type of treated soil. For the buttress type, three different shapes were examined, namely buttress panel shape, L shape and box shape. It was found that the length of the buttress panels should be long enough to escape the plastic zone generated by the retaining wall in order to mobilize the end bearing and side friction of the panels. The L shape and box shape were adopted to increase the end bearing effect. The test results showed that the L shape pattern is more effective than the box shape pattern because the gap between the L shape panels helped to mobilize the side friction and end bearing of the panel.

This study provided a glimpse of some of the factors that could affect the effectiveness of the improved soil such as end bearing, side friction, area of contact, length, configuration, stiffness of the improvement etc. It also revealed that there are complex interactions between the pattern and the length of the improved soil panel. However, it should be noted there are some limitations in the study. Firstly, except for the wall movement and surface heave, there was no other detailed information on the sub-ground movement which is important to understand the mechanisms involved to mobilize the various resistances. Secondly, the tests conducted did not simulate the unloading effect during excavation which may affect the results obtained.

Ohnishi et al. (2000) presented the results of three centrifuge model tests on deep mixing method using fly ash and cement. The study investigated the stability of a 16 m deep braced excavation improved by the deep mixing method with steel sheet pile walls in soft ground. The schematic layout of the centrifuge model tests is shown in Figure 2.27. The three models had different base conditions. Case-1 was an unimproved case with q_u =60 kPa at the upper part of the ground. Case-2 was an improved case with q_u =400 kPa below the formation level. Case-3 was also an improved case with unconfined compressive strength q_u =100 kPa. From the Case-2 test results, the authors showed that the resultant force of strut loads at the end of excavation was only about 35% of that in Case-1 and Case-3. This meant that a considerable part of the load is supported by the improved soil. It was also found that a small basal heave of about 5 cm in prototype scale was observed in Case-2 due to excavation. However, in the other two cases, over 20 cm of heave was observed. It was then concluded that the deep braced excavation stabilized with q_u =400 kPa was sufficient to limit the basal heave to a desired magnitude.

Goh (2004) studied the behaviour of an excavation stabilized with an improved soil raft using both centrifuge tests and numerical analysis. The centrifuge experimental set up for a typical test is shown in Figure 2.28. His study showed that the improved soil raft behaved like a strut below the excavation level and the effectiveness of it is very much dependent on its stiffness. The results also revealed that a stiffer improved soil raft provided a higher resistance to the retaining wall, but also induced a much higher bending moment in the wall. Goh also studied the effect of a gap of untreated soil in between the improved soil raft and the retaining wall as shown in Figure 2.29. It was shown that the performance was governed by the width of this untreated gap and the overburden above the gap.

Thanadol (2003) conducted a number of complicated centrifuge experiments and numerical analyses to study how the length and the stiffness of an improved soil berm would influence the wall and ground performance. The centrifuge experimental set up for a typical test is shown in Figure 2.30. His study showed that an embedded improved soil berm behaved like a friction pile. Both the end bearing and interfacial shear resistance played an important role in controlling the ground movement and wall deflection. He also showed that increasing the length of the improved soil berm was an effective way of reducing the wall movement since more interfacial shear resistance could be mobilized as shown in Figure 2.31. The length of the berm should also be longer than the global passive zone for it to be effective, whereas the effectiveness of increasing the stiffness of the improved soil berm is marginal once the stiffness is greater than a threshold value shown in Figure 2.32.

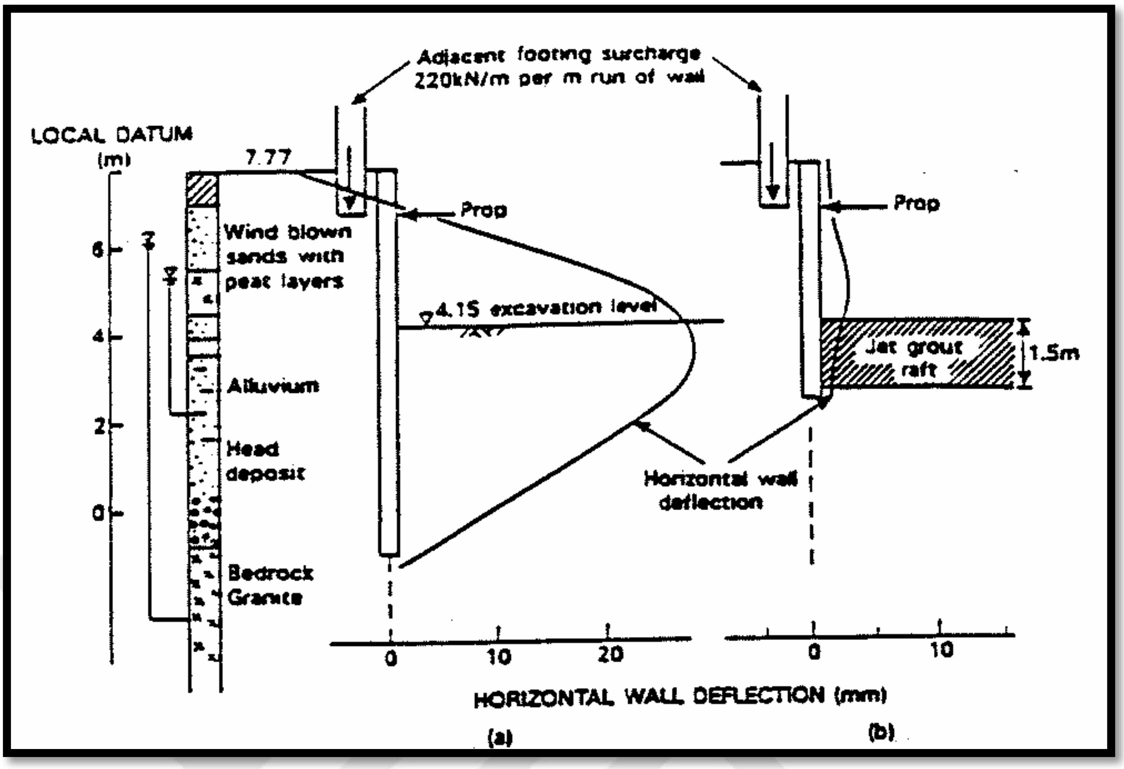

Figure 2.1 Maximum wall deflection (a) without a jet grout raft; (b) with a jet grout raft (after Newman et al., 1992)

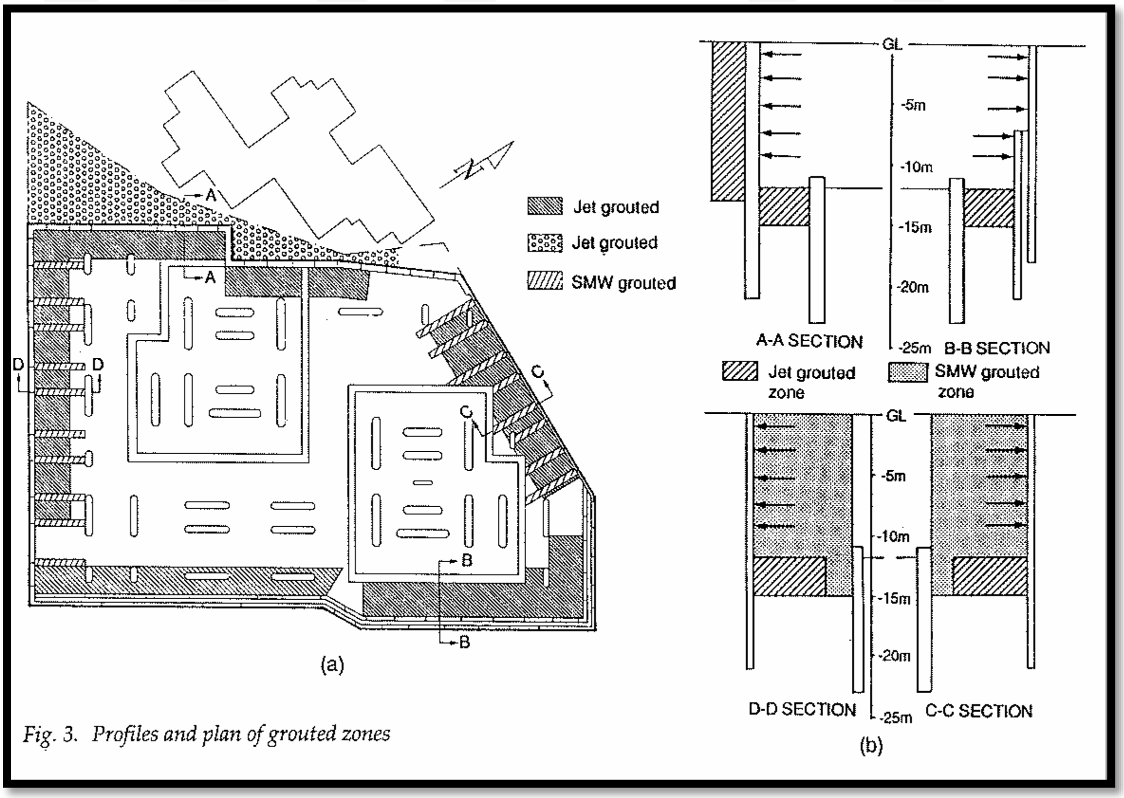

Figure 2.2 Plan layout of the improvement scheme (after Liao et al.,1992)

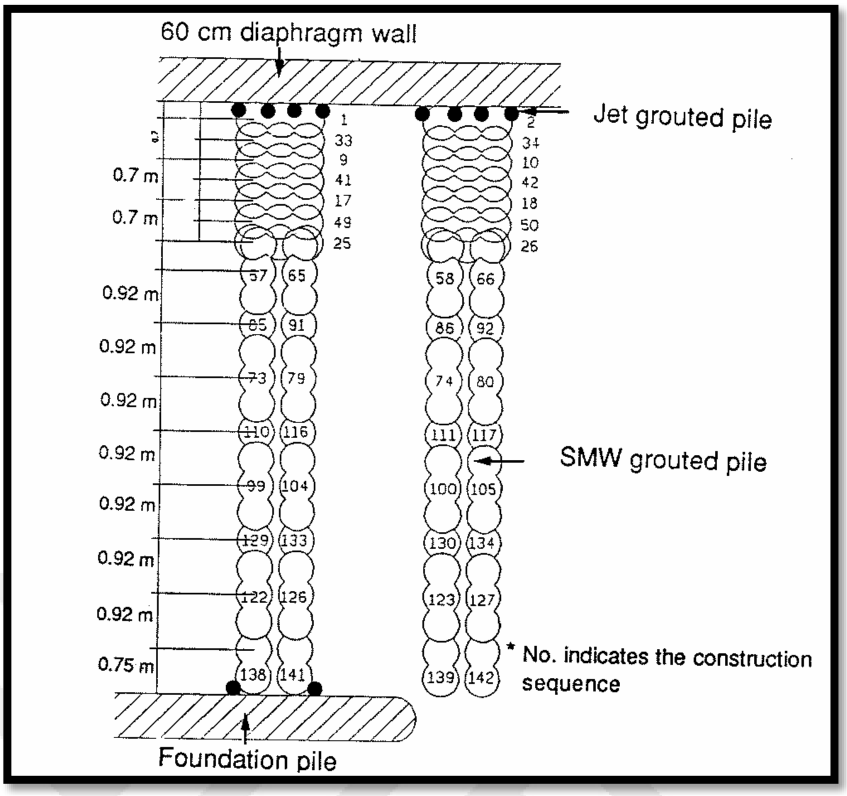

Figure 2.3 Plan view of base stabilization schemes (after Liao et al.,1992)

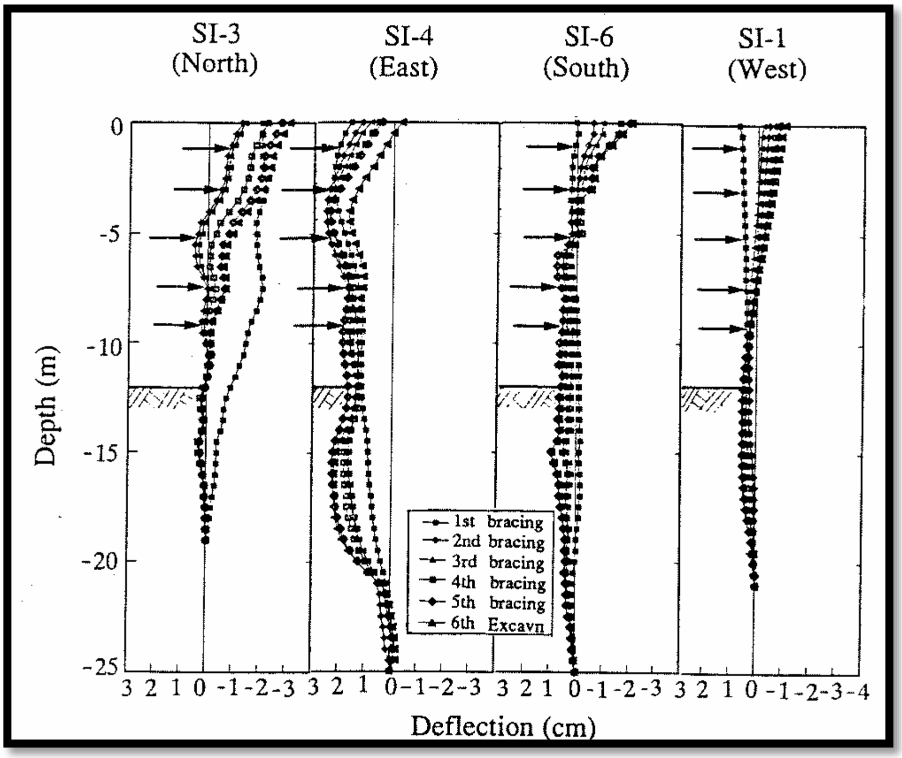

Figure 2.4 Lateral deflection profiles of retaining wall (after Liao et al., 1992)

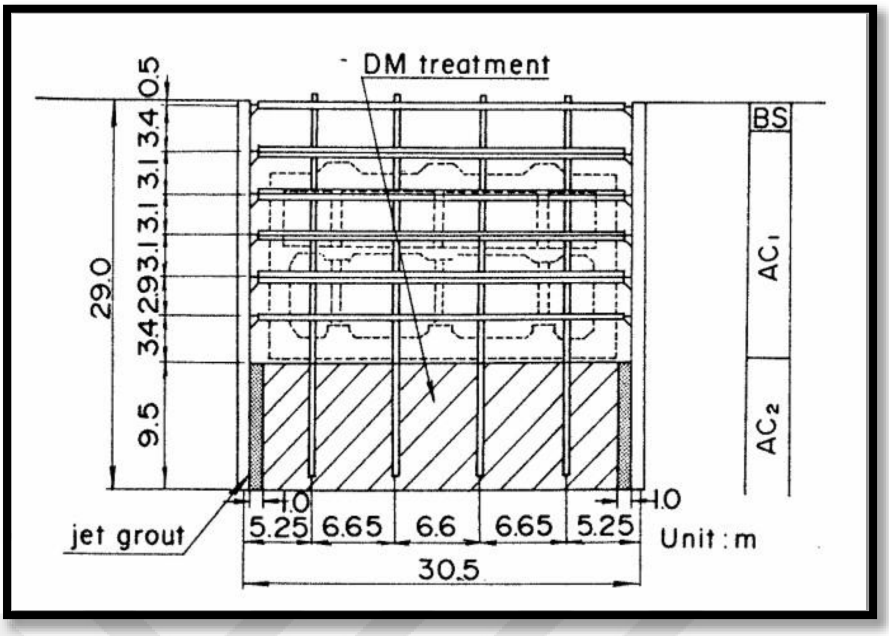

Figure 2.5 Section of braced excavation (after Tanaka, 1993)

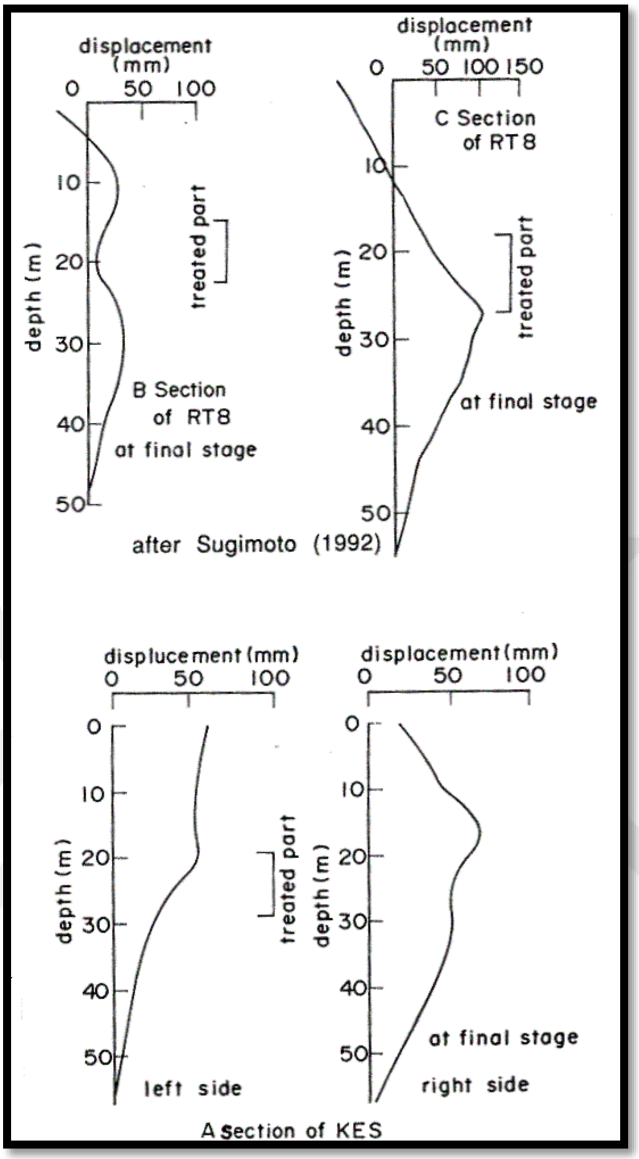

Figure 2.6 Lateral deflection profiles of retaining wall (after Tanaka, 1993)

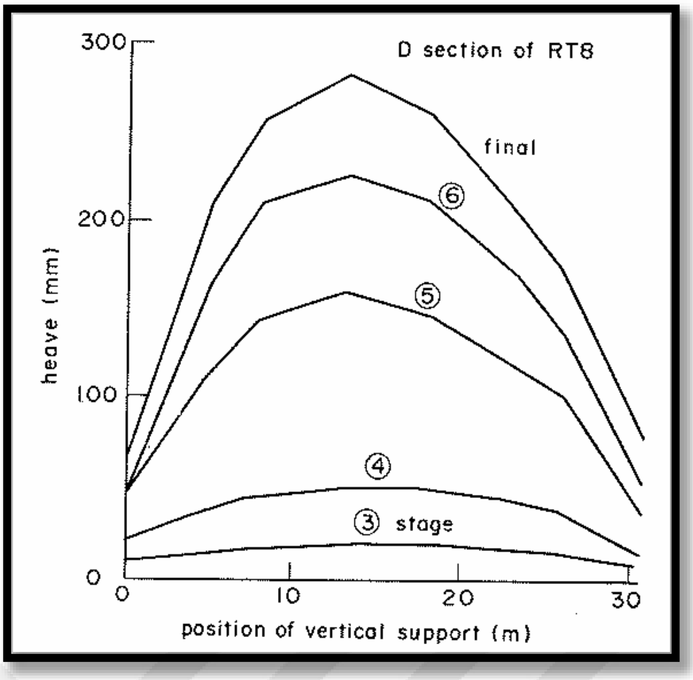

Figure 2.7 Large heave of vertical supports due to base heave (after Tanaka, 1993)

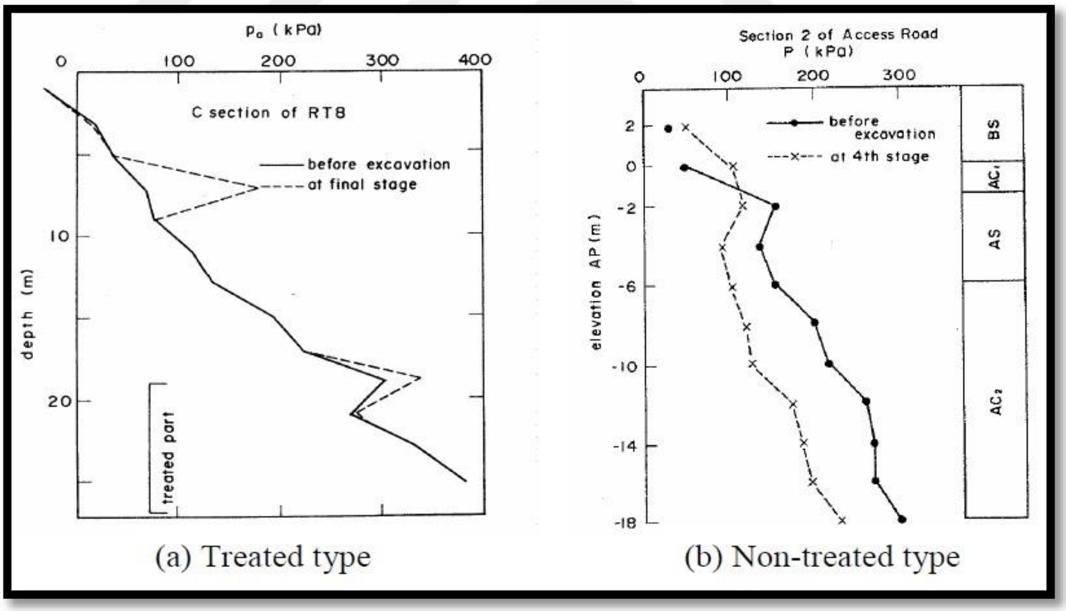

Figure 2.8 Distribution of measured earth pressure on the back side (after Tanaka, 1993)

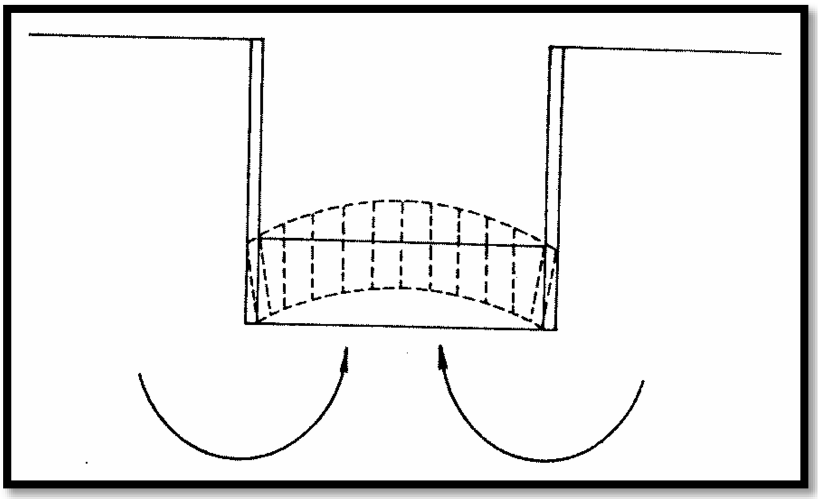

Figure 2.9 Proposed displacement pattern for fully treated soil layer (after Tanaka, 1993)

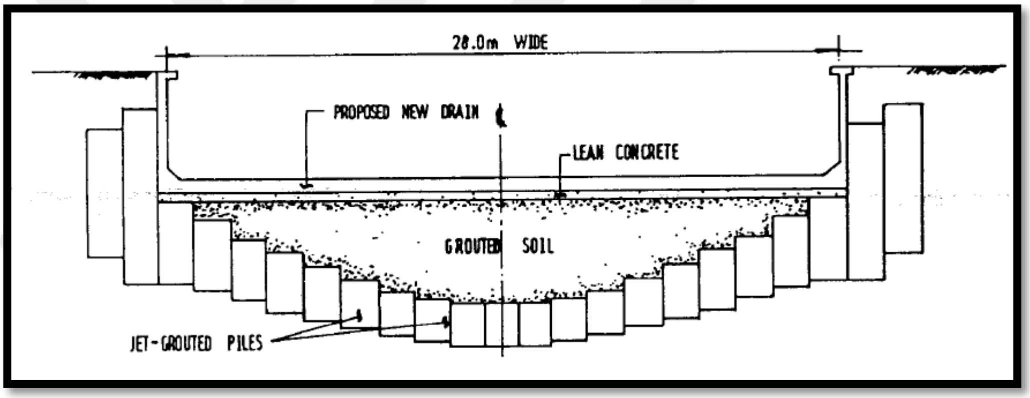

Figure 2.10 Proposed grouted soil mass (after Liang et al., 1993)

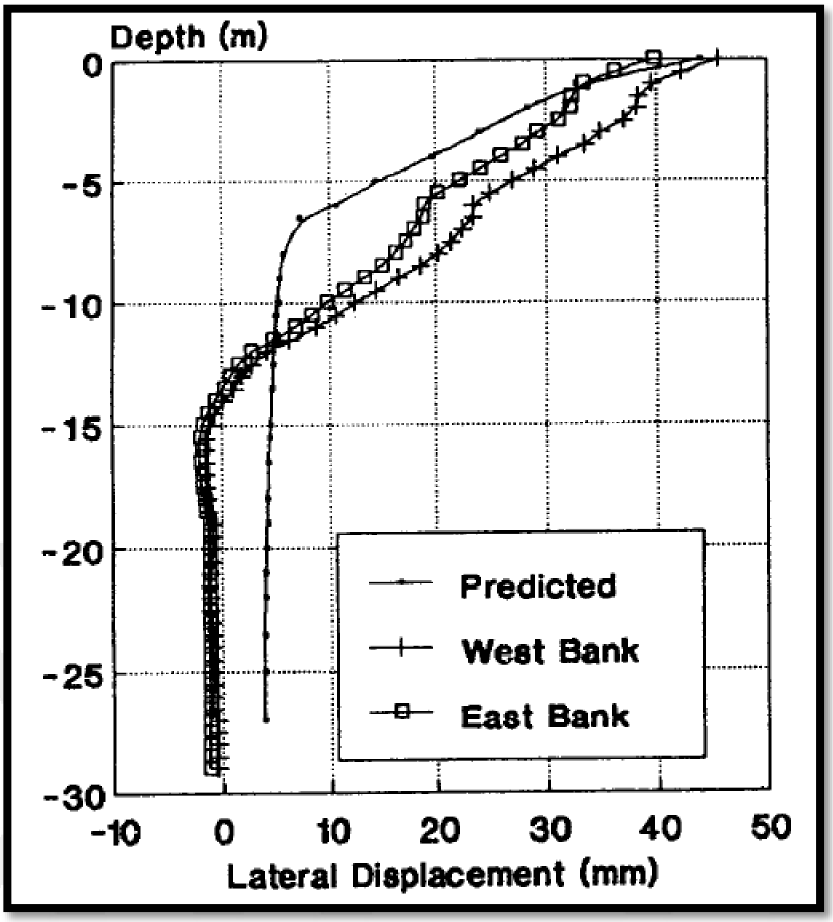

Figure 2.11 Comparison between observed and predicted wall deflection (after Liang et al., 1993)

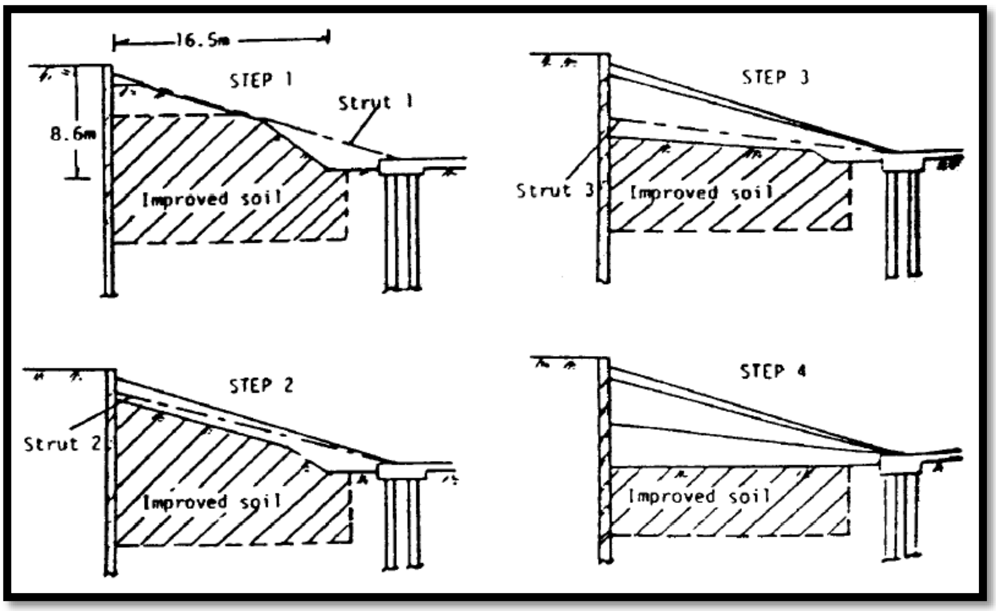

Figure 2.12 Construction sequence and soil improvement (after Khoo et al., 1997)

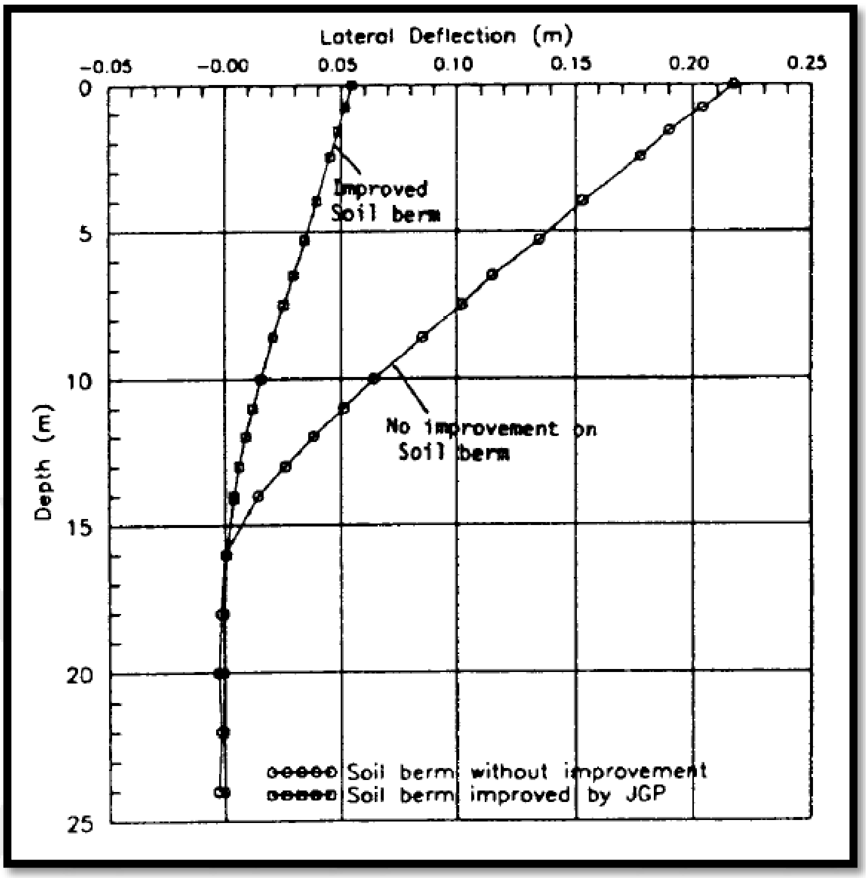

Figure 2.13 Comparison of lateral deformation of diaphragm wall with and without improvement in soil berm (after Khoo et al., 1997)

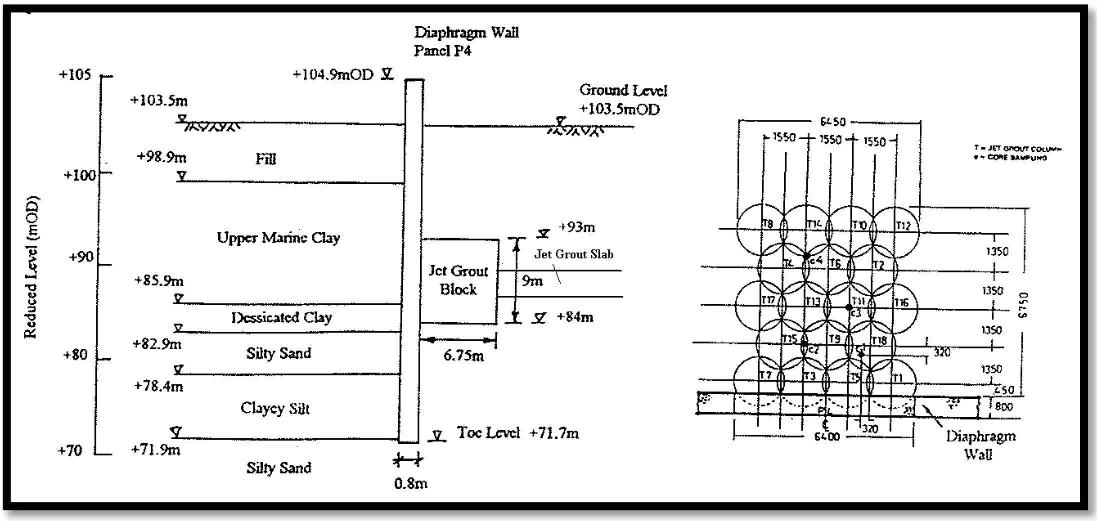

Figure 2.14 Typical-cross sections of the jet grout scheme (after Ho et al., 1998)

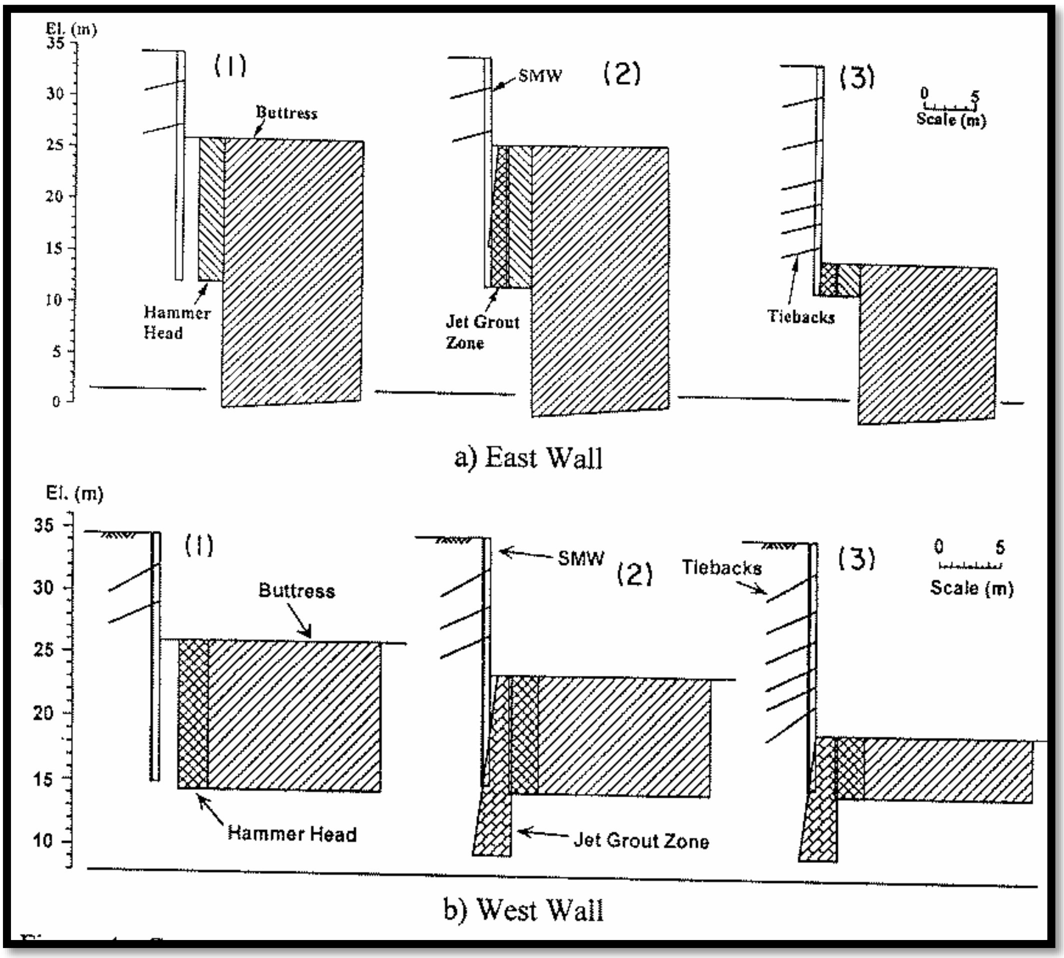

Figure 2.15 Typical cross sections of DMM and jet grout improvement near the east and west walls (after O'Rourke et al., 1998)

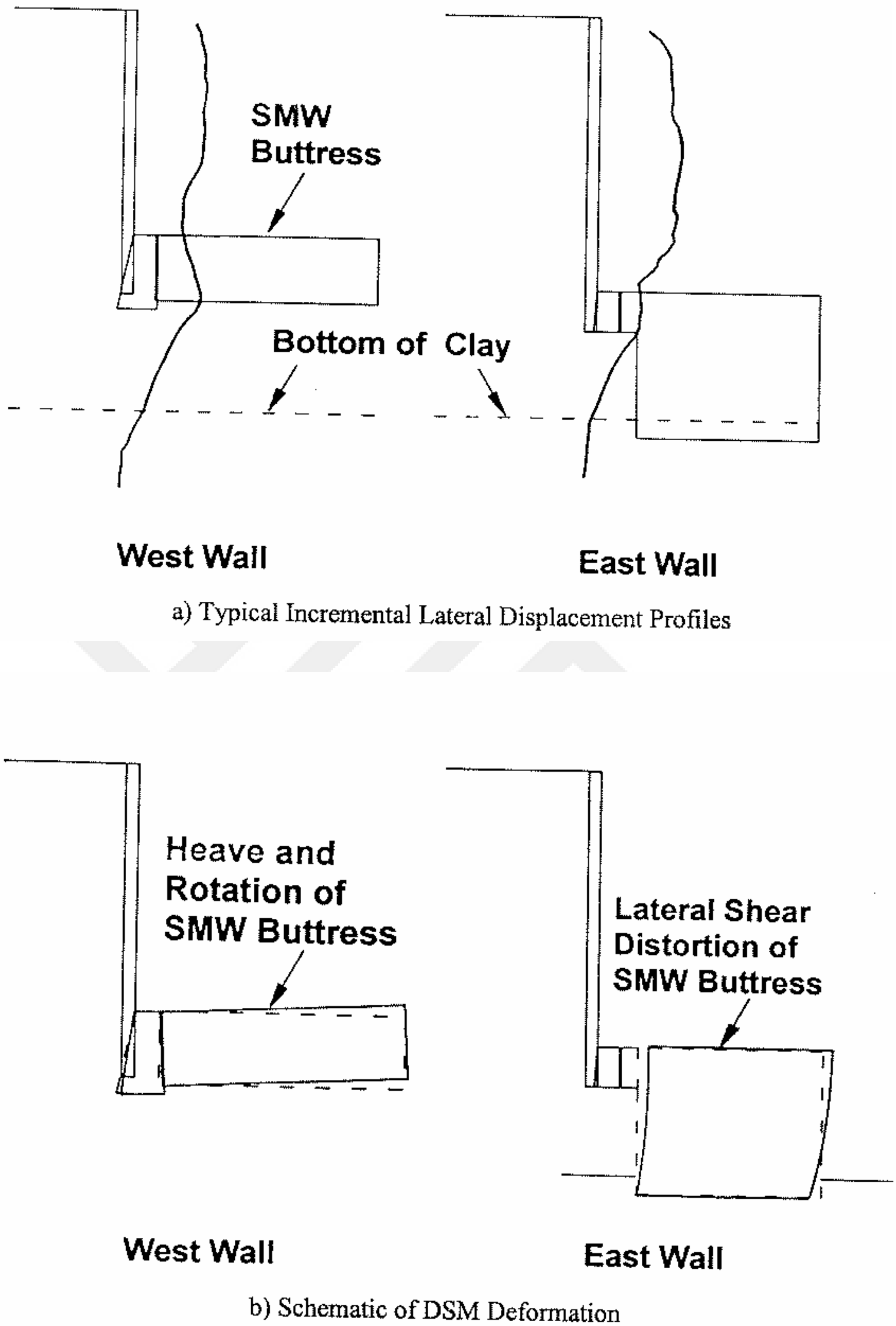

Figure 2.16 Typical observed incremental lateral deformation profiles and the proposed deformation patterns for the treated mass (after O'Rourke et al., 1998)

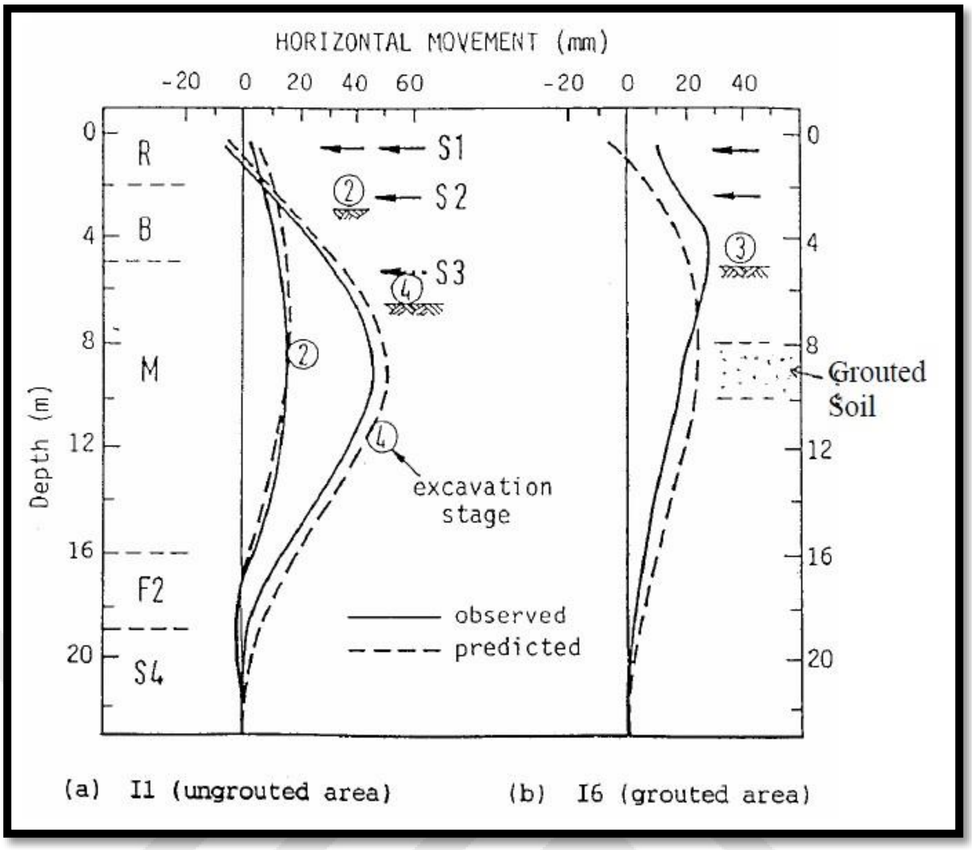

Figure 2.17 Comparison of lateral wall movement in project A (after Lee and Yong, 1991)

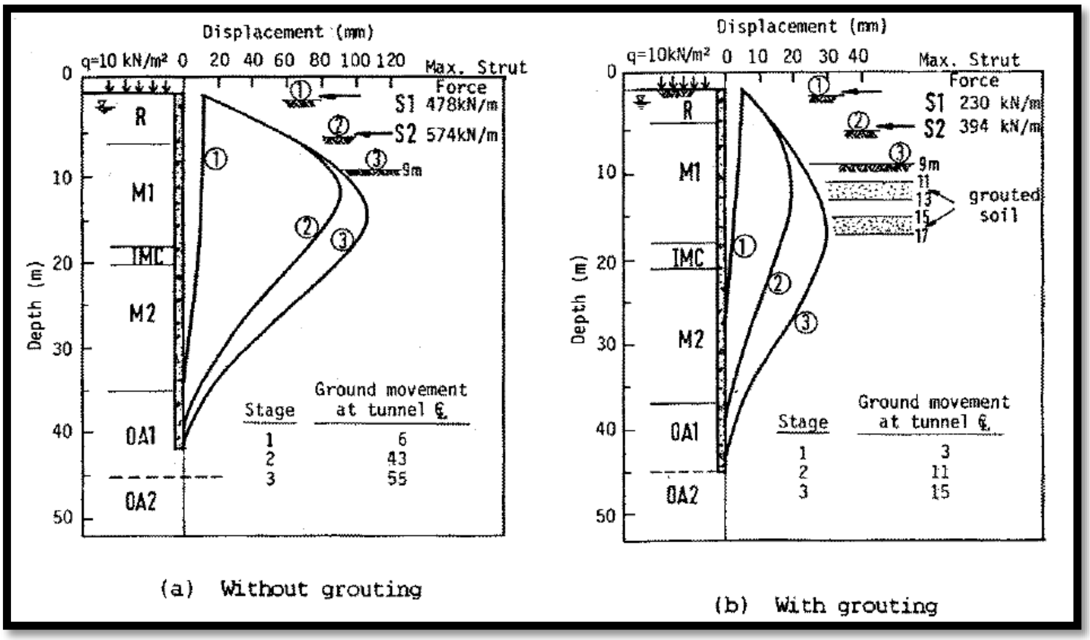

Figure 2.18 Predicted wall movement for project B (after Lee and Yong, 1991)

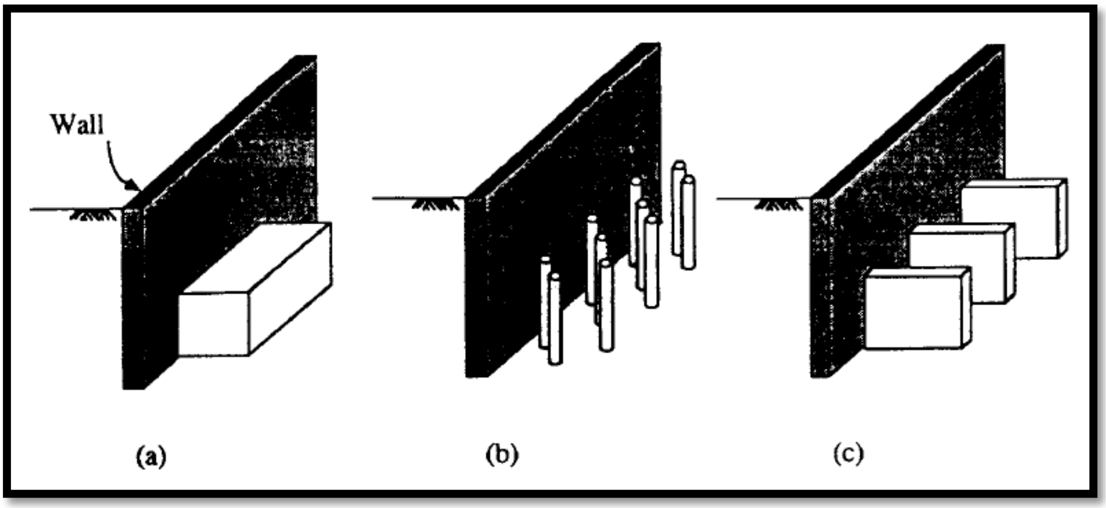

Figure 2.19 Typical patterns of treated soil mass in excavation (a) block type; (b) column type; (c) wall type (after Ou et al., 1996)

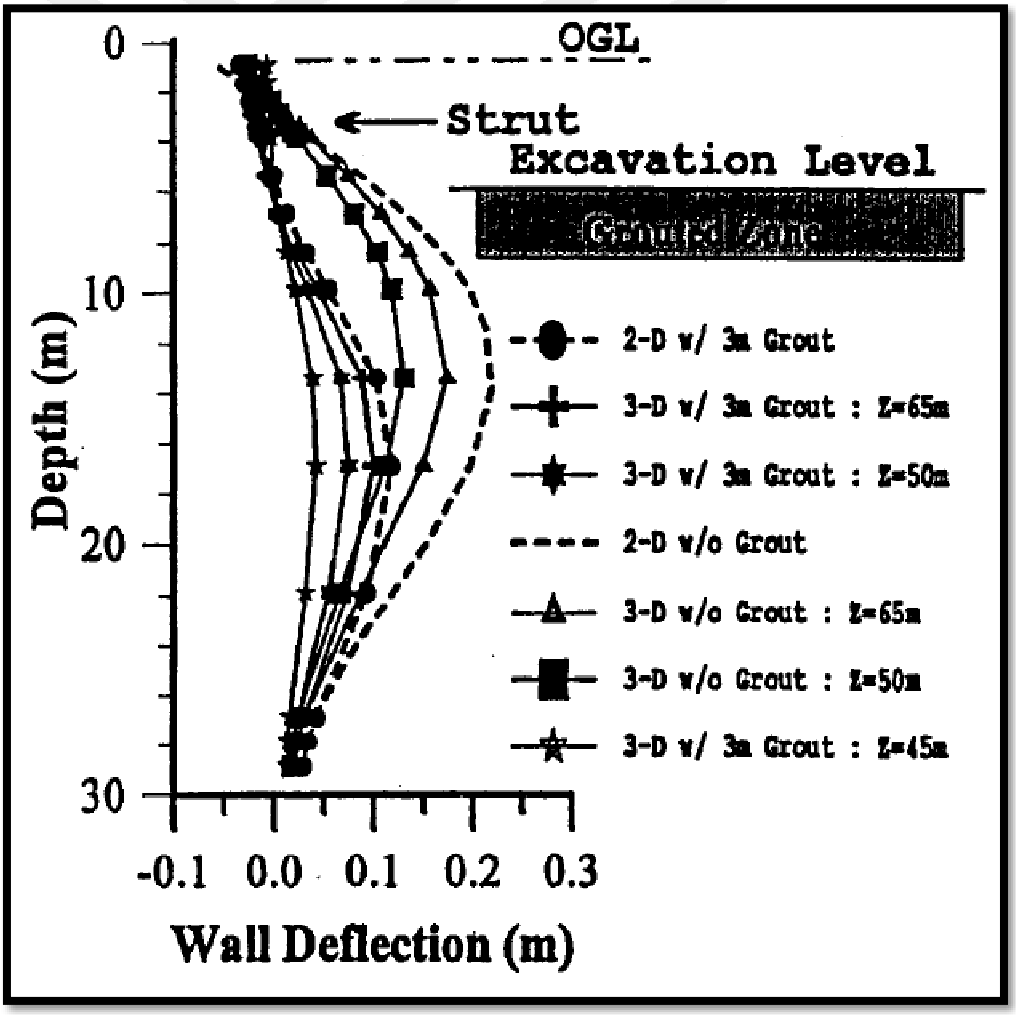

Figure 2.20 Comparison of wall deflection profiles with and without grouted layer (after Yong et al.,1998)

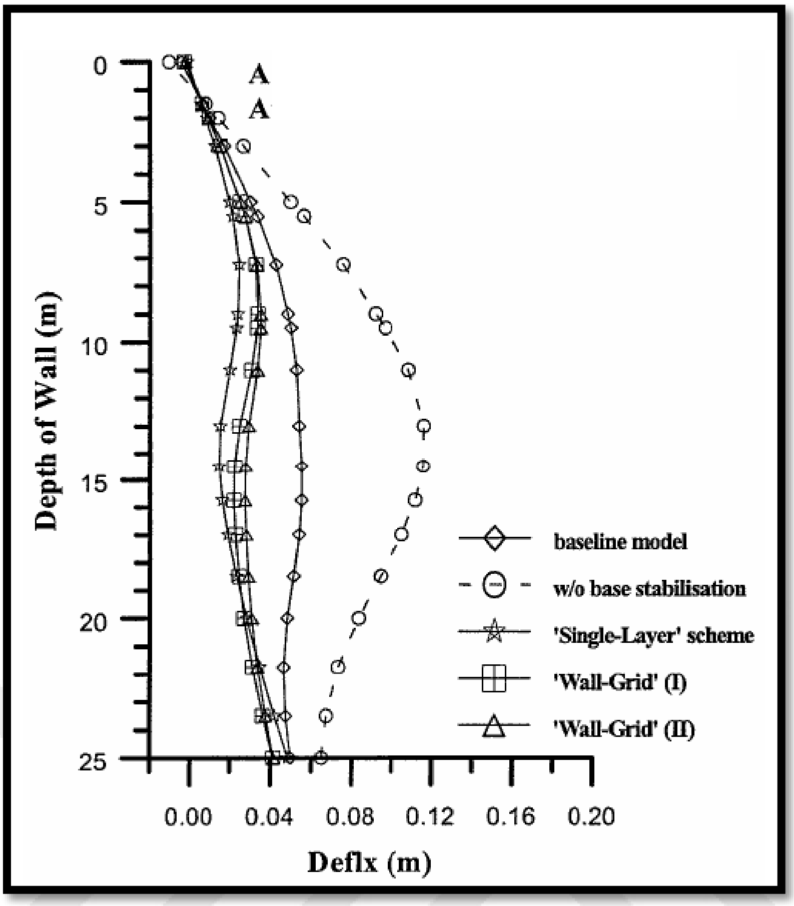

Figure 2.21 Wall deflection profiles of various improvement schemes (after Lim, 1999)

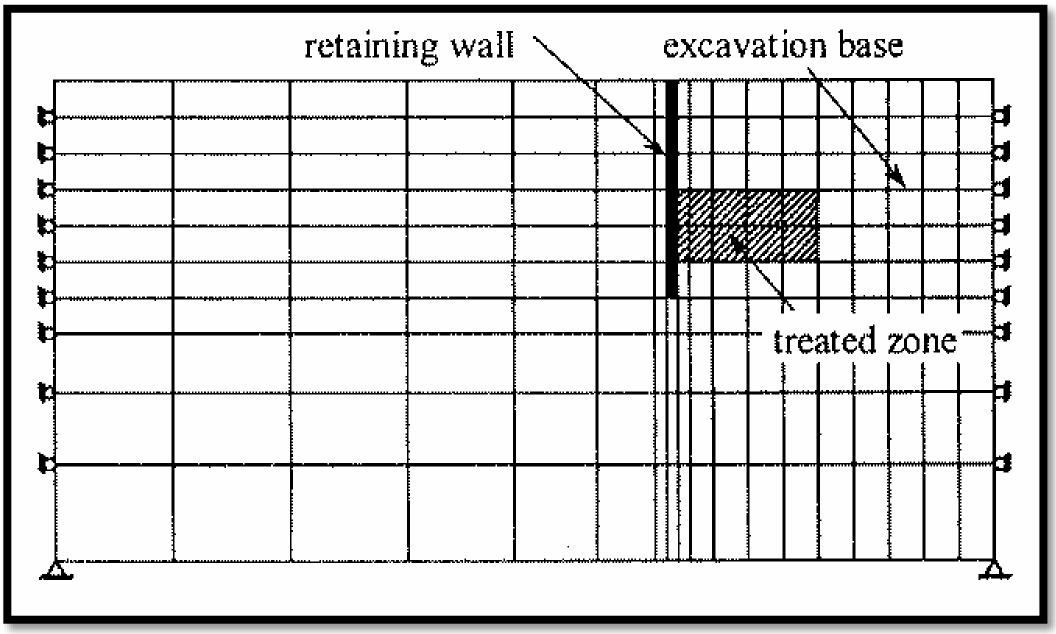

Figure 2.22 Finite element mesh of the cantilever excavation analysis with treated zone in passive side (after Xie et al., 1999)

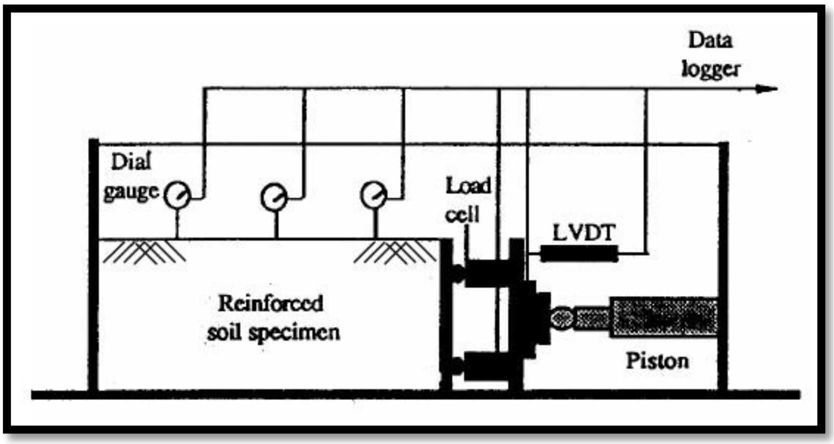

Figure 2.23 Schematic diagram of testing apparatus (after Liao et al., 1993)

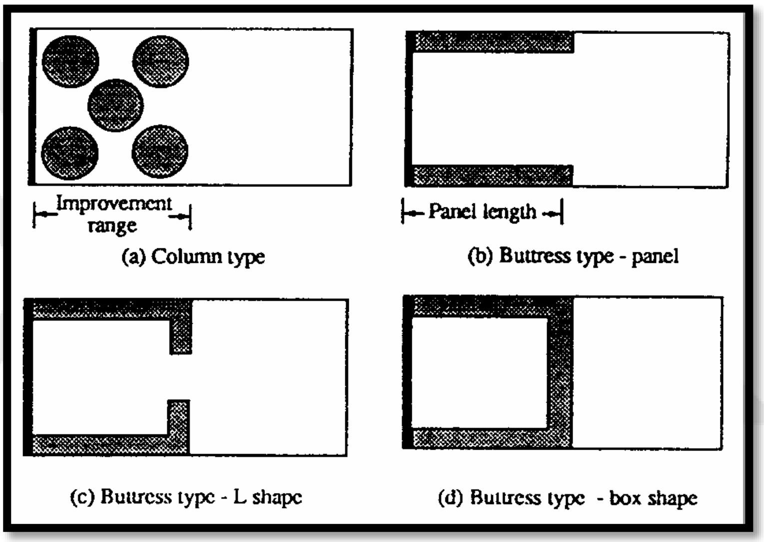

Figure 2.24 Layout patterns for reinforced soil specimens (after Liao et al., 1993)

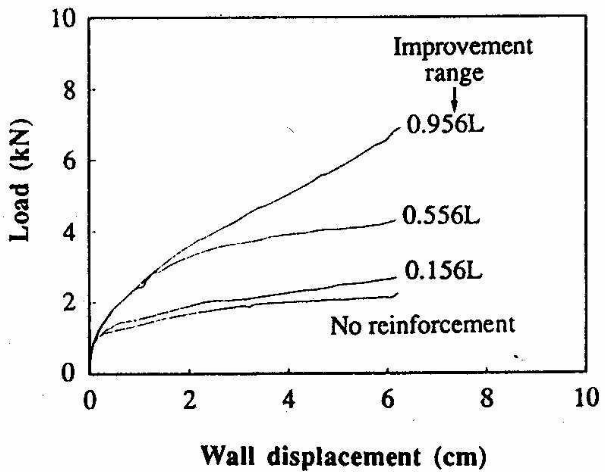

Figure 2.25 Load-deformation relationship for column reinforced specimens with different improvement ranges (after Liao et al., 1993)

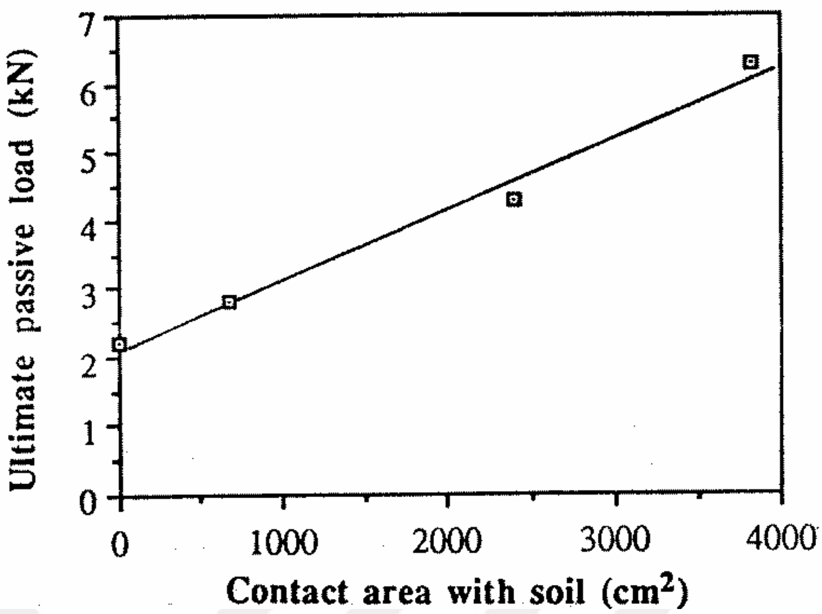

Figure 2.26 Relationship between passive load and contact area for column reinforced

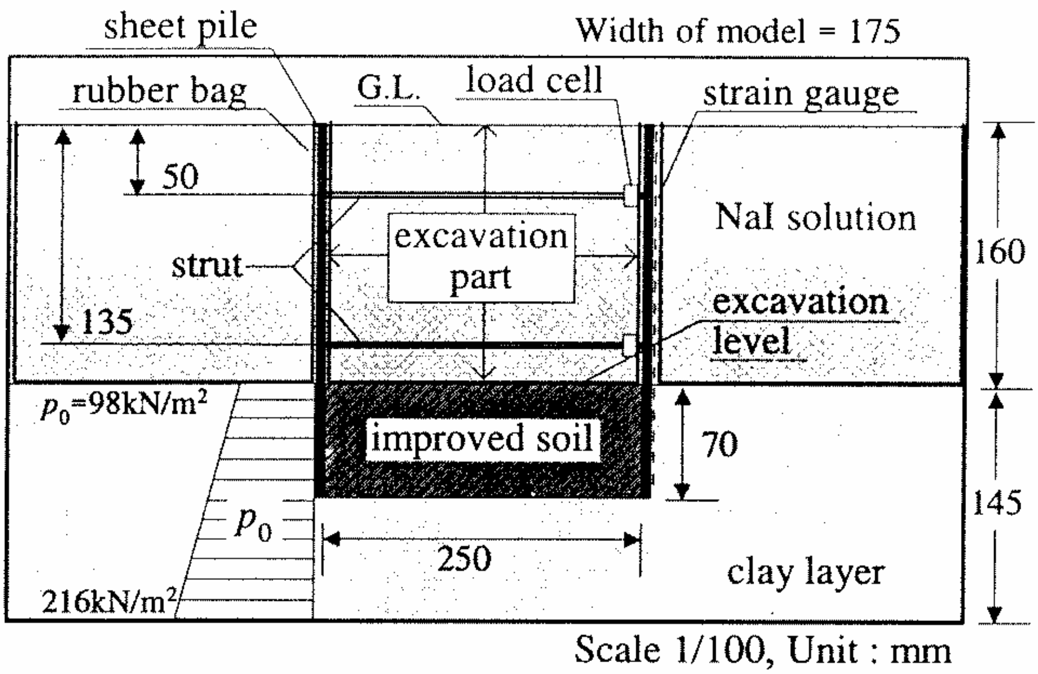

Figure 2.27 Layout of centrifuge model (after Ohnishi et al., 1999)

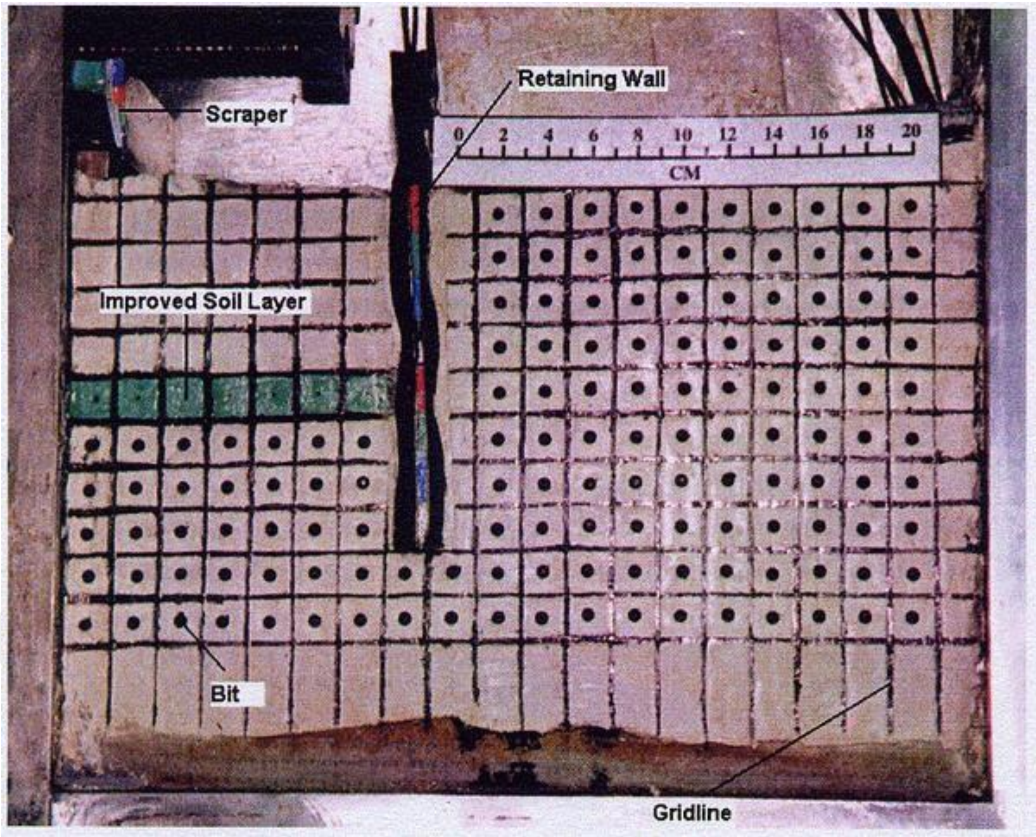

Figure 2.28 Experimental setup for an excavation with an improved soil strut (after Goh, 2004)

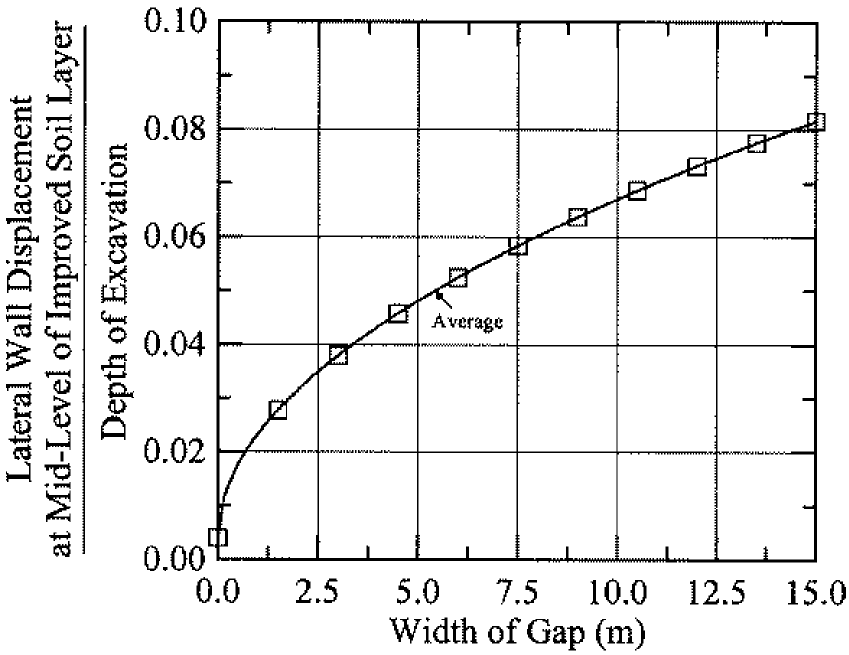

Figure 2.29 Effect of gap width on the lateral normalized wall displacement (after Goh, 2004)

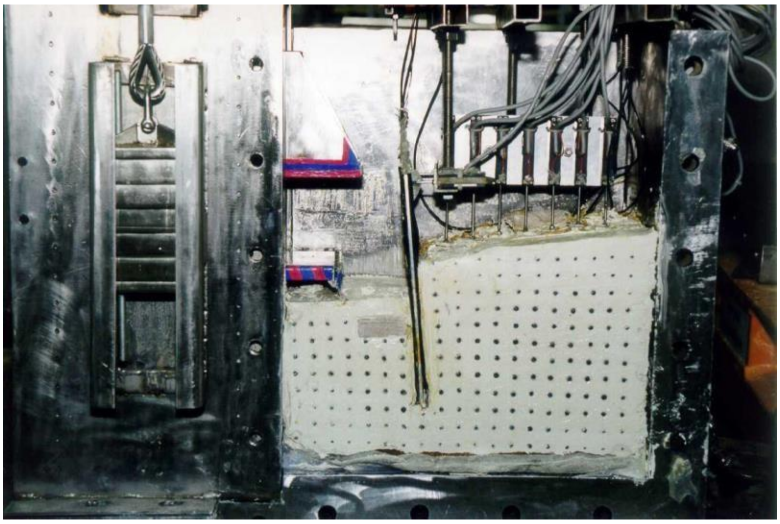

Figure 2.30 Experimental setup for an excavation with an improved soil berm (after Thanadol, 2003)

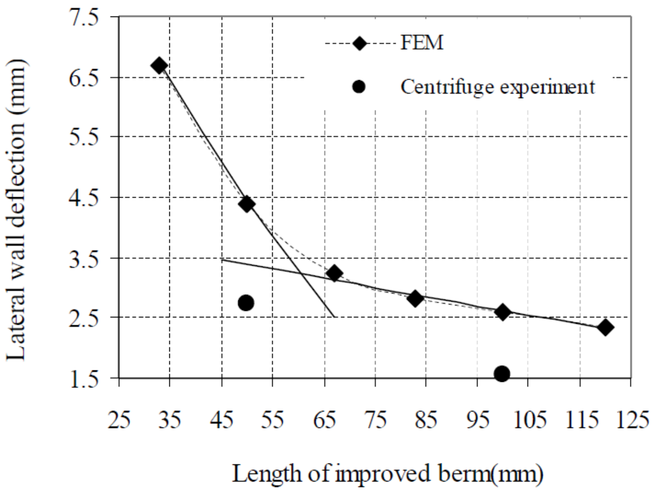

Figure 2.31 Effect of the berm length on the wall movement - model scale (after Thanadol, 2003)

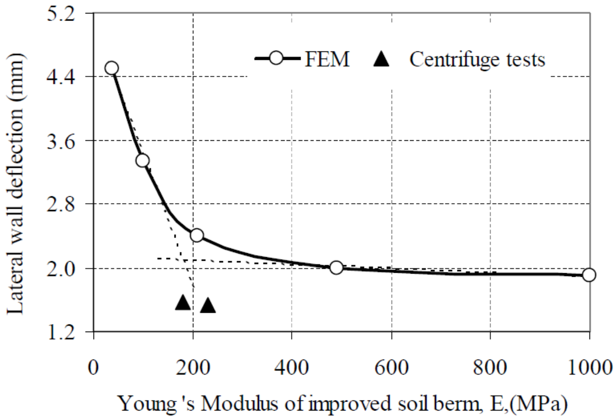

Figure 2.32 Effect of the berm stiffness on the wall movement – model scale (after Thanadol, 2003)

CHAPTER 3

ANALYSIS MADE BY DIFFERENT FINITE ELEMENT METHODS

3.1. The softwares used in this study

In order to make analysis for a case study in deep excavation two common softwares were used. One of them is Plaxis 2D and the other one is DeepXcav. Both of them are very beneficial and common programs to make design and analysis for geotechnical works. When they were compared with each other, there are some differences between each other. Making an analysis with Plaxis 2D is easy, reasonable and reliable, on the other hand for some cases DeepXcav is more remarkable than Plaxis 2D. Therefore, the comparison results that were obtained from Plaxis 2D and DeepXcav were observed and presented in this study.

As it is clearly seen from Table 3.1 Stage dependent analysis, Elastoplastic soil models and Linear-elasticity soil models can be defined for both programs; however, General finite element mesh for soil mass can be defined for only Plaxis 2D program. However, Non-linear winkler spring approach and Subgrade reaction modulus approach can be done for only DeepXcav program. Though there is no limit in using number of walls for Plaxis 2D, number of walls can be defined up to two walls in DeepXcav program. Changes in soil properties at any stage, Groundwater flow analysis and Undrained clay conditions can be done for both programs. While consolidation analysis can be done for only Plaxis 2D, Soil property estimation tools from SPT and CPT can be used for only DeepXcav program. Expansion and contraction effects on struts and slabs can be observed for both programs. Although Limit equilibrium analysis methods can be used for only DeepXcav, Customizable reports in word and pdf can be obtained from both programs. While slope stability analysis can be done for DeepXcav, Only through strength reduction approach can be done for Plaxis 2D. Soil nails and soil nail design can be done for both programs, but only as anchor elements can be defined for Plaxis 2D.

Automatic calculation of wall stiffness and automatic calculation of tieback and strut stiffness can be done in only DeepXcav. Whereas Eurocode 2, 3, 7, and 8 design methods can be used for DeepXcav, Only Eurocode 2 can be applied in Plaxis 2D. AISC, and ACI design codes, AASHTO LRFD combinations, Automatic structural wall optimization and Warnings diagnostic can be used in only DeepXcav program. As it is clearly seen, there are several advantages and disadvantages among two programs. Therefore, a deep excavation work as a case study was solved by using both Plaxis 2D and DeepXcav. And also, their results were compared with each other in this study.

Table 3.1 Comparison the basic features of the two programs for deep excavations, slope stability, and soil nailing

No.	Feature	Plaxis 2D	DeepXcav
1	Stage dependent analysis	Yes	Yes
2	Elastoplastic soil models	Yes	Yes
3	Linear-elasticity soil models	Yes	Yes
4	General finite element mesh for soil mass	Yes	No
5	Non-linear winkler spring approach	No	Yes
6	Subgrade reaction modulus approach	No	Yes
7	Number of walls	No limits	Upto Two walls
8	Changes in soil properties at any stage	Yes	Yes
9	Groundwater flow analysis	Yes	Yes
10	Undrained clay conditions	Yes	Yes
11	Consolidation	Yes	No
12	Soil property estimation tools from SPT and CPT	No	Yes
13	Expansion and contraction effects on struts and slabs	Yes	Yes
14	Limit equilibrium analysis methods	No	Yes
15	Customizable reports in word and pdf	Yes	Yes
16	Slope stability analysis	Only through strength reduction approach	Yes
17	Soil nails and soil nail design	Only as anchor elements	Yes, with Clouterre recommendations
18	Automatic calculation of wall stiffness	No	Yes
19	Automatic calculation of tieback and strut stiffness	No	Yes
20	Eurocode 2, 3, 7, and 8 design methods	Only EC2 can be applied but complex	Automatic
21	AISC, and ACI design codes	No	Yes
22	AASHTO LRFD combinations	No	Yes
23	Automatic structural wall optimization	No	Yes
24	Automatic support optimization	No	Yes
25	Warnings diagnostic	No	Yes

3.2. The case study and site conditions

In this case study, the summary of results that were obtained from the deep excavation Project in Tourism Complex Project that was constructed in Bakırköy-İstanbul were presented. For this work, 10 number of boreholes were done and in all site yellow-brown sand, silty clay and clay and layered limestone were observed. Groundwater level was observed in the region that was near to surface level. In design section, diaphragm wall was selected as retaining structure. Critical retaining wall sections were selected according to the depth of limestone profile. Minimum limestone depth was selected as 13 meter and surcharge load was applied as 15 kPa with respect to adjacent structure and traffic loads. The excavation was supported by the diaphragm wall, whose thickness is 80 cm, and the tiebacks that was located between 180 cm intervals and have 60 tones load capacity (see Figure 3.1).

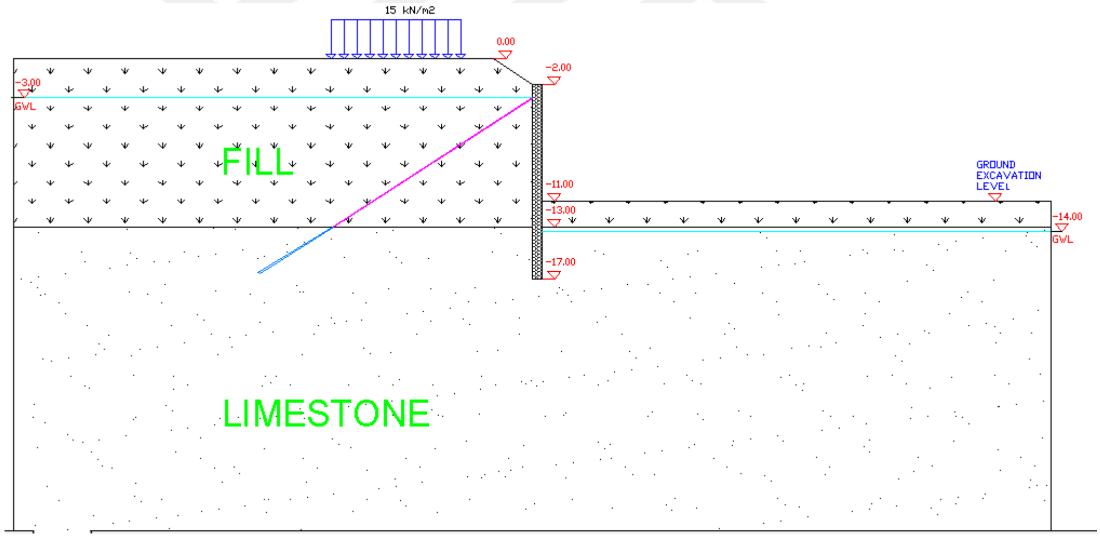

Figure 3.1 General section of the case study.

The soil parameters were given in Table 3.2 and according to these parameters analysis were done by using Plaxis 2D and Deepxcav software programs. And also, the materials properties of Diaphragm Wall, grout root of anchor (geogrid) and anchor were presented in Tables 3.3-3.4-3.5.

Table 3.2 Geotechnical parameters of the Fill and Limestone for Hardening Soil Model

Identification	**Fill (HS Model)**	**Limestone (HS Model)**
Drainage type	Drained	Drained
γ_{unsat} (kN/m^3)	18	20
γ_{sat} (kN/m^3)	19	20
Dilatancy cut-off	No	No
e_{init}	0.5	0.5
e_{min}	0.0	0.0
e_{max}	999.0	999.0
Rayleigh α	0.0	0.0
Rayleigh β	0.0	0.0
E_{50}^{ref} (kN/m^2)	10.0E3	150.0E3
E_{oed}^{ref} (kN/m^2)	10.0E3	150.0E3
E_{ur}^{ref} (kN/m^2)	30.0E3	450.0E3
power (m)	0.5	0.5
Use alternatives	No	No
C_c	0.03450	2.300E-3
C_s	0.01035	0.6900E-3
e_{init}	0.5	0.5
c_{ref} (kN/m^2)	1.0	20.0
φ (phi)	30	38
ψ (psi)	0.0	8.0
Set to default values	Yes	Yes
ν_{ur}	0.2	0.2
p_{ref}	100.0	100.0
$K^{nc}{}_0$	0.5	0.3843
c_{inc} (kN/m^2/m)	0.0	0.0
y_{ref} m	0.0	0.0
R_f	0.9	0.9
Tension cut-off	Yes	Yes
Tensile strength (kN/m^2)	0.0	0.0
Strength	Manual	Rigid
R_{inter}	0.9	1.0
δ_{inter}	0.0	0.0
K_0 determination	Automatic	Automatic
$K_{0,x}$	0.5	0.3843
OCR	1.0	1.0
POP (kN/m^2)	0.0	0.0
Data set	Standard	Standard

Table 3.3 Engineering parameters of the Diaphragm Wall

Identification		**DW-80 (80 cm-Diaphragm Wall)**
Material type		Elastic
Isotropic		No
EA_1	kN/m	20.87E6
EA_2	kN/m	0.0
EI	$kN.m^2/m$	1.113E6
d	m	0.8
w	kN/m/m	4.0
ν (nu)		0.0
Rayleigh α		0.0
Rayleigh β		0.0

Table 3.4 Engineering parameters of the grout (Geogrid)

Identification		**Geogrids (Grout)**
Material type		Elastic
Isotropic		Yes
EA_1	kN/m	236.4E3
EA_2	kN/m	236.4E3

Table 3.5 Engineering parameters of the node to node anchor

Identification		**4*0,6"/1,8** (In one anchor, 4 number of 0.6 cm-steel bar for each 1.8 m intervals)
Material type		Elastic
EA	kN/m	145.9E3
$L_{spacing}$	m	1.8

In order to define stiffness of the diaphragm wall in DeepXcav program, reinforcement details must be entered to the program. However, in Plaxis 2D design section stiffness of the structural elements can be directly entered as EA and EI. Therefore, in order to make an analysis in DeepXcav, the reinforcement elements details must be defined to the program. Since all design parameters was known in this study, the calculated reinforcement results were directly used. The detailed calculation process of the reinforcement for the diaphragm wall was presented below.

Table 3.6 Diaphragm wall reinforcement calculations

Diaphragm Wall Reinforcement Calculations							
Diyafram Duvar Donatı Hesabı							
Section No. / Kesit No. :			**1 boyuna donatı**				
Section Dimensions / Kesit Boyutları					**Materials / Malzemeler**		
bw:	3 m	**d':**	5 cm		**Concrete / Beton:**		C35
h:	80 cm	**d:**	75 cm		**Reinf. / Demir:**		STIIIa
Longitudinal Reinforcements / Boyuna Donatılar							
Excavation Side / Kazı Tarafı				**Soil Side / Toprak Tarafı**			
	M (kNm/m)	FS	Md (kNm/m)		M (kNm/m)	FS	Md (kNm/m)
static:			1685	static:			263
earthquake:				earthquake:			
Md:	5054	kNm		Md:	790	kNm	
K (x10^-5) = bw x d² / Md :			33	K (x10^-5) = bw x d² / Md :			214
		ks :	3.12			ks :	2.81
As = ks x Md / d =		210	cm²	As = ks x Md / d =		30	cm²
Asmin=	60 cm	cm²		Asmin=	60 cm	cm²	
Ø32 /	**10 cm**	217	cm²	**Ø24 /**	**22 cm**	59	cm²
Nos. Reinf./Donatı Adedi:			27	Nos. Reinf./Donatı Adedi:			13
Transverse Reinforcements / Enine Donatılar							
	V (kN/m)	FS	Vd (kN/m)				
static:	399	1.35	539		fctd=		1.15 N/mm²
earthquake:					fywd =		365 N/mm²
Vd:	1616	kN			Ac=		2.4 m²
V=Vc+Vw,	Vcr= 0,65 x fctd x A,		Vc= 0.8 x Vcr =		1435	kN	
Ø14	**/20 cm**			Asw =	616	mm²	
n=	4			Vw =	843	kN	
V=Vc+Vw=	2278 kN	>	1616 kN				

CHAPTER 4

TEST RESULTS AND DISCUSSIONS

4.1. Plaxis 2D test results

The excavation work was done by using Plaxis 2D geotechnical software program and its solution results were presented following Figures.4.1-4.7. According to Figure 4.1, horizontal displacements for the excavation section obtained from Plaxis 2D was shown. The extreme total horizontal displacement behind of the excavation was calculated as 32,46 x E-3 meter. Shading view of horizontal displacement is seen Figure 4.1. Extreme total displacement for excavation section obtained from Plaxis 2D is also shown in Figure 4.2. according to this figure, extreme total displacement behind of the excavation site was calculated as 64,61xE-3 meter. Finite element mesh is also shown in this figure. Degree of saturation for excavation section obtained from Plaxis 2D is given Figure 4.3. According to this figure, 100 % degree of saturation region that was obtained after excavation was completed is clearly seen. Shading view of degree of saturation is also seen Figure 4.3. Flow field for excavation section obtained from Plaxis 2D is demonstrated in Figure 4.4. according to this figure, extreme water velocity for flow field was calculated as 31,02 m/day. According to this value, total water amount that entered to the site from the bottom of the excavation level can be calculated easily. Therefore, some calculations related with discharge of the water from the excavation site can be easily done. Horizontal displacement on diaphragm wall for excavation section obtained from Plaxis 2D is shown in Figure 4.5. According to this figure, maximum horizontal displacement on diaphragm wall for excavation section was obtained as 32,44xE-3 meter. This value may be said that it is in the acceptable scale. Therefore controlling the movement of the wall after excavation is completed is vital for construction work, the calculation of this value as acceptable is one of the most important issues.

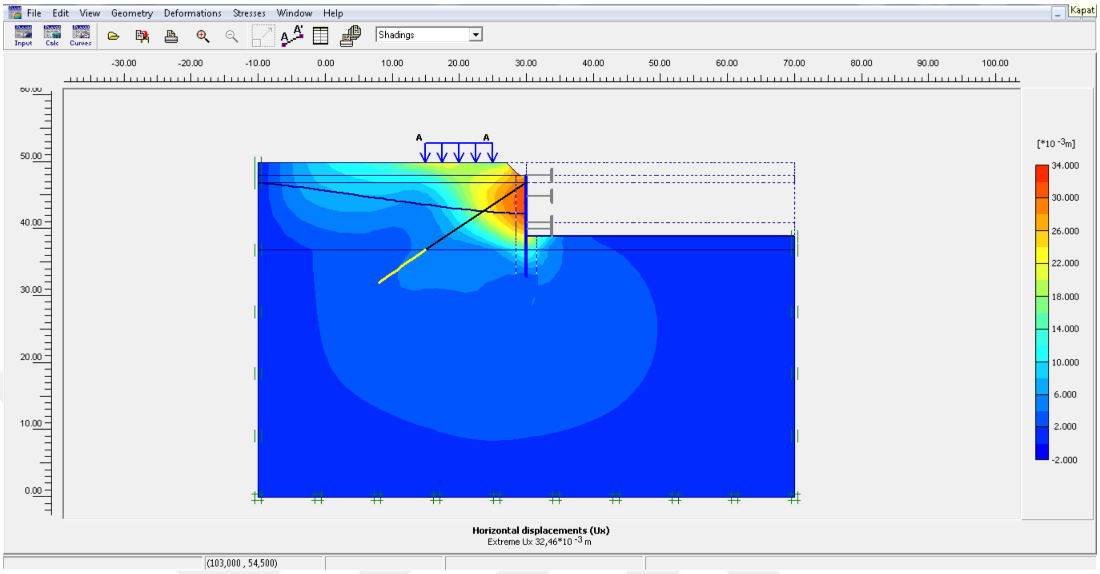

Figure 4.1 Horizontal displacement for excavation section obtained from Plaxis 2D

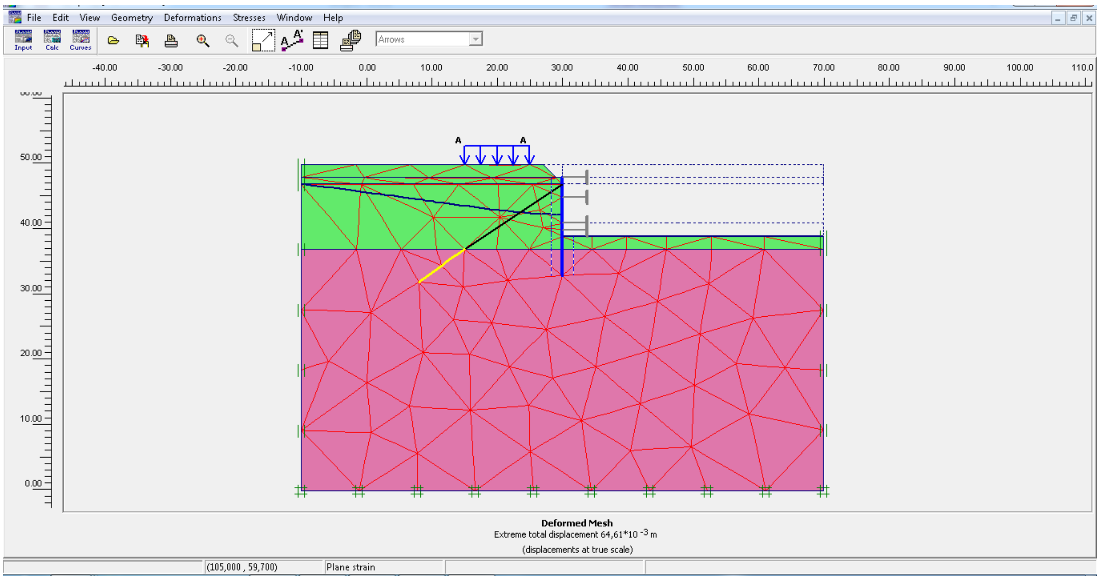

Figure 4.2 Extreme total displacement for excavation section obtained from Plaxis 2D

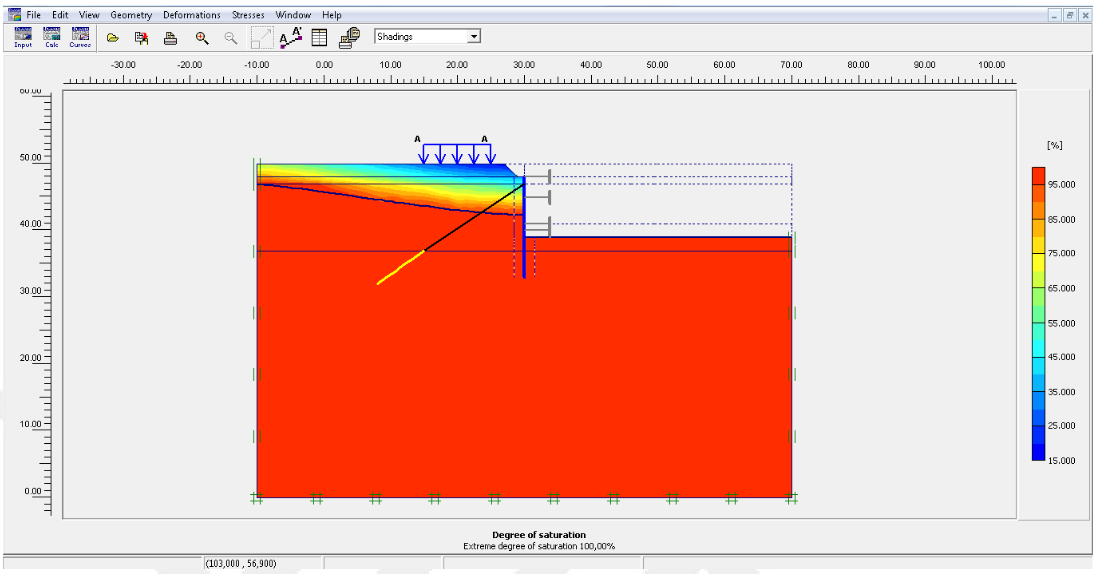

Figure 4.3 Degree of saturation for excavation section obtained from Plaxis 2D

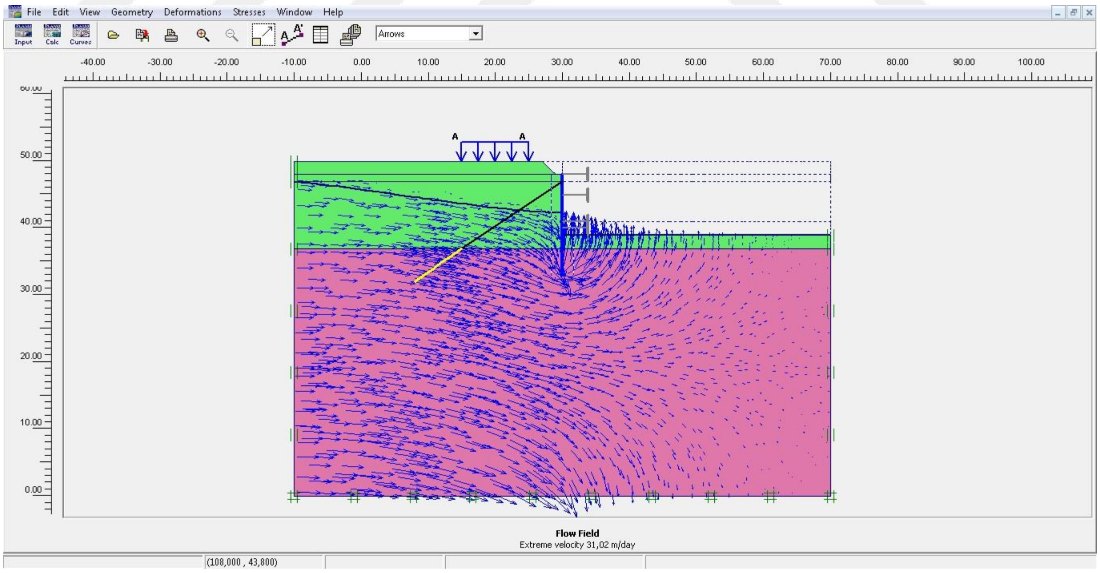

Figure 4.4 Flow field for excavation section obtained from Plaxis 2D

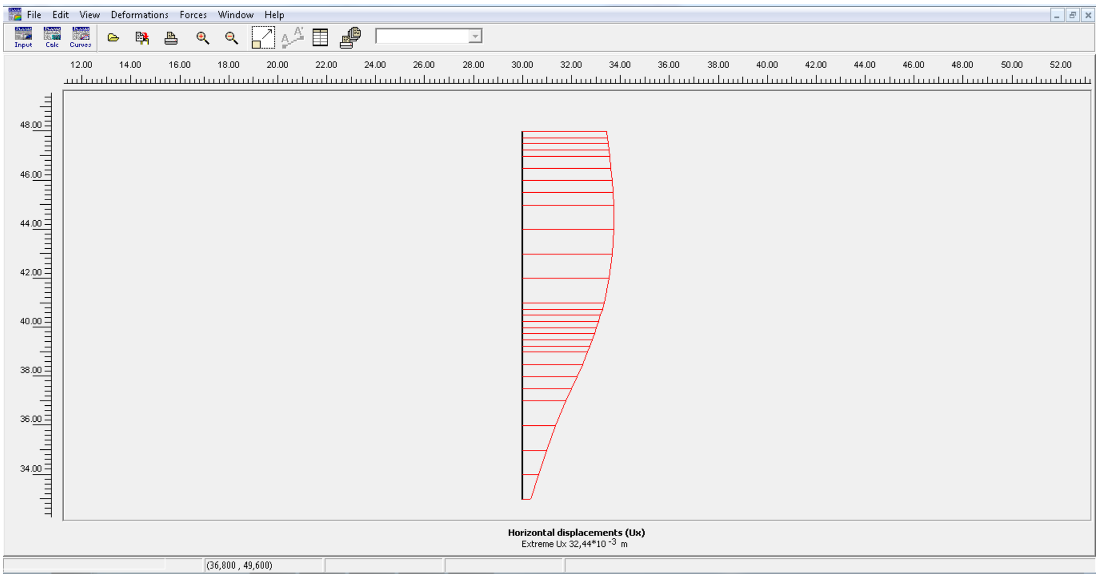

Figure 4.5 Horizontal displacement on diaphragm wall for excavation section obtained from Plaxis 2D

Shear forces on diaphragm wall for excavation section obtained from Plaxis 2D is presented in Figure 4.6. According to this figure, all shear forces acted on the diaphragm wall can be easily seen. The maximum shear force on the diaphragm wall was obtained as 341,45 kN/m. As the reinforcement design can be done according to this value, the calculation of this value as correctly is vital geotechnical engineering.

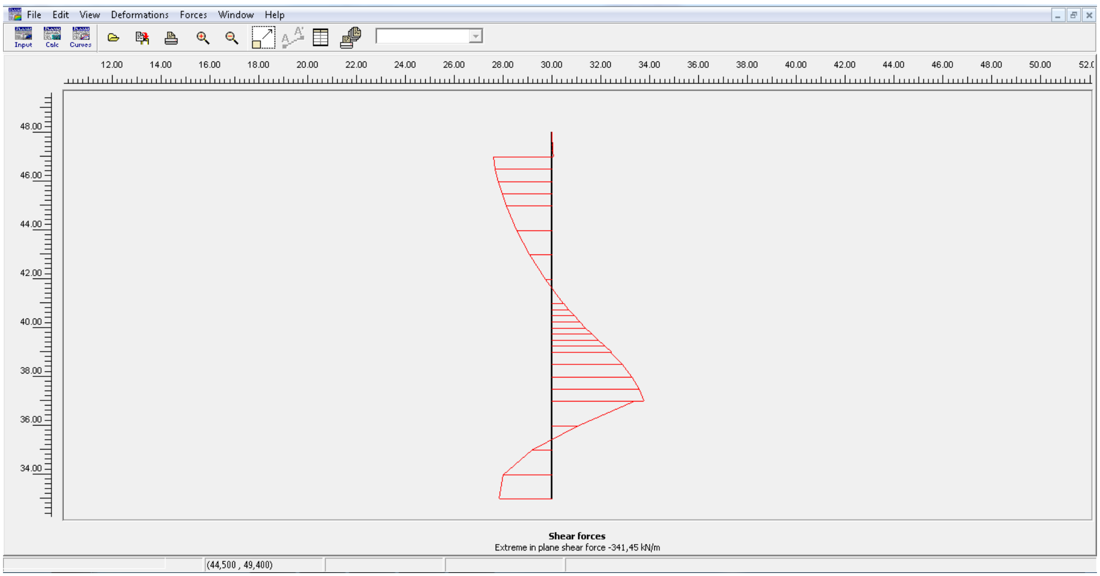

Figure 4.6 Shear forces on diaphragm wall for excavation section obtained from Plaxis 2D

Bending moments on diaphragm wall for excavation section obtained from Plaxis 2D is also presented in Figure 4.7. According to this figure, all bending moments acted on the diaphragm wall can be easily seen.

The maximum bending moment on the diaphragm wall was obtained as 713,09 kN.m/m. As the reinforcement design can be done according to this value, the calculation of this value as correctly is also vital geotechnical engineering.

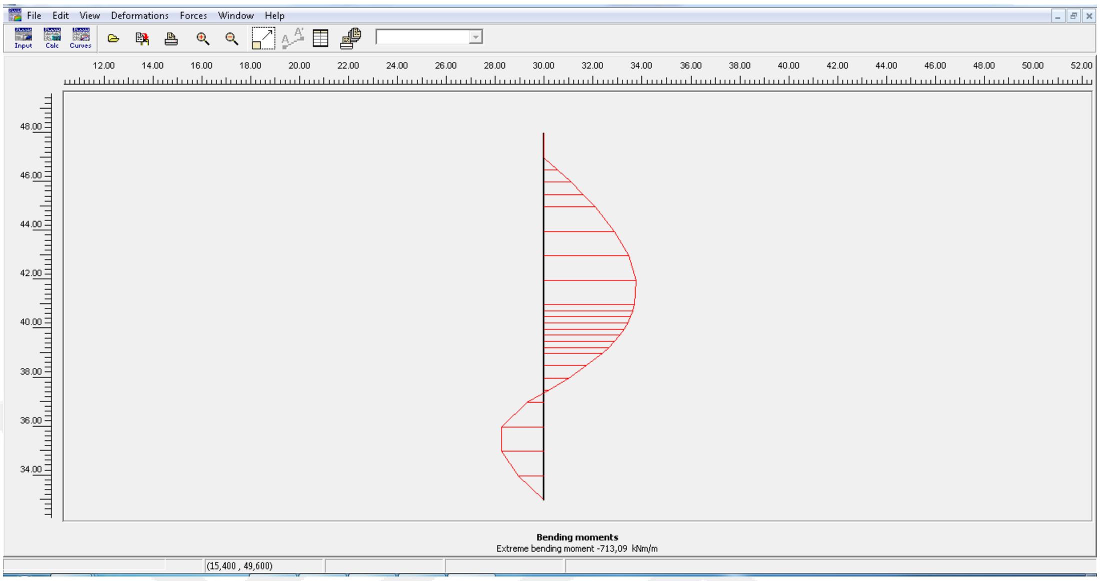

Figure 4.7 Bending moments on diaphragm wall for excavation section obtained from Plaxis 2D

4.2. DeepXcav test results

The excavation work was done by using DeepXcav geotechnical software program and its solution results were presented following Figures.4.8-4.11. According to Figure 4.8, general cross section view of excavation region obtained from DeepXcav was shown. Shear forces on diaphragm wall for excavation section obtained from DeepXcav is presented in Figure 4.9. According to this figure, all shear forces acted on the diaphragm wall can be easily seen. The maximum shear force on the diaphragm wall was obtained as 360,8 kN/m. As the reinforcement design can be done according to this value, the calculation of this value as correctly is vital geotechnical engineering.

Bending moments on diaphragm wall for excavation section obtained from DeepXcav is also presented in Figure 4.10. According to this figure, all bending moments acted on the diaphragm wall can be easily seen. The maximum bending moment on the diaphragm wall was obtained as 1201,7 kN.m/m. As the reinforcement design can be done according to this value, the calculation of this value as correctly is also vital geotechnical engineering.

Horizontal displacement on diaphragm wall for excavation section obtained from DeepXcav is shown in Figure 4.11. According to this figure, maximum horizontal displacement on diaphragm wall for excavation section was obtained as 2,33 cm. This value may be said that it is in the acceptable scale. Therefore controlling the movement of the wall after excavation is completed is vital for construction work, the calculation of this value as acceptable is one of the most important issues.

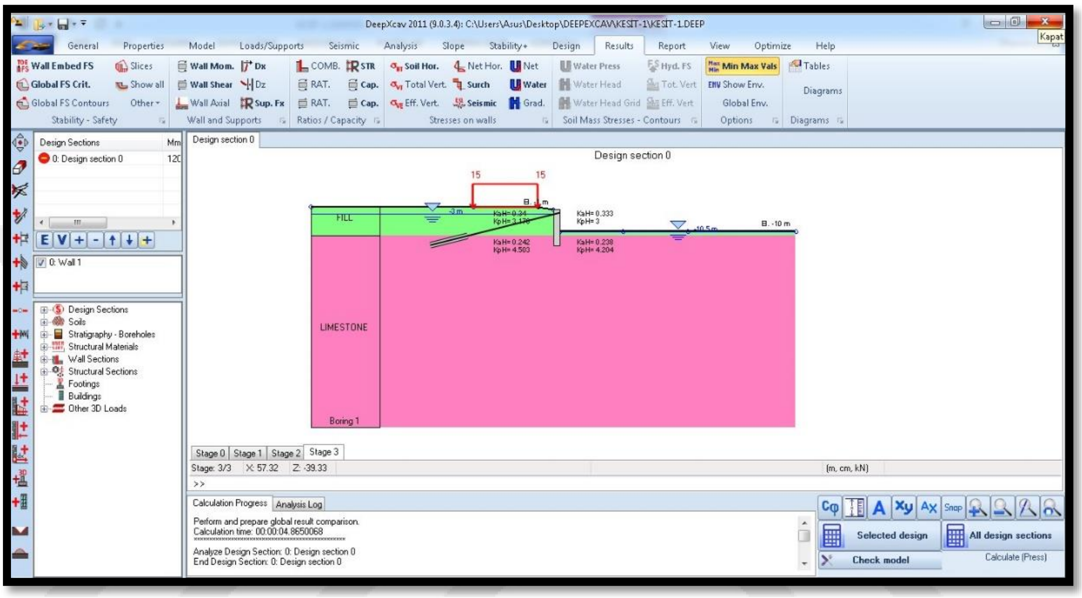

Figure 4.8 General cross section view of excavation region obtained from DeepXcav

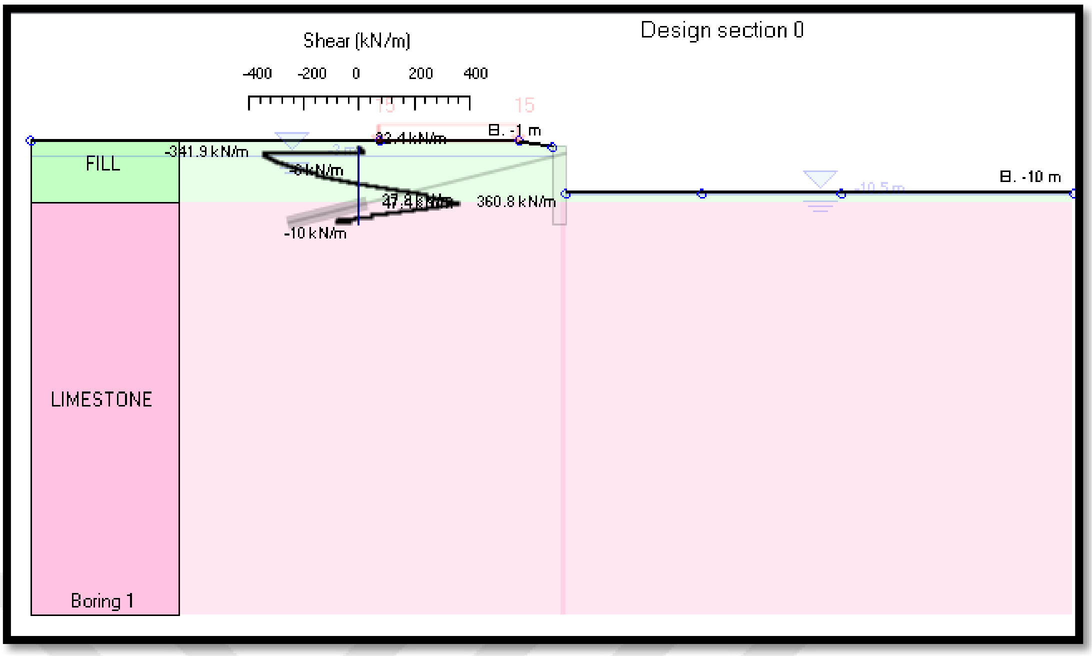

Figure 4.9 Shear forces on diaphragm wall for excavation section obtained from DeepXcav

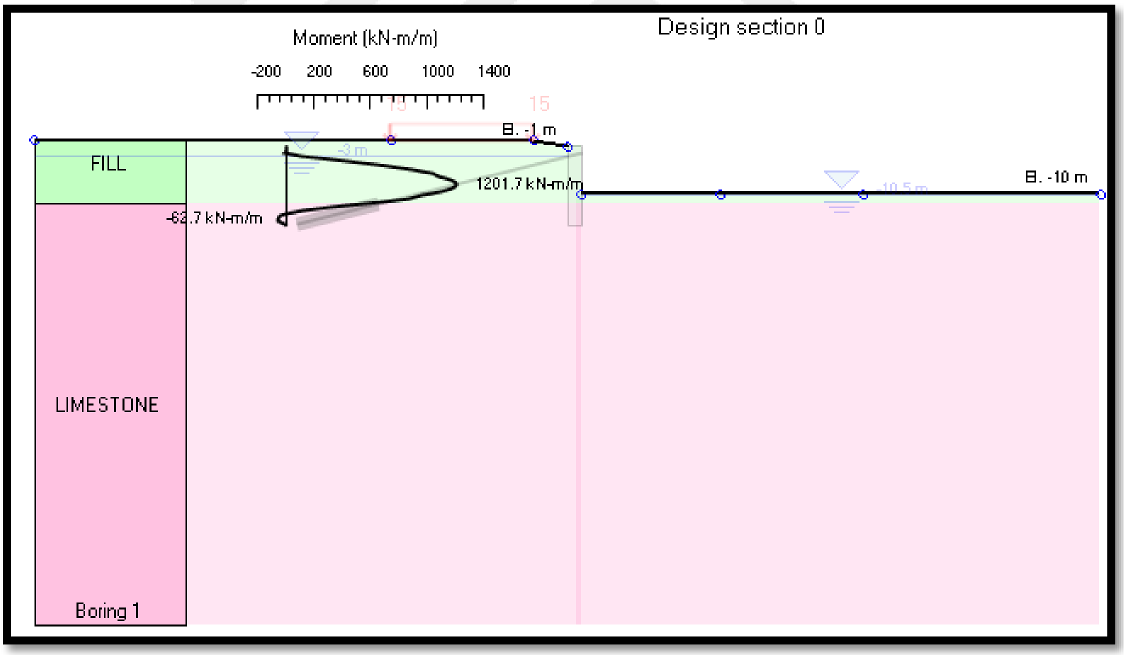

Figure 4.10 Bending moments on diaphragm wall for excavation section obtained from DeepXcav

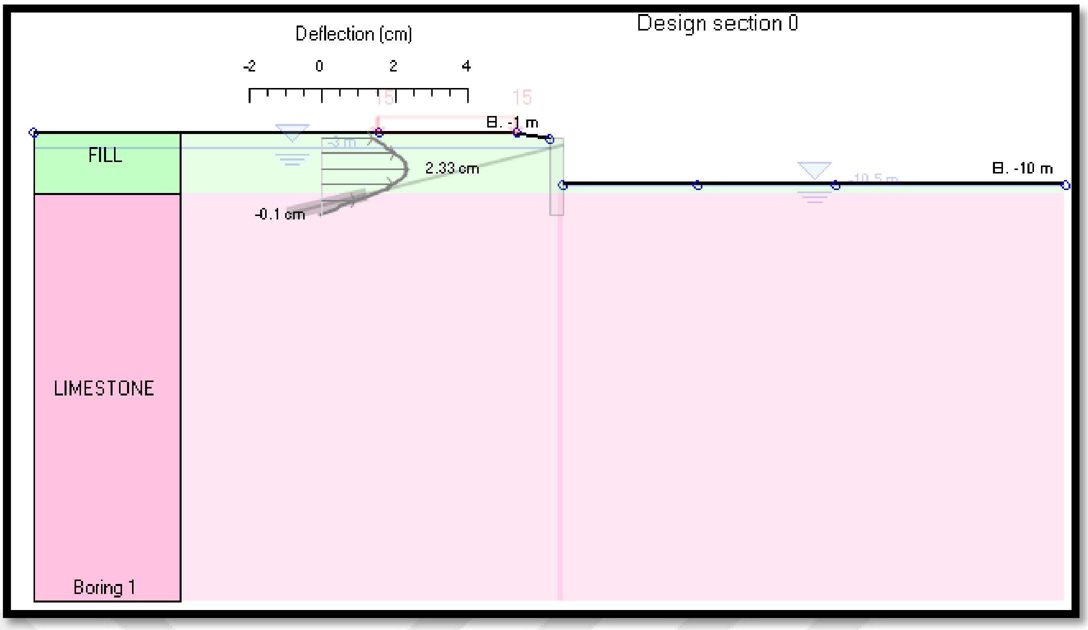

Figure 4.11 Horizontal displacement on diaphragm wall for excavation section obtained from DeepXcav

4.3. Comparison test results obtained from Plaxis 2D and DeepXcav with measured results in the site

Comparison test results obtained from the both software programs with measured real datas in the site are presented Table 4.1 and Figure 4.12. According to Table 4.1, there is a difference among bending moments values that were obtained from the analysis. The bending moment value that was obtained from DeepXcav is greater than the other obtained from Plaxis 2D. This difference may be based on the stiffness of the structural element. While stiffness values directly entered to Plaxis 2D as EA and EI, this value was estimated automatically in DeepXcav solution because of entering directly reinforcement details. However, the selected design value is greater than both Plaxis 2D and DeepXcav. Therefore, it can be said that both bending moments that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable. On the other hand, the shear forces that were obtained from the Plaxis 2D and DeepXcav are near to the each other. Furthermore, the selected design value for shear force is greater than both Plaxis 2D and DeepXcav. Therefore, it can be said that both shear forces that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable. And also, the horizontal displacement that were obtained from the Plaxis 2D and DeepXcav are very near to the each other.

Moreover, the measured real value by an inclinometer for horizontal displacement is also very near to both Plaxis 2D and DeepXcav. As a result, both Plaxis 2D and DeepXcav can estimate the horizontal displacement accurately. Therefore, it can be said that both horizontal displacement that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable.

Table 4.1 Comparision results for all outputs

	Plaxis 2D	**DeepXcav**	**Measured results in the site**
Bending moment	713,09	1201,7	1544 (Design value)
Shear force (kN/m)	341,45	360,75	428 (Design value)
Horizontal displacement (cm)	3,24	2,33	2,05 (measured by inclinometer)
Flow Field (water flow) (m^3/day/m)	3,16	-	2,18

Horizontal displacement estimation for excavation work is vital for geotechnical engineers. If this displacement could not be controlled and estimated correctly, this would be disaster. The movement of the wall must be under the control and in the safe and acceptable rage. Therefore, geotechnical engineers always use inclinometer in order to observe the movement of the wall during/after excavation work is done. In this study, horizontal measurements were collected by using inclinometers in order to observe and control the displacement. According to the inclinometer results (see Figure 4.12) the maximum horizontal displacement was measured as 2,05 cm. Therefore, it can be said that both horizontal displacement that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable in order for near to measured value.

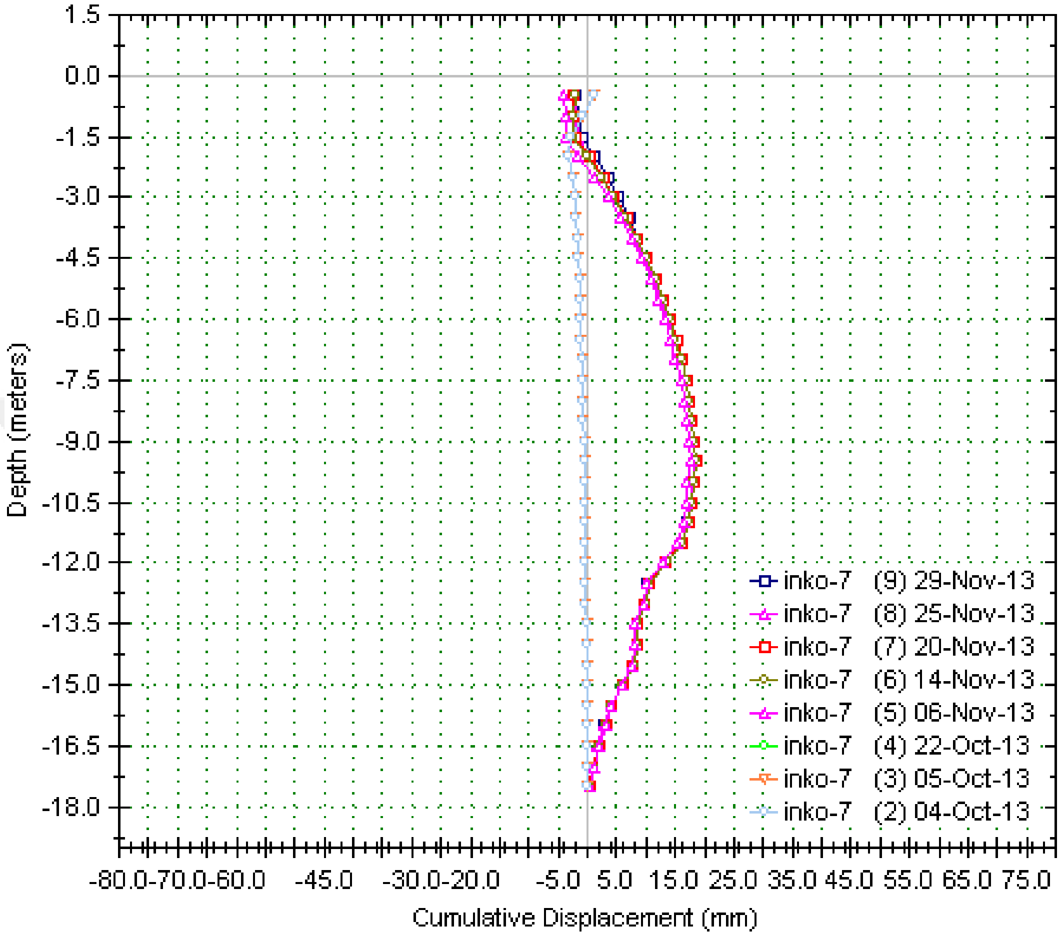

Figure 4.12 Inclinometer results obtained from the back site of the diaphragm wall

CHAPTER 5

CONCLUSION

The conclusions that were extracted from this study given as following below;

- In this case study, the summary of results that were obtained from the deep excavation Project in Tourism Complex Project that was constructed in Bakırköy-İstanbul were presented.
- 10 number of boreholes were done and in all site yellow-brown sand, silty clay and clay and layered limestone were observed.
- The extreme total horizontal displacement and extreme total displacement behind of the excavation were calculated as 32,46xE-3 and 64,61xE-3 meter, respectively.
- Extreme water velocity for flow field was calculated as 31,02 m/day in Plaxis 2D.
- Maximum horizontal displacement on diaphragm wall for excavation section was obtained as 32,44xE-3 meter in Plaxis 2D. This value may be said that it is in the acceptable scale. Therefore controlling the movement of the wall after excavation is completed is vital for construction work, the calculation of this value as acceptable is one of the most important issues.
- The maximum shear force on the diaphragm wall was obtained as 341,45 kN/m in Plaxis 2D. As the reinforcement design can be done according to this value, the calculation of this value as correctly is vital geotechnical engineering.
- The maximum bending moment on the diaphragm wall was obtained as 713,09 kN.m/m in Plaxis 2D. As the reinforcement design can be done according to this value, the calculation of this value as correctly is also vital geotechnical engineering.
- The maximum shear force on the diaphragm wall was obtained as 360,8 kN/m in DeepXcav. As the reinforcement design can be done according to this value, the calculation of this value as correctly is vital geotechnical engineering.

- The maximum bending moment on the diaphragm wall was obtained as 1201,7 kN.m/m in DeepXcav. As the reinforcement design can be done according to this value, the calculation of this value as correctly is also vital geotechnical engineering.
- Maximum horizontal displacement on diaphragm wall for excavation section was obtained as 2,33 cm in DeepXcav. This value may be said that it is in the acceptable scale. Therefore controlling the movement of the wall after excavation is completed is vital for construction work, the calculation of this value as acceptable is one of the most important issues.
- The bending moment value that was obtained from DeepXcav is greater than the other obtained from Plaxis 2D. This difference may be based on the stiffness of the structural element. While stiffness values directly entered to Plaxis 2D as EA and EI, this value was estimated automatically in DeepXcav solution because of entering directly reinforcement details.
- The selected design value is greater than both Plaxis 2D and DeepXcav. Therefore, it can be said that both bending moments that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable.
- The shear forces that were obtained from the Plaxis 2D and DeepXcav are near to the each other. Furthermore, the selected design value for shear force is greater than both Plaxis 2D and DeepXcav. Therefore, it can be said that both shear forces that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable.
- The horizontal displacement that were obtained from the Plaxis 2D and DeepXcav are very near to the each other. moreover, the measured real value by an inclinometer for horizontal displacement is also very near to both Plaxis 2D and DeepXcav. As a result, both Plaxis 2D and DeepXcav can estimate the horizontal displacement accurately. Therefore, it can be said that both horizontal displacement that were obtained from Plaxis 2D and DeepXcav are considerable and acceptable

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