

FINITE ELEMENT STUDY ON AXIALLY STATIC AND BIDIRECTIONAL STATIC
OSTERBERG CELL PILE LOAD TESTING

by

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ABSTRACT

FINITE ELEMENT STUDY ON AXIALLY STATIC AND BIDIRECTIONAL STATIC OSTERBERG CELL PILE LOAD TESTING

There are several methods used in the design of bored piles. Mathematical modeling of the pile and soil parameters, provide various formulas and theories for ultimate bearing capacity and settlement calculations of a pile. Many factors, including construction methods and real site conditions affect the vertical load capacity, so such model based analyses seldom give fully reliable results. From that reason, pile load tests are the most fundamental part of pile foundation design. Although many pile tests have been constructed in all kinds of engineering projects, it is unclear what difference arises from newer test methods such as the O-cell test.

In this thesis, the finite element method (FEM) was used to carry out the research. The commercial finite element code PLAXIS was used for the numerical simulation of pile load test in the following manner.

The thesis focuses on some particular interest which is to compare the allowable capacity and displacement values of several test results with each other and also with the numerical simulation of five different case studies. Furthermore, O-cell test is compared with static pile load test and equivalency and discrepancy of the test results between the two types of pile load test are demonstrated and analyzed. It is concluded that O-cell test result can provide not only the same soil-pile interaction information as conventional head-down static loading test, but also allow for separate determination of the shaft resistance and end bearing components. However, the equivalent head down load-movement curve of the O-cell test simulated by PLAXIS 8 gives a slightly stiffer load-movement response and slightly higher ultimate capacity than those of conventional test.

ÖZET

STATİK VE O-CELL KAZIK YUKLEME TESTLERİ UZERİNE SONLU ELEMENLAR METODU İLE ÇALIŞMA

Fore kazık tasarımında kullanılan birkaç yöntem vardır. Kazık ve zemin parametrelerinin matematiksel modellemesi, nihai taşıma kapasitesi ve kazığın oturum hesaplamaları için çeşitli formüller ve teoriler sağlar. İnşaat yöntemleri ve gerçek saha koşulları da dahil olmak üzere, birçok faktör, düşey yük kapasitesini etkiler, bu nedenle böyle model tabanlı analizler nadiren ve tamamen güvenilir sonuçlar verir. Bu nedenden dolayı, kazık yükleme deneyleri kazıklı temel tasarımının en temel parçasıdır. Her türlü mühendislik projelerinde birçok kazık testi kullanılmış olsa da, O-Cell testi gibi yeni test yöntemlerinden ne gibi farkların doğacağı henüz belirsizdir.

Bu tezde, araştırmayı yürütmek için sonlu elemanlar yöntemi (FEM) kullanılmıştır. Ticari sonlu elemanlar kodu PLAXIS kazık yükleme testlerinin sayısal simülasyonu için aşağıdaki şekilde kullanılmıştır.

Tez; beş farklı vaka çalışmasının test sonuçlarının kapasite ve deplasman değerlerinin birbirleriyle ve aynı zamanda bu beş vaka çalışmasının nümerik simülasyonu ile karşılaştırılması üzerine odaklanır. Ayrıca, O-Cell ve statik kazık yükleme testleri birbirleriyle karşılaştırılır; bu iki test sonuçlarının benzerlikleri ve farklılıkları analiz edilip açıklanır. O-Cell test sonuçları, geleneksel statik yükleme testleri gibi sadece zemin-kazık ilişkisi bilgisini vermez; aynı zamanda kazık yüzey direnci ve uç taşıma kapasitesinin ayrı ayrı belirlenmesine olanak sağlar. Fakat, Plaxis ile simüle edilen O-Cell testi geleneksel statik yükleme testinden daha dik yük-hareket eğrisi ve daha yüksek nihai kapasite sonuçları verir.

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LIST OF SYMBOLS

c	Cohesion
c_{actual}	Actual cohesion
c_i	Cohesion of interface element
$c_{\text{increment}}$	The increase of cohesion per unit depth
c_{ref}	Reference cohesion
c_{soil}	Cohesion of soil
c_u	Undrained shear strength
E	Young's modulus
E_{50}	Confining stress-dependent stiffness modulus for primary loading
E_{50}^{ref}	Reference stiff modulus corresponding to the reference confining pressure
E_{actual}	Actual Young's modulus
E_i	Young's modulus of interface element
$E_{\text{increment}}$	The increase of the Young's modulus per unit of depth
E_{ref}	Reference Young's modulus
E_s/E_{soil}	Young's modulus of soil
E_p	Young's modulus of pile
E_{oed}	Tangent stiffness for primary oedometer loading
E_u	Undrained state young's modulus
E_{ur}	Reference Young's modulus for unloading/reloading
F	Adjustment factor
G	Shear modulus
K	Pile stiffness factor
K'	Effective bulk modulus
K_w	Bulk modulus of water

K_o	Coefficient of lateral stress in in-situ condition
K_o^{NC}	Coefficient of lateral stress in normal consolidation
L-E	Linear Elastic
M-C	Mohr Coulomb
N	Porosity
OLT	Osterberg load test
$Q_u = P$	Ultimate load bearing capacity
$Q_s = F$	Ultimate friction bearing capacity
$Q_p = Q$	Ultimate end bearing (point or tip) capacity
PC	Personal computer
p_{steady}	Steady state pore stress
R_{inter}	Interface strength reduction factor
t-y	Side shear vs vertical movement
u_{excess}	Excess pore water pressure
u_{steady}	Steady state excess pore water pressure
U	Displacement
x	Dimension
y	Dimension
y_{ref}	Reference depth
γ_{unsat}	Unsaturated unit weight of soil
γ_{sat}	Saturated unit weight of soil
γ_w	Unit weight of water
$\underline{\sigma}'$	Vector notation of effective normal stress
σ_n	Normal stress of soil
σ_w	Pore pressure
ϵ_{ij}	Cartesian normal strain component

τ	Shear strength of soil
ν	Poisson's ratio
ν_u	Poisson's ratio for undrained
ϕ	Internal friction angle
ϕ'/ϕ_{soil}	Effective friction angle of the soil
ψ	Dilatancy angle

LIST OF ABBREVIATIONS

ASTM	American Society of Testing Materials
FEM	Finite element method
GWL	Ground Water Level
SPT	Standard penetration test
TLT	Top loading static test

1. INTRODUCTION

1.1. Objectives

Pile analysis and design are dependant both on the pre and post piling soil conditions, as well as both on the superimposed load mechanisms and pile installation techniques, and also both on the pile-soil and ground-structure interaction; so it becomes a very complex task to deal with. Piles bear superposed loads by undergoing deformations resulting on two distinct load bearing mechanisms which are End (Tip/Point)-Bearing and Frictional Resistance shown in Figure 1.1.

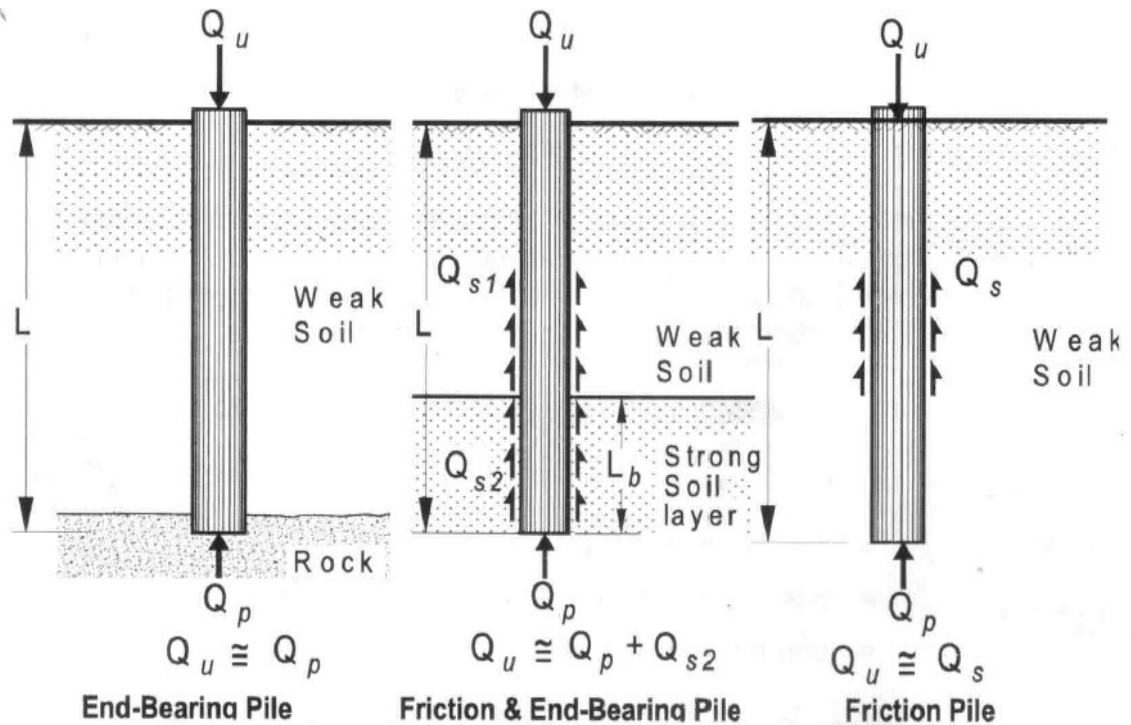


Figure 1.1. Categorization of piles on their load bearing mechanisms (Candoğan, 2008).

Pile load tests can afford an effective way to check on the uncertainties in soil parameter measurement and design assumptions that occurs in the design and construction of piles. Static load tests, statnamic and pseudo-static tests, Osterberg-cell tests, dynamic

tests and integrity tests are some of the test methods. Pile load tests provide the most essential information such that:

- The ultimate load capacity of a single pile
- The load transfer behavior of a pile
- The load-settlement behavior of a pile
- The structural integrity of a pile as constructed,

which are used as a means of verification of design assumptions and also obtaining design data on pile performance which may allow for a more effective and reliable design of the piles in a particular site.

As it is improbable to correctly simulate the real conditions that correlate to the actual installed state of piles in the field, we need to be very careful during the interpretation of pile test. In order to get reliable results, these are need to be considered:

- Whether the test load on the pile is applied the same manner as the structure will load the prototype piles
- Whether the test set-up induces inappropriate stress changes in the ground or cause inaccuracies in the measurements of settlement
- Whether other factors exist which may have other side-effects on the result

With the development of the numerical methods and the improvement of the performance of computers, the extent to which these tests can satisfy the above requirements of the designer can be extended by simulating the pile loading test in a numerical model and analyzing the results in combination with the field test data.

In this thesis, the finite element method (FEM) was used to carry out the research. This method has the advantage over traditional analysis techniques as more realistic test condition can be taken into account; as well as, displacements and stresses within the soil body and pile are coupled, thus more realistic pile-soil interaction behaviour can be represented with more realistic assumptions. The commercial finite element code PLAXIS was used for the numerical simulation of pile load tests that will be studied in the following.

1.2. Scope of Study

Due to the limitation of the time and length of the thesis, only some particular interest which is associated with the conventional static load test and Osterberg-cell test were studied. Four different static pile load tests are analyzed to study the difference between the field test data and the numerical models of them, as well as one O-Cell pile loading test is analyzed to study the difference between the static pile loading test results and O-Cell pile loading test results and the comparison of both of them with the field test data. Equivalency and discrepancy of the test results are demonstrated.

The overall project is divided into 7 major tasks as follows:

Task 1, Literature review: The set-up of static pile loading tests and Osterberg Cell loading tests are described. Besides, the principles of both of them are illustrated and numerical and practical aspects of them are highlighted.

Task 2, Finite element modeling in geotechnics is reviewed and a commercial finite element modeling code Plaxis is introduced in order to model the structure (pile) and the interaction between the structure and the soil.

Task 3, Case history of Static Loading Test in Osmangazi-Demirtaş Mevkii BURSA (Case1) is analyzed and the numerical results are compared with the reported field measurements. Equivalency and discrepancy of the results are demonstrated.

Task 4, Case history of Static Loading Test in Demirtaş BURSA (Case2) is analyzed and the numerical results are compared with the reported field measurements. Equivalency and discrepancy of the results are demonstrated.

Task 5, Case history of Osterberg Cell Loading Test in Ataşehir (Case3) is analyzed and the numerical results are compared with the static loading test data of the same test and also with the reported field measurements. Equivalency and discrepancy of the results are demonstrated.

Task 6, Case history of Static Loading Test in Bartın (Case4) is analyzed and the numerical results are compared with the reported field measurements. Equivalency and discrepancy of the results are demonstrated.

Task 7, Case history of Static Loading Test in Ataşehir (Case5) is analyzed and the numerical results are compared with the reported field measurements. Equivalency and discrepancy of the results are demonstrated.

2. LITERATURE REVIEW

2.1. Review of Pile Load Test

Full scale field tests are the most dependable way of measuring the load bearing and settlement behaviour of piles. Testing measures the response of a pile-soil system to loads in load testing or electronic impulses as in integrity testing and thus provide data for research and development, engineering design, quality assurance in accordance with the specifications and contract documents.

A number of forms of pile load test have been used in practice. Some methods such as static loading test and dynamic test have been a routine in geotechnical engineering for many years, while Osterberg cell test and static test have been developed for less than twenty years. This thesis concentrates on the static loading test and Osterberg cell test. The test procedures and results can be modeled by finite element analysis method, so that the actual soil-pile relationships of ultimate capacity, distribution between shaft resistance and end bearing, load settlement response of the particular characteristics assumed in the design can be re-analyzed and verified by the finite element model.

2.1.1. Static Loading Test

Static load test is the most basic test and involves the application of vertical load directly to the pile head. The objectives of this test is to determine the ultimate bearing capacity of the piles, to separate the total resistance contributed by the side shear and end bearing capacity, and to determine the stiffness of the soil and pile system at design load. A back analysis of the tests data will enable the soil modulus to be evaluated, and so the deformation of pile groups may be predicted with confidence. In addition to providing load vs settlement behaviour plus ultimate load bearing value of the pile, static load tests also provide substantiation of successful implementation of piles. There are compressive, tensile and lateral pile static loading tests. Two principal types of test are used for the most commonly applied of the three which is compressive loading on piles. The first of these is

constant rate of penetration (CRP) test, in which the pile head is pushed downward at a constant rate. The second type of test is maintained load (ML) test in which pile is exposed to the discrete increases of load over a series of intervals of time. Figure 2.1 , Figure 2.2 and Figure 2.3 are commonly used static load test systems.

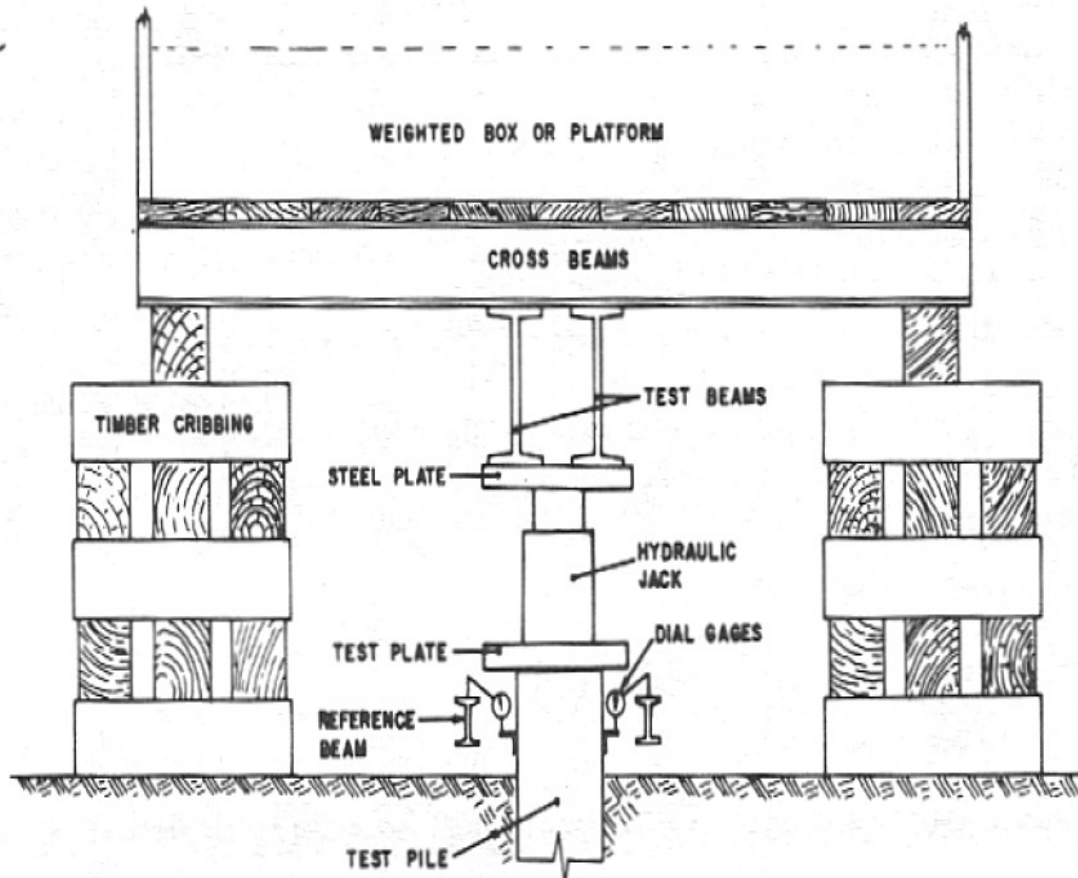


Figure 2.1. Static Pile Loading Test Using Kentledge (Yi, 2004).

The Constant Rate of Penetration method is essentially a test to determine the ultimate load on a pile and is therefore applied only to preliminary test piles or research-type investigations. The method has the advantage of speed in execution, less than one working day, and because there is no time for consolidation or creep settlement of the ground the load-settlement curve is easy to interpret. The CRP test is not suitable for checking the compliance with the specification requirements for the maximum settlement at given stages of loading.

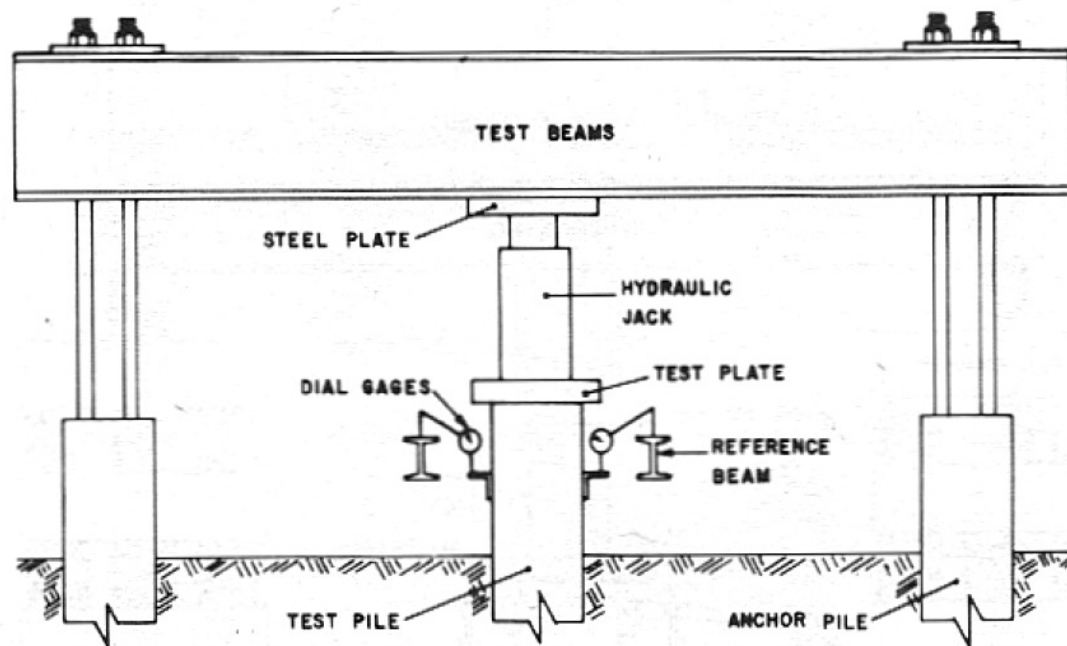


Figure 2.2. Static Pile Loading Test Using Anchored Reaction Piles (Yi, 2004).

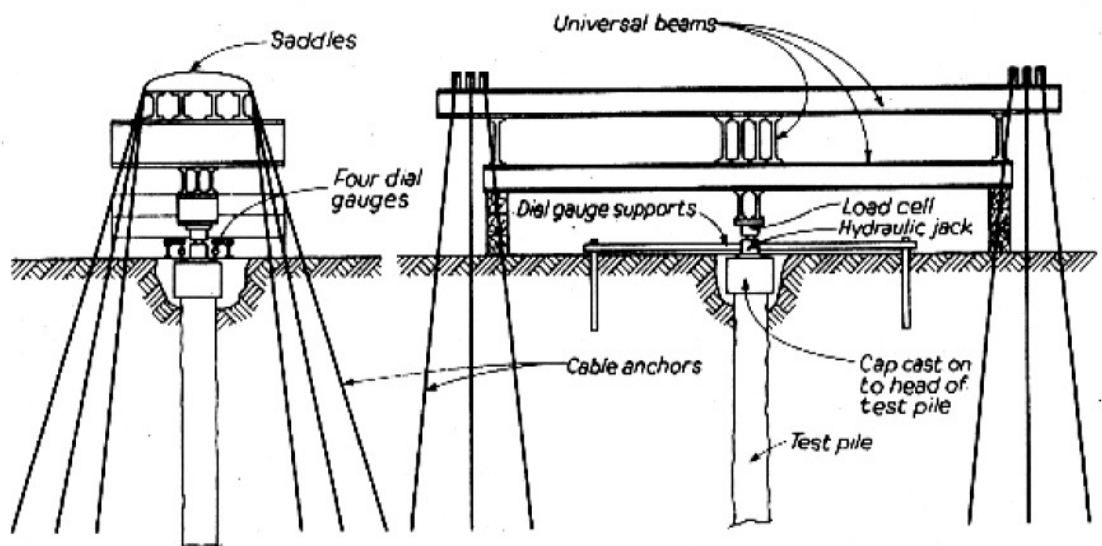


Figure 2.3. Static Pile Loading Test Using Ground Anchor (Yi, 2004).

The Maintained Load test is best suited for contract work, particularly for proof loading tests on working piles. It is also suitable for use where empirical methods are

employed to predict the ultimate load from measurement of residual deflections after returning the test load to zero at four or five stages up to the maximum. The test consists of loading the pile incrementally in stages and returning the load to zero after each increment of 100 percent, 150 percent and 200 percent. This test is very time consuming, requiring from 30 to 70 or even more hours to complete. This form of test is necessary if the net settlement curve is used as the basis of defining the failure load. It is essential to maintain a constant rate of load application. It has been found that the slower the rate the smaller is the ultimate failure load. It is a high possibility that the ultimate load might not be estimated by test results as maximum load increment of 200 percent of the working load on the pile is smaller than the failure load (Tomlinson, 2004).

Load-settlement curve is constructed simply by plotting the loads applied onto the pile head vs. the pile head displacement. A typical load-settlement curve for the CRP test and load-settlement curve for ML test are shown in Figure 2.4 and Figure 2.5. The static load test is generally regarded as the definitive test and the one against which other types of test are compared.

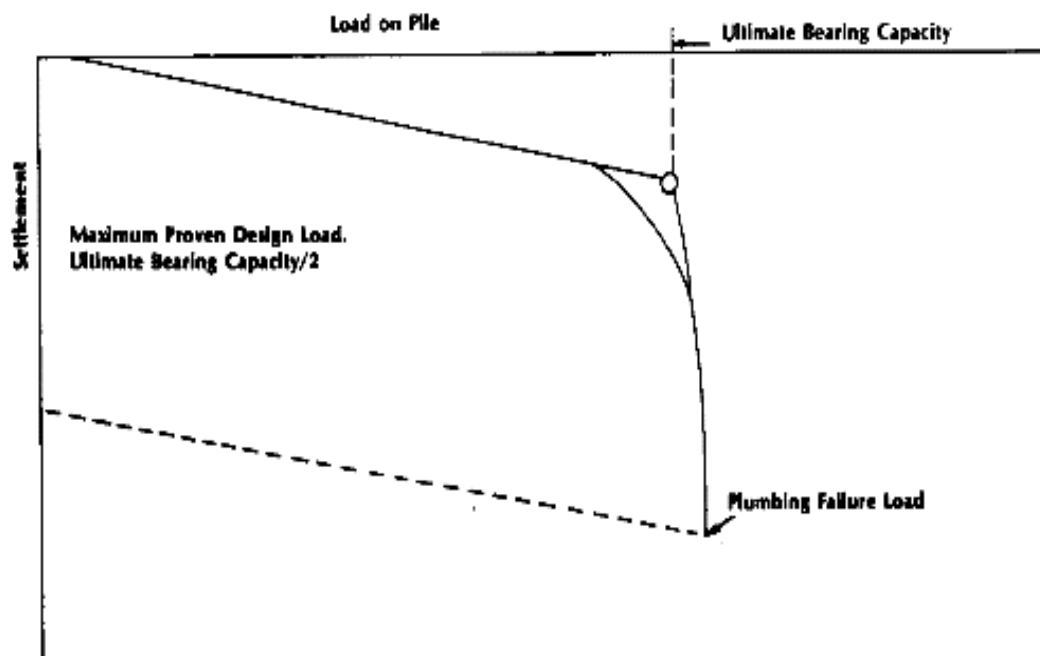


Figure 2.4. Typical load-settlement curve for CRP method (rdl.train.army.mil).

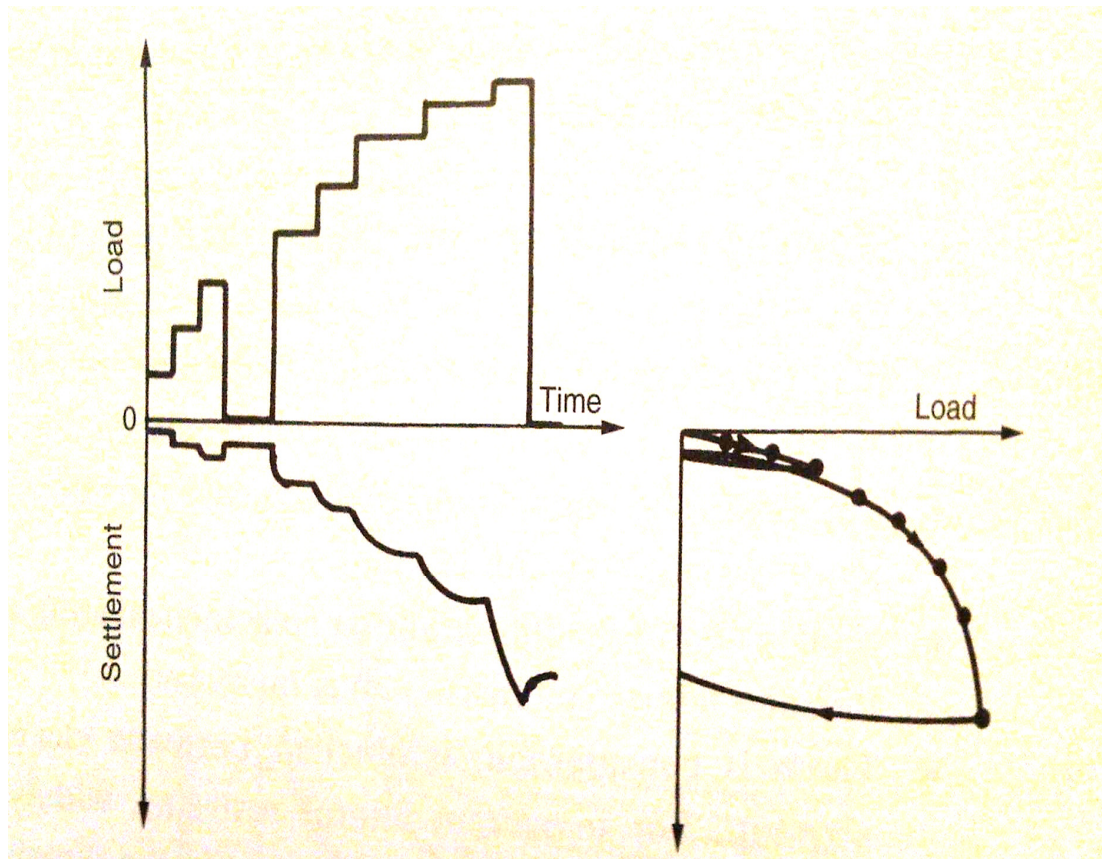


Figure 2.5. Typical load-settlement curve for maintained load test (Simons& Menzies, 2001).

2.1.1.1. Testing Equipment. The apparatus for applying compressive loads to the test pile should be constructed so that the loads are applied to the central longitudinal axis of the pile to minimize eccentric loading. The main equipments for the test are load reaction system see Figure 2.1, Figure 2.2 and Figure 2.3, test girder, reference beams, jacking system and so on.

2.1.1.2. Loading methods. A platform is constructed on the head of the shaft and a kentledge is placed on the platform, see Figure 2.1;

or, a bridge, carried on temporary supports, is constructed over the test shaft and loaded with enough dead weight. The ram of a hydraulic jack, placed on the shaft head,

bears on a cross-head beneath the bridge beams, so that a total reaction equal to the weight of the bridge and its load may be obtained;

or, reaction shafts capable of withstanding an upward force are constructed on each side of the test shaft, with a beam tied down to the head of the reaction shafts and spanning the test shaft. A hydraulic jack on the head of the test shaft applies the load and obtains a reaction against the underside of the beam, see Figure 2.2.;

or, ground or rock anchors are constructed to transfer the reaction to stiff strata below the level of the shaft base, see Figure 2.3. (Zhang, 2004).

Because the pile head is subjected to highest stress level, its concrete strength should be safely sufficient during the test. This often requires test pile head preparations with early strength concrete as soon as the test pile is chosen. Normally, a thick-walled steel pipe is encased around the pile head and a thick steel plate is welded on to the top so as to be perfectly level. For large diameter piles, still a thick pipe encasement is used but the pile head is completed with high and early strength concrete which must be finished smooth and level (Candoğan, 2008).

2.1.1.3. Measurement of Load. Calibrated pressure gauges are used in conventional load tests to determine the load applied to the test shaft by measuring the hydraulic pressure in the loading ram. However, calibrated pressure gauges may only provide acceptable accuracy of load measurement for increasing load because unloading cycle is usually nonlinear owing to friction within the hydraulic jack. Therefore, load measurement is preferably carried out using load cell (Fleming, 1992).

The load distribution along the length of a drilled shaft can be obtained using sister bars. A sister bar is a section of reinforcing steel at the middle of which is placed a strain transducer as shown Figure 2.6. The electrical output can be converted to strain in the steel rebar through an appropriate calibration factor, which then can be assumed to be equal to the strain in the concrete section. The internal load in the shaft can then be obtained by multiplying the axial stiffness of the test shaft by the strain obtained at the depth of interest.

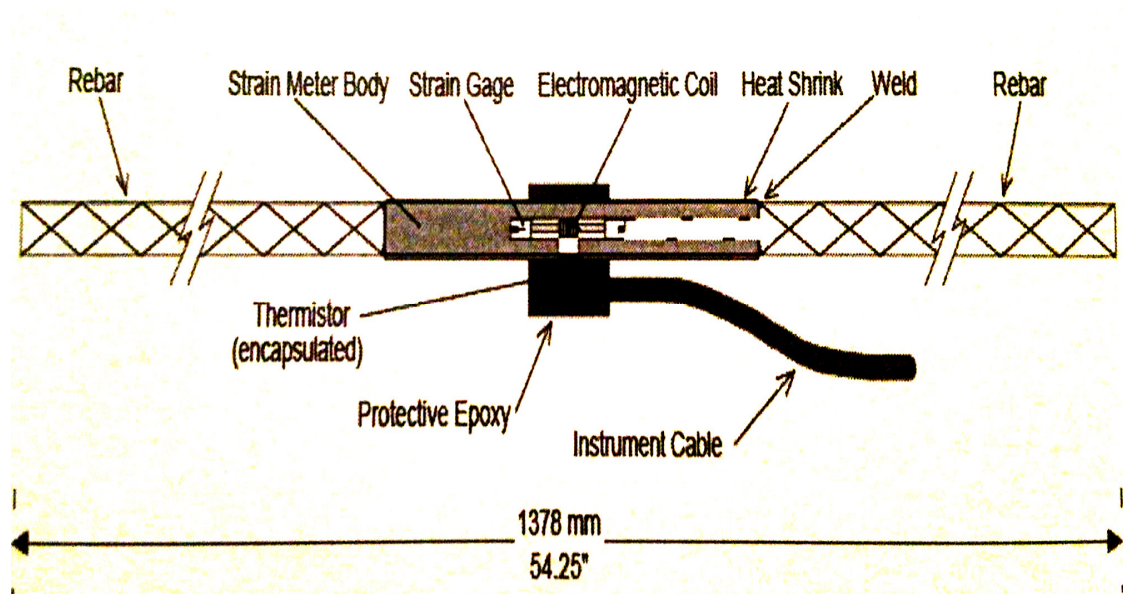


Figure 2.6. Rebar strain meter (Hayes and Simmonds, 2002).

Besides, the mustran cell is mounted on the rebar cage before inserting it into the borehole, in a manner similar to that for sister bars. The mustran cell indicates strains that are larger than, but proportional to, the actual strains in the concrete. This feature is advantageous for testing large diameter drilled shafts subjected to small loads. Data from mustran cells are collected and interpreted in a manner similar to that for sister bars.

On the other hand, telltales are unstrained metal rods that are inserted into one or more tubes that prevent them from bonding to the concrete in the shaft. By using displacement transducers, the difference in the movement of the shaft head and the top of the unstrained rod is measured. For a particular applied load, the average deformation measured at each depth can be used to plot a compression of the shaft vs depth curve. Differentiation of this curve with respect to depth will yield strain of the shaft as a function of depth. By multiplying the axial stiffness of the shaft by the strain obtained at the depth of interest, the internal load can be obtained.

Lastly, pressure cells placed at the bottom of the cage can be used to measure the base resistance directly. The base resistance can be simply obtained by multiplying the average pressure from the cells with the contact area of the shaft base (Arda, 2009).

2.1.1.4. Measurement of Displacement. In order to measure the head displacement of the test shaft, dial gauges or electronic displacement transducers such as linear variable differential transformers, which are called LVDT, that are held by stable reference beams, are used mostly. It is noted that a compromise must frequently be made between a long beam prone to vibration and temperature induced displacements, and a shorter beam with support foundations in the zone of influence of the test shaft or reaction shafts.

Besides, optical leveling may also be used to measure the head displacement of the test shaft. When tests are run to obtain information on load transfer in side shear and end bearing resistance, telltales can be used to measure the displacement of the test shaft from point to point along its length (Fleming, 1992).

2.1.1.5. Interpretation of test data. Static pile load tests' main data are loads, deformations and time. Loads and deformations are recorded at least twice at each loading stage. The results are presented in load vs. settlement or time, and deformation vs. time diagrams. The most common load vs. settlement diagrams give us information on the elastic and plastic settlements, and indications on reaction and end-bearing capacities (Tomlinson, 2004).

With experience the load-settlement curve from a compression test can be used to interpret the mode of failure of a pile. A defective pile shaft is also indicated by the shape of the curve. Some typical load-settlement curves and their interpretations are shown in Figure 2.7.

According to Figure 2.7:

- Friction pile in soft-firm clay or loose sand (a)
- Friction pile in stiff clay (b)
- Pile end bearing on weak porous rock (c)
- Pile lifted off setting on hard rock due to soil heave and pushed down by test load to new bearing on rock (d)
- Gap in pile shaft closed up by test load (e)
- Weak concrete in pile shaft sheared completely through by test load (f) (Tomlinson, 2004).

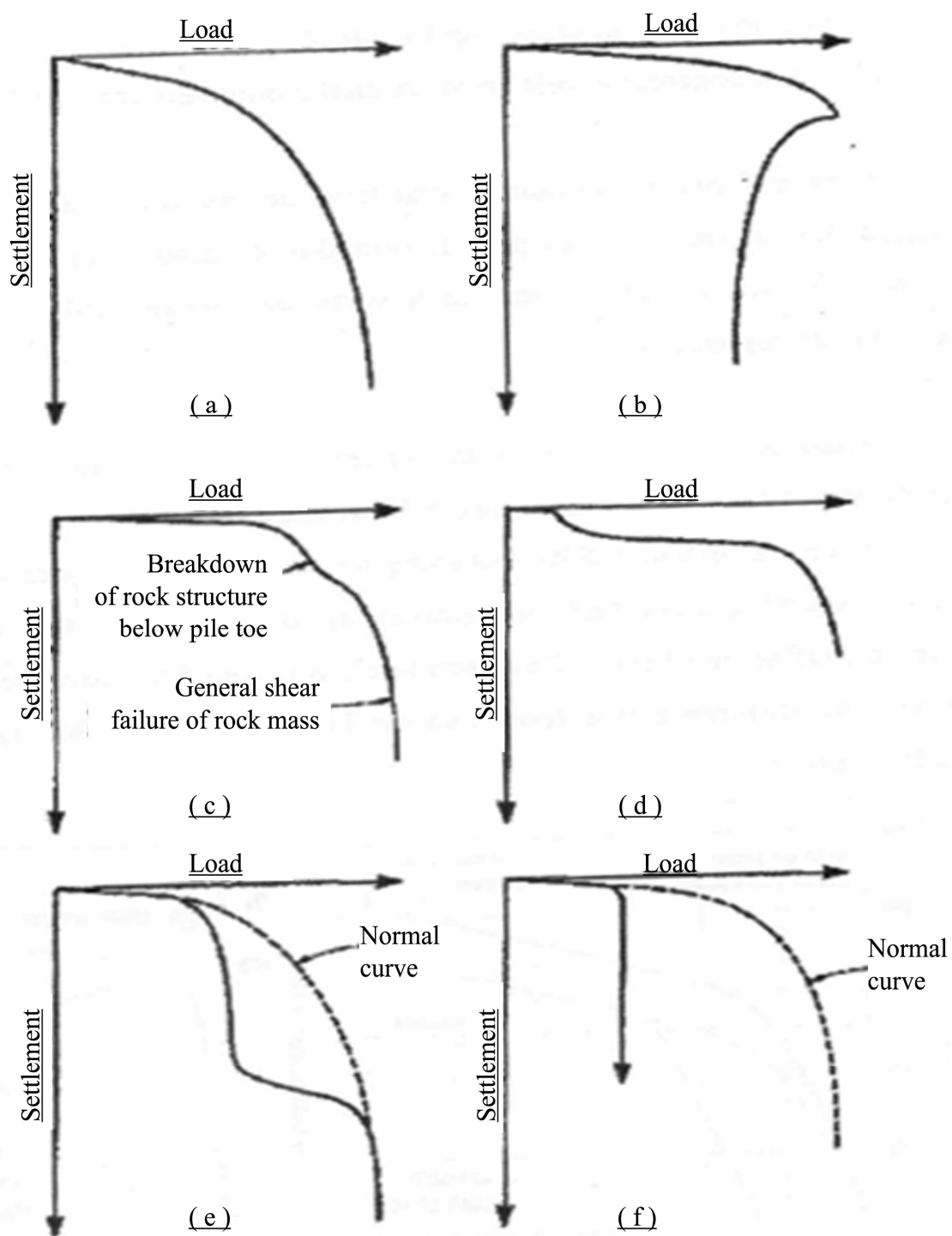


Figure 2.7. Typical load-settlement curves for compressive load tests (Tomlinson, 2004).

If the deformations under a load level do not stabilize, the pile is deemed to have reached the failure load. As pile deformation continues for some time to establish full failure, it is hard to clearly distinguish elastic and plastic settlements of piles failed during the test (Candoğan, 2008).

2.1.2. Osterberg Cell Loading Test

The Osterberg Cell (O-Cell) method was developed by Osterberg (1989). The O-Cell is a sacrificial jack like device installed within the foundation unit, see Figure 2.8.

The O-Cell is specifically engineered to offer negligible internal friction even with eccentric movement. The depth of the O-Cell within the pile is set according to the test program requirements. A typical level for the O-Cell can be determined where there will be equal capacity above and below to maximize the load that can be mobilized in the pile during the test.

The O-Cell is attached to the reinforcing steel cage or other support structure to ensure its location and depth is located precisely.

After the concrete reaches a minimum strength, the test may be started. Separation of the pile into two elements is induced by hydraulic pressure applied at the cell.

As the load is applied to the O-Cell, it begins working in two directions; upward against upper skin friction and downward against base resistance and lower frictional capacity (if applicable), the O-Cell test requires no reaction beam or anchor piles.

The test is considered complete after reaching ultimate capacity above or below the O-Cell or upon reaching the maximum capacity of the O-Cell.

Instrumentation is included in each test pile for direct measurement of O-Cell expansion, shaft compression and top-of-shaft movements. The test pile often includes monitoring strain gauges embedded in the shaft. The O-cell load versus the upward

movement of the O-cell top is the load-movement curve of the pile shaft. The O-cell load versus the downward movement of the O-cell base is the load-movement curve of the pile

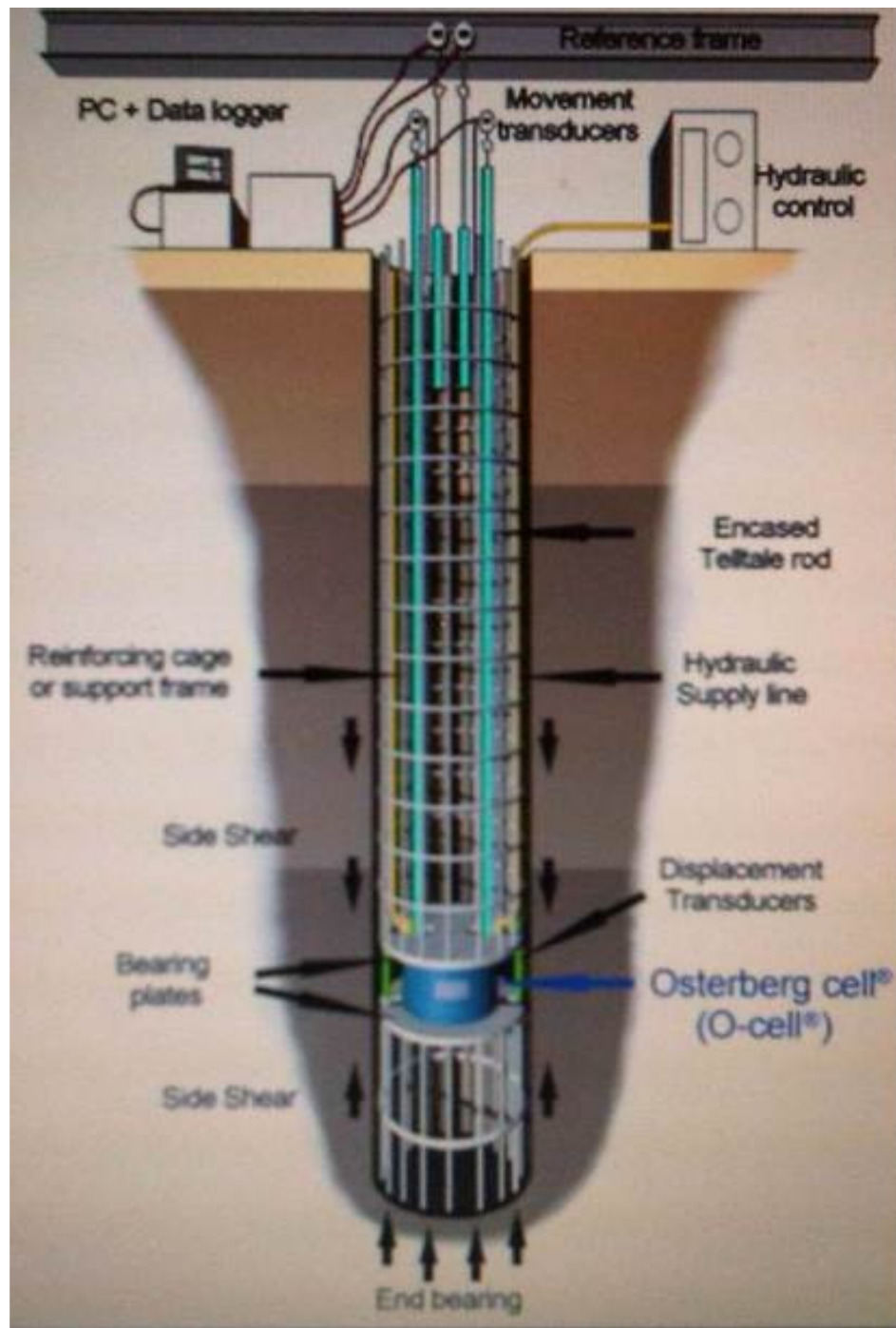


Figure 2.8. The Osterberg Cell Test Set-up (Loadtest, 2010).

toe. This separate information on the load-movement behaviors of the shaft and toe is not obtainable in a conventional static loading test (Loadtest, 2010).

2.1.2.1. Testing Equipment. For a proper testing, Osterberg Cell, with a desired capacity in each direction and its equipment of hydraulic lines, fittings, pressure source, pressure gage, telltale devices and vibrating wire-type strain-gauges are needed. In addition to them, welding equipment, air compressor for pump operation during load testing, enough labor, electric power and materials sufficient to construct a stable reference beam system for monitoring movements of the shaft during testing are needed for the test.

Telltale extensometers with linear vibrating wire displacement transducers connected to a computerized data acquisition system are used for measurement of the movement of the bearing plates. The lower ends of the transducers attached to the bottom plate of the O-Cell measure the total opening, or extension of the O-Cell and the steel telltales which extend to the top of the O-Cell as shown in Figure 2.9 as “C” and “D”, measure the upward movement of the top of the O-Cell and the compression of the test pile. Subtracting the upward movement of the top of the O-Cell from the total extension of the O-Cell provides the downward movement of the pile.

A high pressure bourdon gauge and calibrated electronic pressure transducer is used to monitor the hydraulic cell pressure, from which the loading is derived by applying the calibration factor of the hydraulic cell to the pressure. The upward movement of the test pile is measured with digital gages mounted on a reference beam and set over the top of the test pile as shown as “A” and “B” in Figure 2.9. (Schmertmann and Hayes, 1997).

2.1.2.2. Installation. Pile excavation will proceed under the piling contractor’s work plan as approved by the engineer. Upon reaching the final toe elevation, the pile bottom will be cleaned and approved by the engineer for concrete placement. The O-cell assembly, bearing plates, related hydraulic supply, and instrumentation will be lowered into the excavation attached to the steel reinforcement cage. For the proposed test pile, the steel cages may be fabricated in a number of cage sections, the lengths being agreed with the piling contractor. The lifting of each cage to vertical will be carried out using a lifting beam or several pick points to ensure the cage remains reasonably straight during lifting.

After the entire reinforcing cage has been lowered into the excavation, the cage will be supported at the correct level during concrete placement

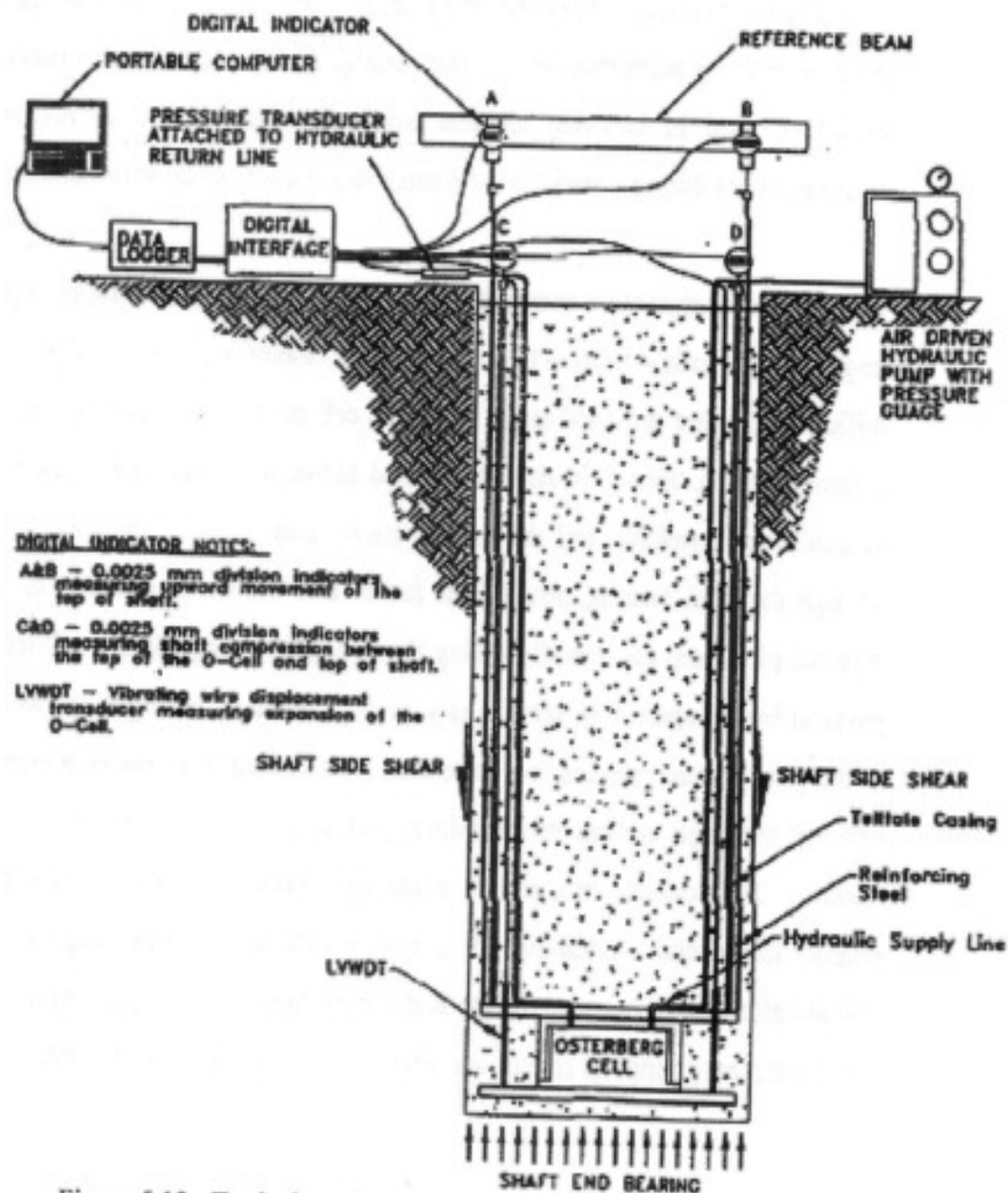


Figure 2.9. Typical Test Instrumentation (Shmertmann and Hayes, 1997).

Figure 2.10 and Figure 2.11 show the installation sequence of a typical pile. All reinforcing cages will be fabricated with sufficient cage stiffeners or welded mild steel hoops at regular spacing, lifting points and lifting beams as required for preferred lifting

technique. The fabrication of the O-cell assembly will be carried out on site or in the cage fabrication yard if available. The O-cells will be accurately welded into place between the top and bottom bearing plates. The completed assembly will then be welded to the reinforcement cage, perpendicular to the cage axis at the given location. Three or more pieces of reinforcing steel or steel angle iron will be welded between the top and bottom O-cell bearing plates; as a temporary measure prior to the lifting process. These temporary supports will be severed when the cage is being lowered into the excavation. This procedure will ensure that there is no steel reinforcement extending through the pile separation plane at the O-cell level.

Concrete placement will commence utilizing a slick line tremie pipe as shown on Figure 2.11 (with no joints below O-cell level) of sufficient length so as to extend beyond the O-cell assembly to the toe of the pile. Cut-outs of sufficient size will be provided in the O-cell steel bearing plates to accommodate the nominal diameter of the tremie pipe. A “funnel” will be constructed between the opening in the top plate of the O-cell assembly and the main vertical rebar to help guide the tremie pipe through the cut-outs in the O-cell assembly. The funnel also serves as a means of preventing the tremie pipe from accidentally hitting the hydraulic fittings on the cell top by forming a physical barrier apart from serving as a guide. The approved concrete mix must allow for workability considerations and should contain sufficient retarding agent to maintain workability. The concrete will be placed up to the designed cut off level as per standard/approved concreting procedures. A cage with nominal reinforcement will be extended beyond the concrete cut off level to 300-500 mm above top of permanent casing level, where appropriate, to allow for instrumentation monitoring. This cage will be reinforced with additional stiffeners as required to ensure it remains sufficiently rigid above the concrete cut off level. Telltales, instrumentation wiring, and hydraulic supply lines extending beyond the concrete cut off level will be supported on this cage.

2.1.2.3. Testing Procedures. After the concrete has reached a minimum required strength, or as required per the project specifications, testing can be initiated. The O-cell tests are generally performed in accordance with the ASTM D1143D, 2007 specification.

Concrete curing time of ten days would usually provide enough concrete strength to

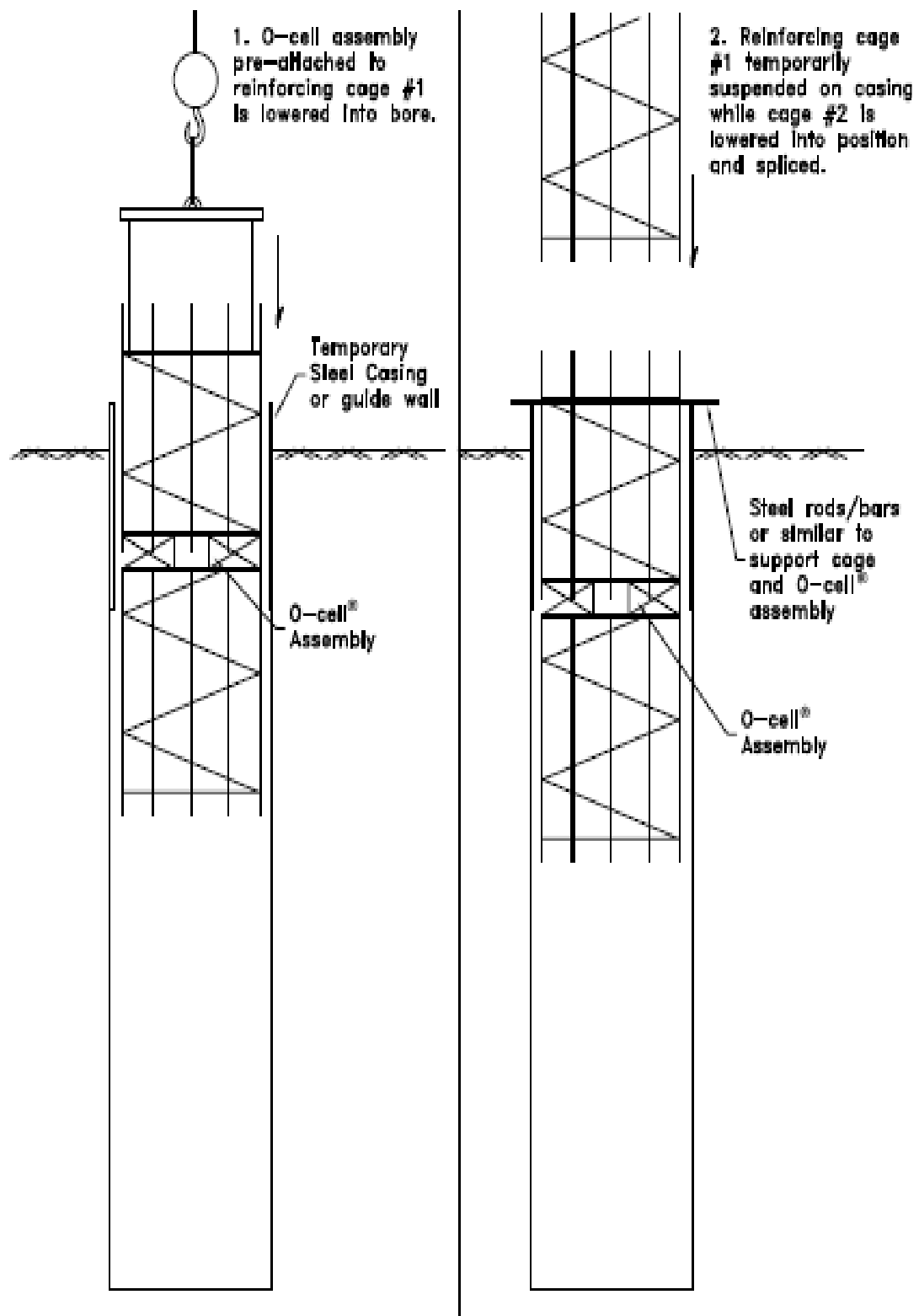


Figure 2.10. Typical pile installation sequence 1 (Loadtest, 2010).

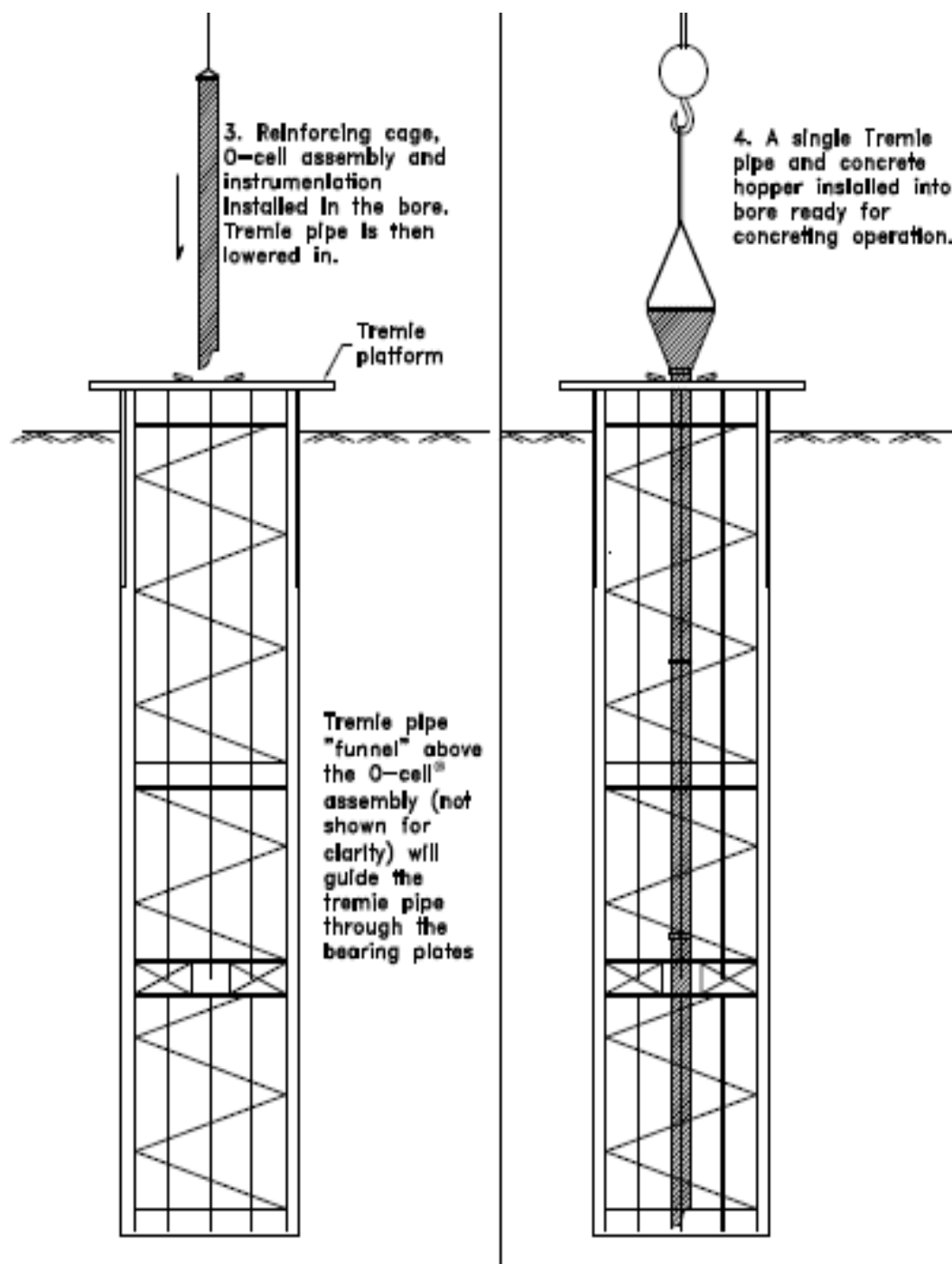


Figure 2.11. Typical pile installation sequence 2 (Loadtest, 2010).

conduct the pile testing. Nonetheless the strength of concrete shall be determined from cube samples. There shall be at least 3 cubes taken for each test pile, and these are to be tested 7, 10 and 21 days after sampling. Apart from the concrete reaching the minimum

required structural strength, a minimum of 21 days is recommended between the installation and testing to allow the soil-concrete interface to recover and provide favourable results.

The O-cell(s) will be internally pressurized, creating an upward force on the pile in upper skin friction and an equal, but downward force in combined lower skin friction and/or end bearing. The O-cell load is determined by relating the applied hydraulic pressure to the O-cell load calibration curves. A calibrated, high-pressure bourdon gauge will be used to display the pressure on the pump line and a calibrated high resolution pressure transducer will read the pressure on the return line (used as the reference). Pressurizing the O-cells simultaneously loads the pile below the O-cells in downward end bearing and/or skin friction which resists downward movement and loads the pile above the O-cells in upward skin friction which resists upward movement (Osterberg, 1998).

The loading schedule describes a single loading cycle up to test load or up to the maximum O-cell capacity in steps. Each load will be held for a minimum of 30 minutes or until the prescribed settlement rate is achieved, subject to a maximum duration of 3 hours. The required settlement rate to be met will be 0.1mm per 20 minute interval. Settlement rate will be measured as the combined displacement of upward and downward movements measured directly by the expansion transducers monitoring the O-cell expansion or by the difference between top and bottom plate measured using telltale rods. This is the most sensitive location for the measurements and the most immune to external influences. The unloading will be performed in equal load steps from the maximum load applied and held for approximately 10 minutes. The recovery at zero load will be monitored for a 1 hour period.

The load will be removed and testing finished once either one of the following situations occur:

- A predetermined maximum test load has been applied;
- The maximum travel of the O-cell is reached (150 mm); or limited upward movement to prevent exceeding the upward skin friction.
- Ultimate capacity of the pile above or below the O-cell location has been reached.

2.1.2.4. Measurements. The expansion of the O-cell assembly is directly measured by using Linear Vibrating Wire Displacement Transducers (LVWDTs). The LVWDTs are fixed between the top and bottom bearing plates. Measurements will be recorded to the nearest 0.01 mm division. The upward vertical movement of the top of the pile is measured using two (2) LVDTs or attached to a reference beam set to measure traditional telltale rods set in the top of the pile. Details of the reference beam arrangement can be found in Figure 2.9. These LVDTs or LVWDTs are read to the nearest 0.01mm division. The stems of the indicators are plumbed to ensure accurate vertical measurement. Pile compression under load will be measured using a minimum of two (2) LVDTs or LVWDTs attached to the top of the pile and set over traditional telltale rods extending from top of O-cell to top of pile. These LVDTs or LVWDTs are read to the nearest 0.01mm division. The stems of the indicators are plumbed to ensure accurate vertical measurement.

Pile toe displacement under load will be measured using LVDTs or LVWDTs attached at the top of the pile and set over the traditional telltale rod casing extending from pile toe to top of pile or directly by embedded compression telltales. Deducting measured compression from the downward O-cell movement will provide for actual pile toe movement. Unit shaft friction parameters are assessed along the pile shaft using strain gauges.

The measured movements provide for the following:

- Pile Compression (measured by traditional telltale rods set from top of O-cell and referenced to top of pile);
- Top of Pile Movement (measured by LVDTs attached to the reference beam set over the pile reinforcement);
- Upward Top of O-cell Movement (calculated as measured Pile Compression plus Top of Pile movement);
- O-cell Expansion (measured by LVWDTs fixed between the top and bottom bearing plates);
- Downward Bottom of O-cell movement (calculated as measured O-cell Expansion minus measured Pile Compression plus measured Top of Pile movement);

- Downward Pile toe Movement (measured by traditional telltale rods referenced to top of pile);
- Force distribution within the pile shaft (derived from the direct measurement of strain by vibrating wire strain gauges and calculated pile stiffness values). Note, upwards direction load generates shaft friction only whereas downward direction load generates shaft friction and end bearing, which can be isolated using the strain gauge measurements.

Where a conventional reference beam is used, with its supports located at over 3 pile diameters from the test pile, an automated digital survey level (Leica NA2002, NA3000 or NA3003) is used to monitor the reference beam for any movement during testing. Any top of pile movements with respect to the reference beam will be assessed and corrected for variation in vertical position of the reference system.

All the LVDTs, LVWDTs and VWSGs are connected directly to a Data Electronics Geologger 515 or 615 data logger. The logger is, in turn, connected to a laptop computer. This arrangement allows LVDTs/LVWDTs vibrating wire readings to be recorded simultaneously and stored automatically at 30 second intervals during the test (depending on the number of instruments being read). The use of a data logger allows for greater than manual readout box methods. All readings are rounded up or down to the nearest 0.01mm for reporting. In addition the loggers automatically record the of the pile concrete and external ambient temperature during testing (Loadtest, 2010).

2.1.2.5. Interpretation of Test Data. Osterberg Cell test provides load deflection curves for side shear and end bearing. Therefore, it is possible to determine at each increment of load how much of the total load is in side friction and how much is in end bearing. As the deflection of the shaft is small when the ultimate load is reached in side shear compared to the deflection at which the ultimate end bearing is reached, the majority of the load is taken in shear as the load is applied and shifts to end bearing as the load is increased. Besides, the ultimate side shear is larger than design values in most tests and if the side shear is low, it is usually due to construction methods. Therefore, the working load applied to the top of

the pile is mostly resisted by the side shear and little would reach the bottom, which can be used as a more economical design for a bored pile (Osterberg, 1994).

- Construction of the Top Down Equivalent Curve: Osterberg static load test can provide a good estimate of a curve showing the load versus settlement of a top-loaded bored pile with the following assumptions (Osterberg 1994):
- The end load-movement curve in a top loaded shaft has the same loads for a given movement as the net (subtract buoyant unit weight of pile above hydraulic jack) end bearing load-movement curve developed by the bottom of the hydraulic jack when placed at or near the bottom of the shaft.
- The side shear load-movement curve in a top loaded shaft has the same net shear, multiplied by an adjustment factor F for a given downward movement as occurred in the Osterberg test for that same movement at the top of the jack in the upward direction. The same applies to the upward movement in a top-loaded tension test. Unless noted otherwise, a factor $F=0.95$ for compression in cohesionless soils and $F=0.80$ for tension tests in all soils is used.
- The pile behaves as a rigid body, but include the elastic compressions that are part of the movement data obtained from an Osterberg static load test. Procedure 1 interprets an equivalent top-load test movement curve and procedure 2 corrects the effects of the additional elastic compressions in a top loaded test.
- The part of the shaft below the hydraulic jack (one or multi level) has the same load-movement behaviour as when top-loading the entire shaft. The subsequent end bearing movement curve refers to the movement of the entire length of shaft below the jack.

For an Osterberg cell load test, the side resistance versus side movement curve and the base resistance versus base movement curve can be used to construct the load-displacement curve equivalent to applying the load at the top of the shaft (Osterberg, 1999). This is done by determining the side resistance at an arbitrary displacement point on

the side resistance versus side movement curve. If the shaft is assumed rigid, the top and bottom of the shaft move the same amount and have the same displacement but different loads. By adding the side resistance and the base resistance at the same displacement, a single point on the top equivalent load-displacement curve is obtained. By repeating this process for different displacement points, the top equivalent load displacement curve can be obtained. As shown in Figure 2.12, it is very often that the side shear or the end bearing reaches ultimate before the other. In this case, two procedures are possible. The first, which is extremely conservative, is to assume that the component that has not reached ultimate has also reached ultimate and that no further load increase occurs as displacement increases. The other more likely procedure, as shown in Figure 2.12, is to extrapolate the curve that has not reached ultimate. Suitable hyperbolic curve fitting technique can be used for extrapolation of the side shear curve to produce end bearing movement data. Using the same movement matching procedure, the equivalent curve can also be extended. The dashed line on Figure 2.12 signify that this part of the equivalent curve depends partly on extrapolated data (Osterberg, 1998).

The elastic compression in the top load static test always exceeds that in the Osterberg test. It produces more top movement and also additional side shear movement, which then generate more side shear, more compression, so on. An exact solution of this load transfer problem requires knowing the side shear versus vertical movement (t - y) curves for a large number of pile length increments and solving the resulting set of simultaneous equations or using finite element or finite difference simulations to obtain an approximate solution for these equations (Osterberg, 1998).

The attached analysis in Figure 2.13, Figure 2.14, Figure 2.15 gives the equations for the elastic compressions that occur in the Osterberg load test (OLT) with one or two levels of hydraulic jacks. Analysis in Figure 2.16, Figure 2.17 gives the equations for the elastic compressions that occur in the equivalent top loading static test (TLT). Both sets of equations do not include the elastic compression below the hydraulic jack because the same compression takes place in both the OLT and the TLT. This is equivalent to taking $I_3 = 0$. Subtracting the OLT from the TLT compression gives the desired additional elastic compression at the top of the TLT. The additional elastic compression is then added to the rigid equivalent curve obtained from the equivalent top loaded settlement curve to obtain

the final, corrected equivalent load-settlement curve for the TLT on the same pile as the actual OLT.

Figure A

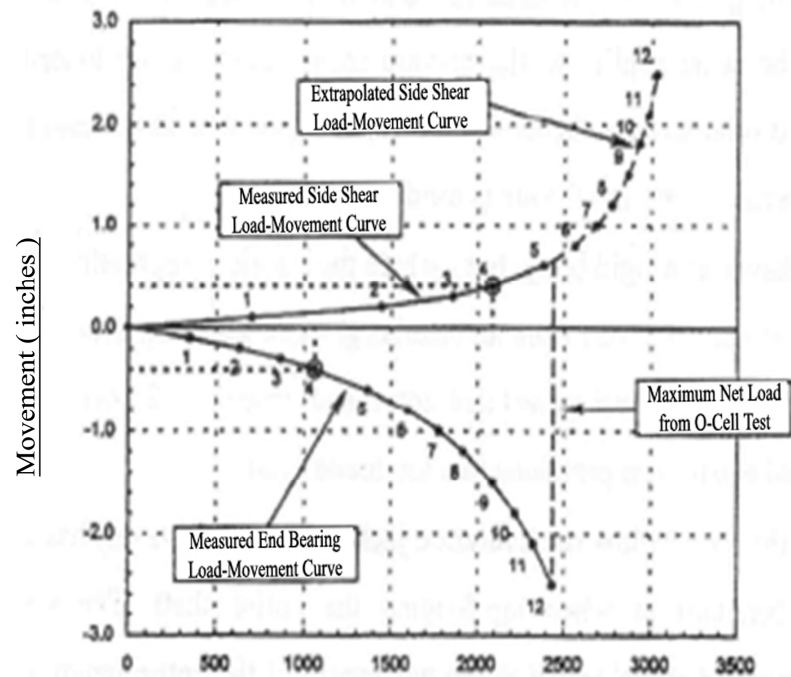


Figure B

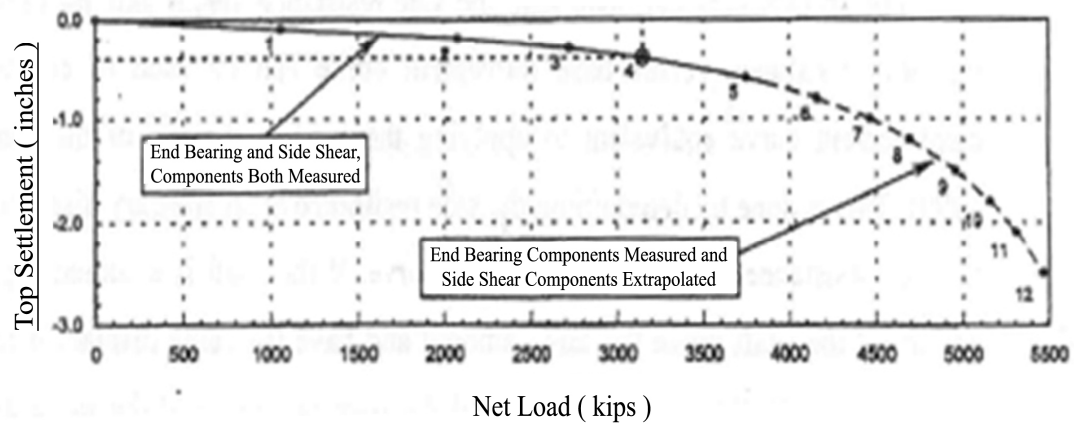


Figure 2.12. Example of the construction of an equivalent top-loaded settlement curve (Figure B) from Osterberg cell test results (Figure A) (Osterberg, 1998).

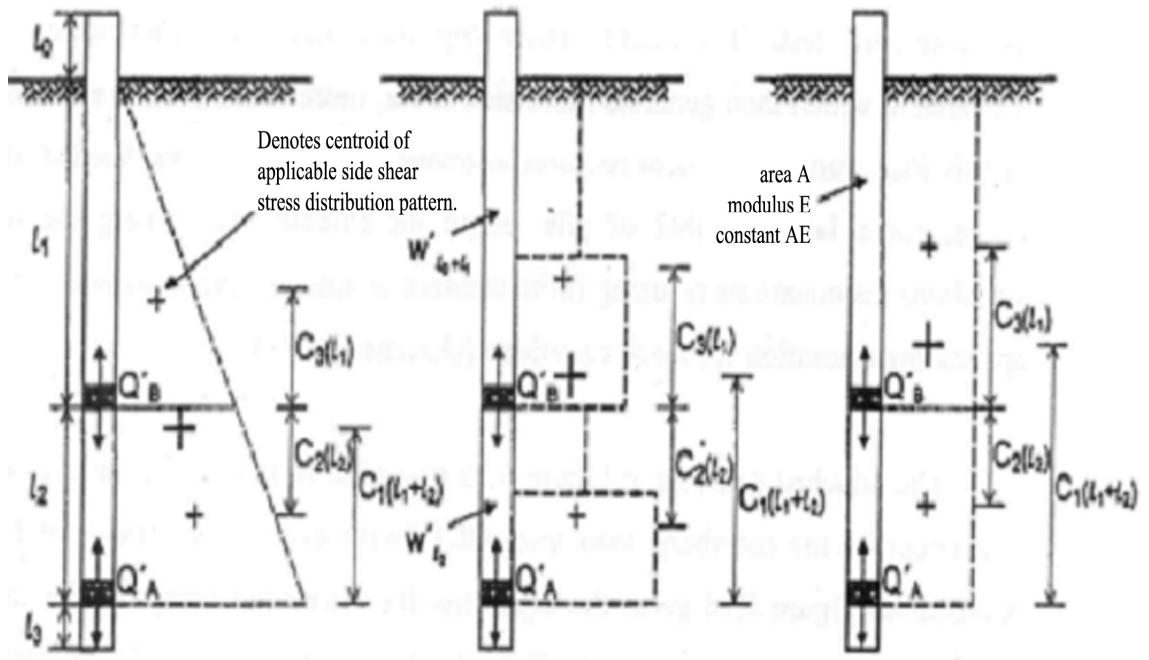


Figure 2.13. Theoretical elastic compression in O-Cell test based on pattern of developed side shear stress (Osterberg, 1998).

One Stage Single Level Test (Q'_A only): $\delta_{OLT} = \delta_{\uparrow(l_1+l_2)}$

$C_1 = \frac{1}{3}$	Centroid Factor = C_1	$C_1 = \frac{1}{2}$
$\delta_{\uparrow(l_1+l_2)} = \frac{1}{3} \frac{Q'_{\uparrow A}(l_1+l_2)}{AE}$	$\delta_{\uparrow(l_1+l_2)} = C_1 \frac{Q'_{\uparrow A}(l_1+l_2)}{AE}$	$\delta_{\uparrow(l_1+l_2)} = \frac{1}{2} \frac{Q'_{\uparrow A}(l_1+l_2)}{AE}$

Figure 2.14. Theoretical elastic compression in O-Cell test based on pattern of developed side shear stress (Osterberg, 1998).

Three Stage Multi Level Test (Q'_A and Q'_B): $\delta_{OLT} = \delta_{\uparrow l_1} + \delta_{\uparrow l_2}$

Net Loads: $Q'_{\uparrow A} = Q'_A - W'_{l_0+l_1+l_2}$ $Q'_{\uparrow B} = Q'_B - W'_{l_0+l_1}$ $Q'_{\downarrow B} = Q'_B + W'_{l_2}$

W = pile weight, buoyant where below water table

$C_3 = \frac{1}{3}$	Centroid Factor = C_3	$C_3 = \frac{1}{2}$
$\delta_{\tau_4} = \frac{1}{3} \frac{Q'_{\tau_4} L_1}{AE}$	$\delta_{\tau_4} = C_3 \frac{Q'_{\tau_4} L_1}{AE}$	$\delta_{\tau_4} = \frac{1}{3} \frac{Q'_{\tau_4} L_1}{AE}$
$C_2 = \frac{1}{3} \left(\frac{3L_1 + 2L_2}{2L_1 + L_2} \right)$	Centroid Factor = C_2	$C_2 = \frac{1}{2}$
$\delta_{\tau_2} = \frac{1}{3} \left(\frac{3L_1 + 2L_2}{2L_1 + L_2} \right) \frac{Q'_{\tau_2} L_2}{AE}$	$\delta_{\tau_2} = C_2 \frac{Q'_{\tau_2} L_2}{AE}$	$\delta_{\tau_2} = \frac{1}{2} \frac{Q'_{\tau_2} L_2}{AE}$

Figure 2.15. Theoretical elastic compression in O-Cell test based on pattern of developed side shear stress (Osterberg, 1998).

The analysis in Figure 2.18 provides an example of calculated results on a hypothetical one stage, single level OLT using the method described above with the centroid of the side shear distribution 44,1 per cent above the base of the hydraulic jack. The analysis in Figure 2.19 provides an example of calculated results on a hypothetical three stage, multi level OLT using the simplified method described above with the centroid of the combined upper and middle side shear distribution 44,1 per cent above the base of the bottom hydraulic jack. The individual centroids of the upper and middle side shear distribution lie 39,6 per cent and 57,9 per cent above and below the middle hydraulic jack, respectively. Figures also include comparison of the corrected with the rigid curve (Osterberg, 1998).

Top Loaded Test: $\delta_{TLT} = \delta_{\downarrow l_0} + \delta_{\downarrow l_1 + l_2}$

Net and Equivalent Loads:

$$Q'_{\downarrow A} = Q'_{\downarrow A} - W'_{l_0 + l_1 + l_2} \quad P_{\text{single}} = Q'_{\downarrow A} + Q'_{\uparrow A} \quad P_{\text{multi}} = Q'_{\downarrow A} + Q'_{\uparrow B} + Q'_{\downarrow B}$$

Component loads Q selected at the same $(\pm)\Delta_{OLT}$

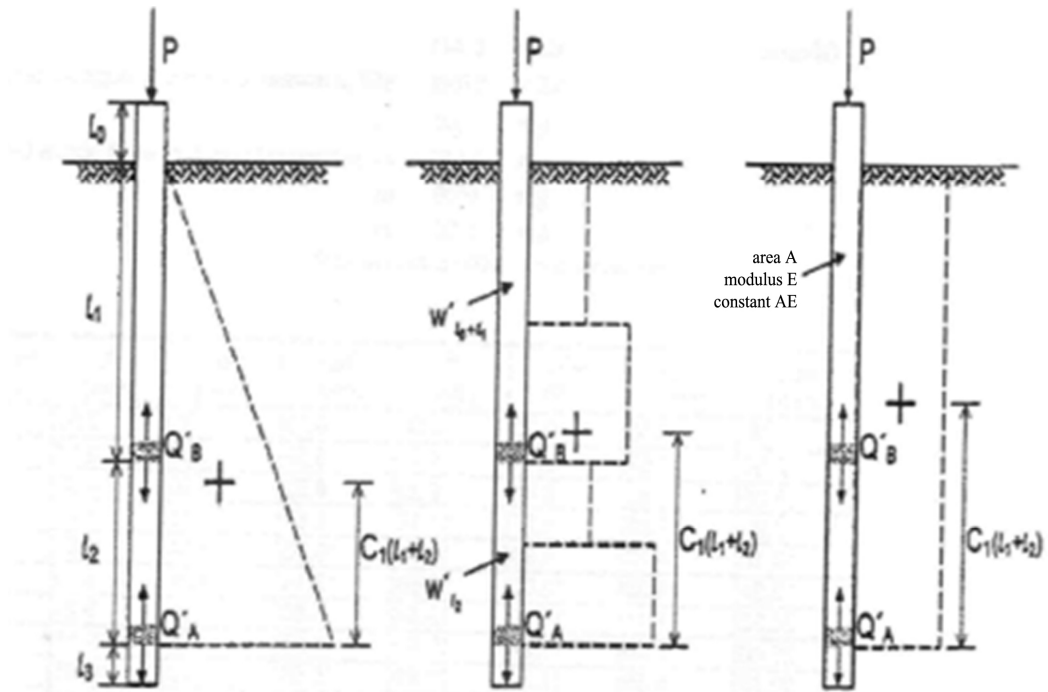


Figure 2.16. Theoretical elastic compression in top loaded test based on pattern of developed side shear stress (Osterberg, 1998).

$\delta_{l_0} = \frac{Pl_0}{AE}$	$\delta_{l_0} = \frac{Pl_0}{AE}$	$\delta_{l_0} = \frac{Pl_0}{AE}$
$C_1 = \frac{1}{3}$	Centroid Factor = C_1	$C_1 = \frac{1}{2}$
$\delta_{l_1+l_2} = \frac{(Q'_{A} + 2P)(l_2 + l_2)}{3 \cdot AE}$	$\delta_{l_1+l_2} = \frac{[C_1 Q'_{A} + (1 - C_1)P](l_1 + l_2)}{AE}$	$\delta_{l_1+l_2} = \frac{(Q'_{A} + P)(l_1 + l_2)}{2 \cdot AE}$

Figure 2.17. Theoretical elastic compression in top loaded test based on pattern of developed side shear stress (Osterberg, 1998).

Given:

$$C_1 = 0.441$$

$$AE = 17000 \text{ MN (assumed constant throughout test)}$$

$$l_0 = 1.80 \text{ m}$$

$$l_1 = 14.69 \text{ m (embedded length of shaft above O-Cell)}$$

$$l_2 = 0.00 \text{ m}$$

$$l_3 = 0.00 \text{ m}$$

Shear reduction factor = 1.00 (cohesive soil)

Table 2.1. Example calculation for the additional elastic compression correction for single level test (Osterberg, 1998).

Δ_{OLT} (mm)	$Q_{\downarrow A}$ (MN)	$Q_{\uparrow A}$ (MN)	P (MN)	δ_{TLT} (mm)	δ_{OLT} (mm)	$\Delta\delta$ (mm)	$\Delta_{OLT} + \Delta\delta$ (mm)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.54	1.57	3.14	4.71	3.37	1.20	2.17	4.71
5.08	2.82	6.43	9.25	6.52	2.45	4.07	9.15
7.62	3.86	8.27	12.12	8.61	3.15	5.46	13.08
10.16	4.72	9.29	14.01	10.05	3.54	6.51	16.67
15.24	6.08	10.60	16.68	12.14	4.04	8.10	23.34
20.32	7.11	11.40	18.50	13.60	4.34	9.26	29.58
25.40	7.90	11.94	19.85	14.70	4.55	10.15	35.55
30.48	8.55	12.33	20.88	15.55	4.70	10.85	41.33
38.10	9.30	12.75	22.05	16.53	4.86	11.67	49.77
45.72	9.88	13.05	22.93	17.27	4.97	12.29	58.01
53.34	10.34	13.27	23.61	17.84	5.06	12.79	66.13
63.50	10.83	13.48	24.31	18.44	5.14	13.30	76.80

Figure D

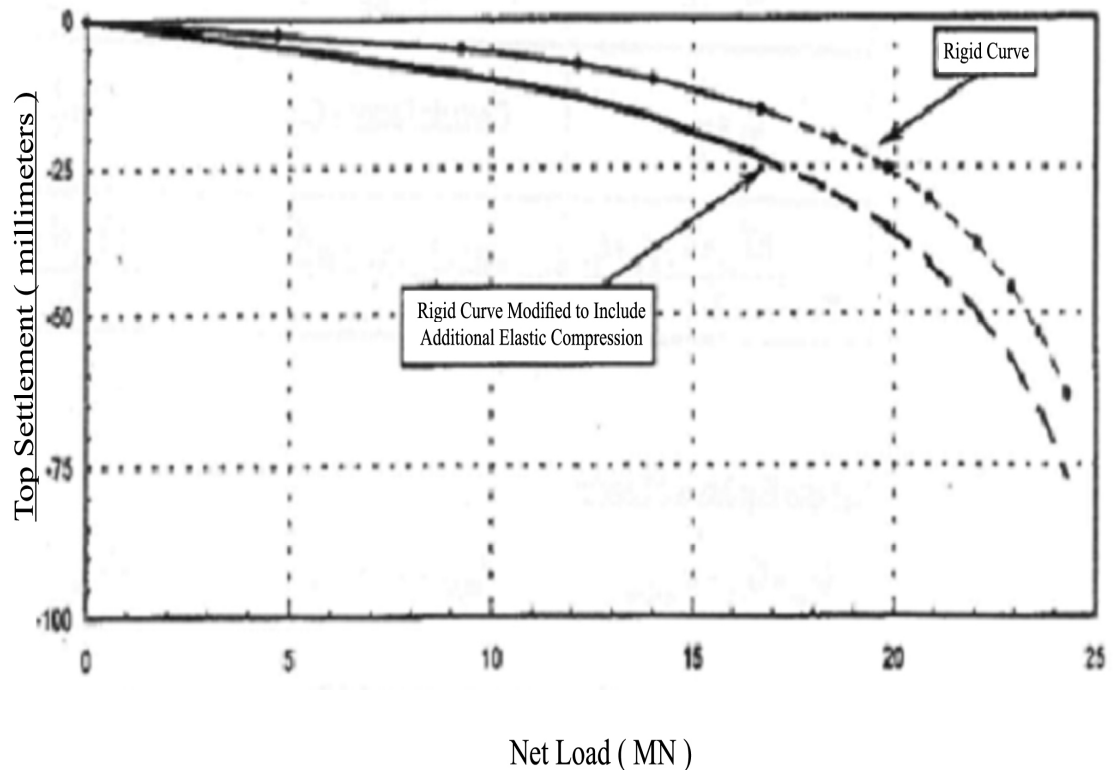


Figure 2.18. Example calculation for the additional elastic compression correction for single level test (Osterberg, 1998).

Given:

$$C_1 = 0.441$$

$$C_2 = 0.579$$

$$C_3 = 0.396$$

$$AE = 17000 \text{ MN (assumed constant throughout test)}$$

$$l_0 = 1.80 \text{ m}$$

$$l_1 = 9.14 \text{ m (embedded length of shaft above mid-cell)}$$

$$l_2 = 5.55 \text{ m (embedded length of shaft between O-Cells)}$$

$$l_3 = 0.00 \text{ m}$$

Shear reduction factor = 1.00 (cohesive soil)

Table 2.2. Example calculation for the additional elastic compression correction for multi level test (Osterberg, 1998).

ΔOLT (in)	$Q_{\downarrow A}$ (kips)	$Q_{\downarrow B}$ (kips)	$Q_{\uparrow B}$ (kips)	P (kips)	δ_{TLT} (in)	δ_{OLT} (in)	$\Delta\delta$ (in)	$\Delta OLT + \Delta\delta$ (in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.54	1.57	1.10	2.04	4.71	3.37	0.64	2.73	5.27
5.08	2.82	2.25	4.18	9.25	6.52	1.31	5.21	10.29
7.62	3.86	2.89	5.37	12.12	8.61	1.69	6.92	14.54
10.16	4.72	3.25	6.04	14.01	10.05	1.90	8.15	18.31
15.24	6.08	3.71	6.89	16.68	12.14	2.17	9.97	25.21
20.32	7.11	3.99	7.41	18.50	13.60	2.33	11.27	31.59
25.40	7.90	4.18	7.76	19.85	14.70	2.44	12.26	37.66
30.48	8.55	4.32	8.02	20.88	15.55	2.52	13.03	43.51
38.10	9.30	4.46	8.29	22.05	16.53	2.61	13.92	52.02
45.72	9.88	4.57	8.48	22.93	17.27	2.67	14.60	60.32
53.34	10.34	4.64	8.62	23.61	17.84	2.71	15.13	68.47
63.50	10.83	4.72	8.76	24.31	18.44	2.76	15.68	79.18

Figure F

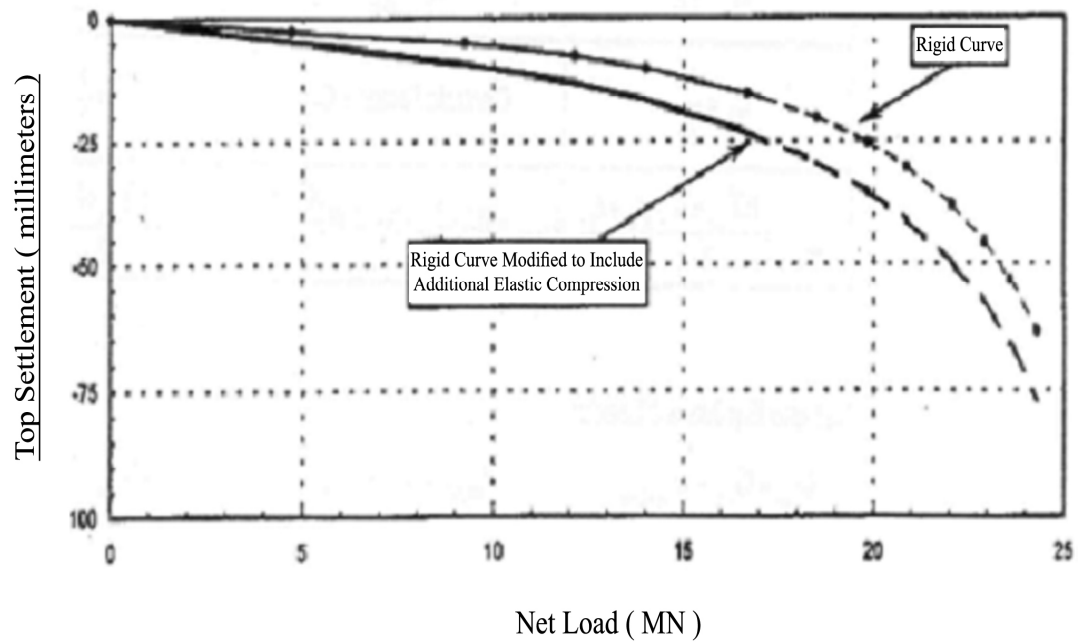


Figure 2.19. Example calculation for the additional elastic compression correction for multi level test (Osterberg, 1998).

- Typical test curves: Figure 2.20 shows an example test result in which the ultimate load in end bearing is reached, meaning that if the working load is less than 1350 tons all the load is taken by side shear.

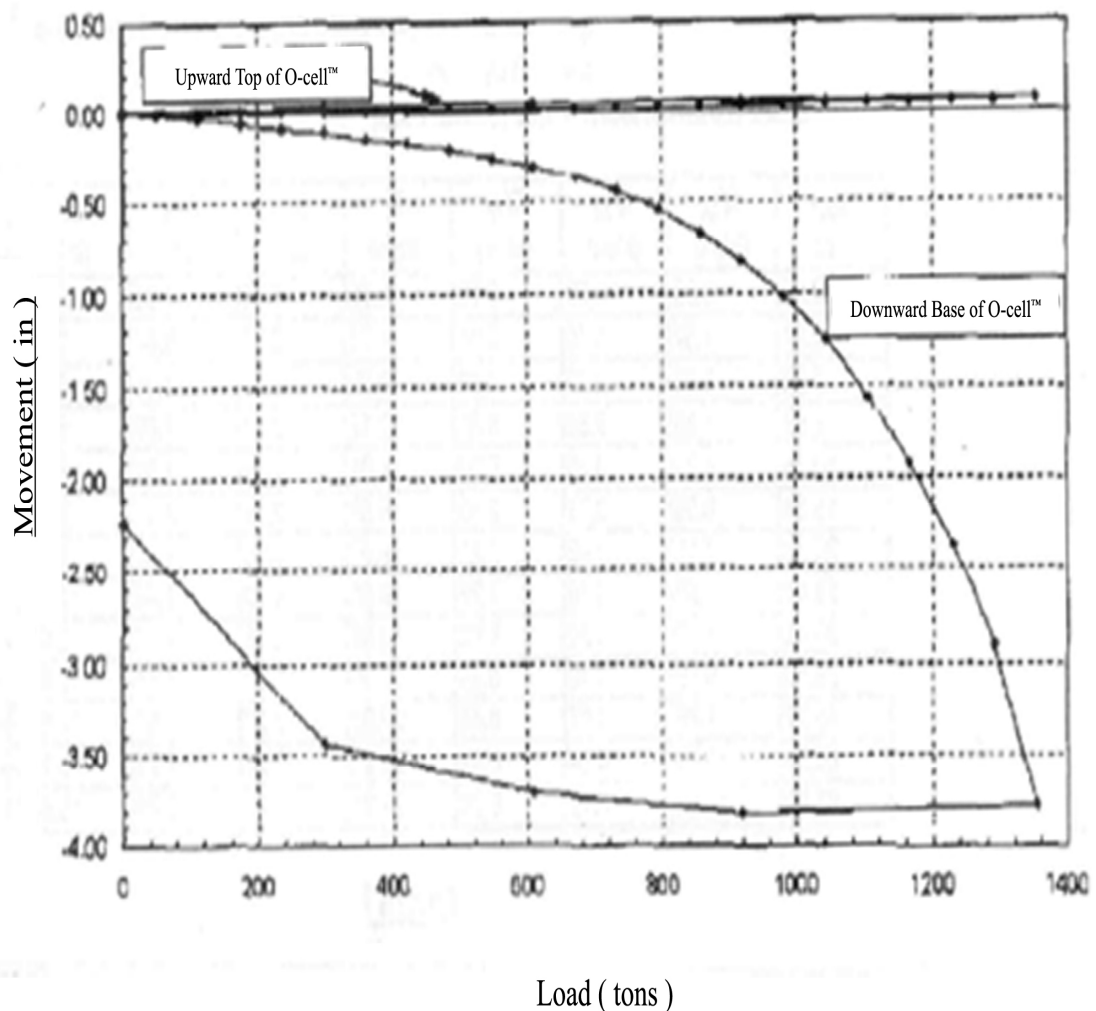


Figure 2.20. Example Osterberg cell load-movement curve when ultimate load is reached in end bearing (Osterberg, 1994).

Figure 2.21 shows an example test curve in which the ultimate load in side shear is reached, as typically seen in clay soils and some rocks. It is seen that when the ultimate in side shear is reached, continuous movement occurs at instant load. When a drilled shaft is constructed on a rock socket, usually the ultimate side shear strength is higher than design

values because in the field the rock is subjected to the weight of overburden and the higher normal pressure makes the shear strength between the laminations increase.

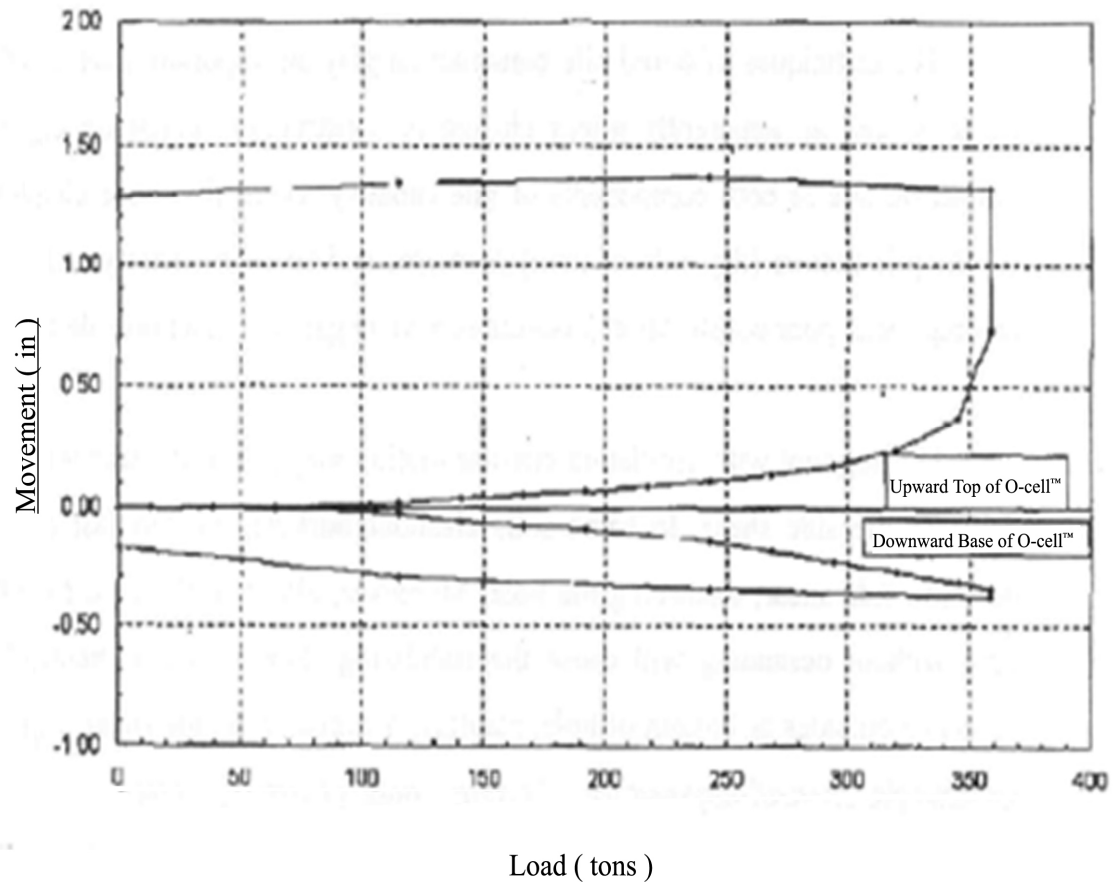


Figure 2.21. Example Osterberg cell load-movement curve when ultimate load is reached in side shear (Osterberg, 1994).

Figure 2.22 shows the load upward movement and load downward movement curves for a drilled shaft where the design load is 500 tons and the test is made to 6000 tons. However, although the capacity of the O-cell was reached so the test discontinued, neither the ultimate load in shear or end bearing is reached (Osterberg, 1994).

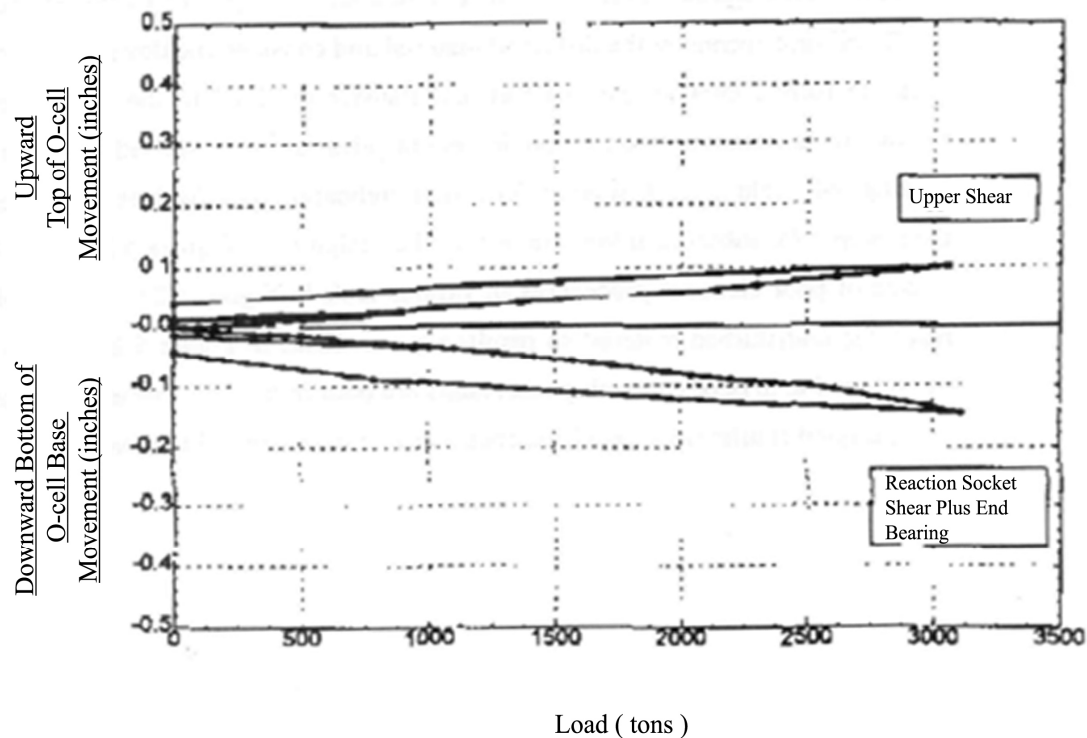


Figure 2.22. Load upward and downward movement curves for a drilled shaft (Osterberg, 1994).

2.2. Comparison of O-Cell Test with Static Load Test

Static top down loading and Osterberg tests were compared and the advantages and disadvantages of O-Cell testing in terms of mobilized loads, evaluating skin friction in rocks sockets, applied stress, test pile selection, testing procedure, interpretation of results and estimated costs were discussed in the following.

By static loading tests the influence of reaction system may be reduced by increasing the spacing between test pile and reaction pile or kentledge; however, it is not always achieved when the working space is restrained.

The results of a conventional static loading test are usually limited to showing the load-movement for the pile head. However, designs concerned with settlement of a pile foundation, or problems and questions arising from the site conditions and construction

procedures, require knowledge of the resistance distribution along the pile, or at least the load-movement characteristics of the pile toe. The shaft and the toe load-movements can be separated by means of analysis of data from instrumented test piles. However, not many have the knowledge and experience of how to design, build, and perform a static loading test on an instrumented pile. The O-cell test comprises a separation of the shaft and toe behavior, see Figure 2.23 , as well as other results important for assessing the adequacy of the pile, and its construction and long-term performance. The O-cell method incorporates a sacrificial hydraulic jack (Osterberg-Cell) placed at or near the toe (base) of the pile to be tested. The test consists of applying load increments to the pile by means of incrementally increasing the pressure in the jack, which causes the O-Cell to expand, pushing the pile shaft upward and the pile toe downward. The measurements recorded are the O-Cell pressure (the load), the upward and downward movements, and the expansion of the O-cell. At the start of the test, the initial load in the pile at the location of the O-cell is carried structurally by the O-cell (fluid pressure in the O-cell is zero). The O-cell assembly is constructed with an internal bond between its top and base, a construction feature, which breaks at a low pressure level allowing the expansion to start. The next few pressure increments transfer the initial load in the pile at the O-Cell location to pressure in the O-cell fluid. This transfer occurs at minimal expansion of the O-Cell. The initial load in the pile is called "residual load" or "locked-in load". Most piles, including bored piles, have a locked-in load developed during and after the construction of the pile. For slender and/or long piles, the load is normally larger than the buoyant weight of the pile. The O-Cell load versus the upward movement of the O-Cell top is the load-movement curve of the pile shaft. The O-Cell load versus the downward movement of the O-Cell base is the load movement curve of the pile toe. This direct information on the load-movement behaviors of the shaft and toe is not obtainable from a conventional static loading test.

A top down load test is not practicable for high loads, which are more than 10-20 MN, as transportation and installation of the system is hard, needs more space, uneconomical and time-consuming.

Tested piles might be used as a working pile in both test methods, provided that the pile meets any differential settlement requirements. O-cell tested piles, however, need fully grouting in and around the cell. One of the main differences in these test methods is that O-

cell test only applies half of the stress to the pile than top-loading test to maintain a ultimate capacity, making the test faster but the structural performance of the pile top remains untested. Inversely, in top loading test more time is needed for load transfer along the length of the pile. Usually, Osterberg testing method is preferred for preliminary testing as it is fast and give more realistic data for design parameters. In proof tests main advantage of this test is its working area availability.

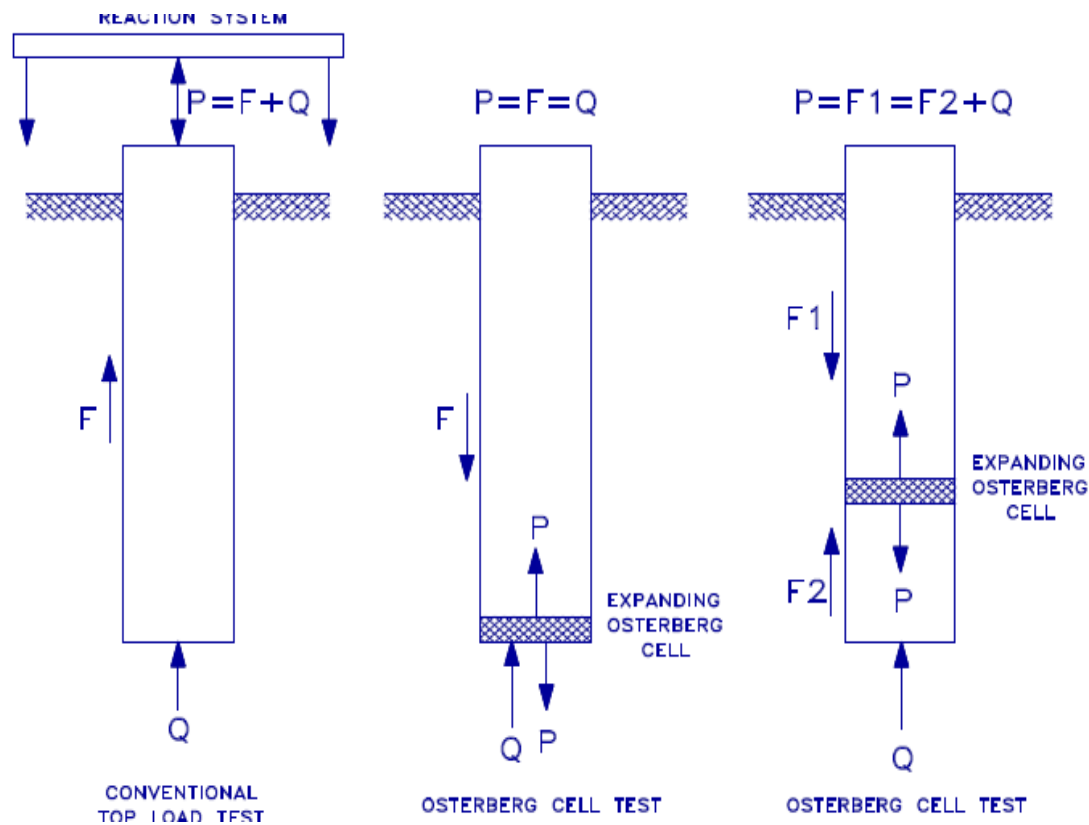


Figure 2.23. Comparison of O-Cell and Conventional Tests (Loadtest, 2010).

At testing sequence, both testing methods use most up to date computerized system; however, Osterberg testing instrumentation is higher as two load test (up and down) are monitored at the same time. The interpretation of test results are more clear for Osterberg test; as stated before, not only the separate behaviour of skin friction and end bearing mobilized are more informative than Fleming, 1992's interpretation method but also assessment of the impact of construction technique is easier (England, 2003 and Schmertmann, 1998).

When Osterberg test results are compared as in a curve with top loaded static tests, the validity of the test can be assured as the curves result in close agreement as in Figure 2.24.

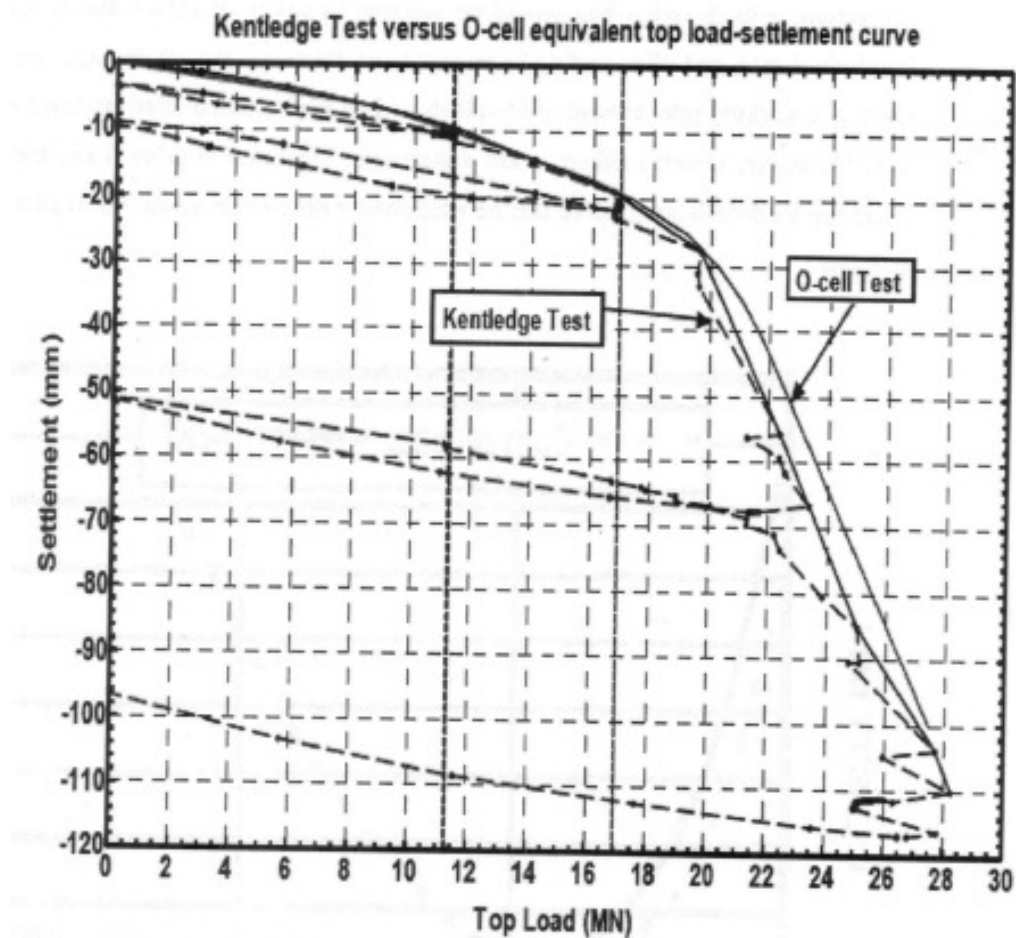


Figure 2.24. Top loaded (kentledge) static test curve versus O-cell equivalent top-loaded settlement curve (England, 2003).

Despite all these advantages of Osterberg cell method, also some limitations of the test compared to the top loaded static load test should be taken into account. First, the O-cell must be installed prior to construction and advance installation is required, including qualified engineer, welders, operators and labor. The installation of tremie from the opening on O-cell is hard and might be unsafe, as the holes are adjacent to the side of borehole. Improper funnel construction may lead to a tremie sticking during casting

operation, resulting in loss of tremie pipes. The cables belonging to all measurement devices should be kept under protection; a thick nylon bag would be suitable to cover all cables standing outside the borehole during and after casting to protect them from any risk of damage. Besides, to be used as a working pile, around of O-cell should be well grouted after the completion of the test. Moreover, O-cell method is not suitable for steel and H piles. Last, the equivalent static top load-deflection curve can be estimated with conservatism, but still remains an estimate (Arda, 2009).

2.3. Finite Element Modeling in Geotechnics

2.3.1. Finite Element Modeling

The finite element method is one of the most powerful approximate solution method that can be applied to solve a wide range of problems represented by ordinary or partial differential equations. The power of such a method derives from the fact that it can easily accommodate changes in the material stiffness which is evaluated at the element level as will be explained shortly. Also, it allows for different boundary conditions to be applied in such a way that an acceptable global approximate solution to the physical problem can be achieved. Considering that closed form solutions cannot be elaborated for a large number of complex physical problems, due to the impossibility of satisfying the boundary conditions related to the corresponding equilibrium equations, the finite element method therefore provides an ideal alternative solution method. In its simplest form, the finite element method consists of

- dividing a given structure or domain into a number of elements; this process is known as discretisation. Elements are connected by nodes at the corners and at the sides as well;
- modeling the behaviour of the unknown variables at different nodes through the use of appropriate interpolation polynomials, known as shape functions.

The shape, size, number, and type of elements depend on the type of structure, and also on the precision required in the solution. In all cases, finite element modeling leads to a matrix formulation that depends on the nature of the physical problem to be solved:

$$[K]\{U\}=\{F\} \quad 2.1$$

where $[K]$ represents the global stiffness matrix (that includes the material properties), $\{U\}$ is the vector of nodal unknown variables, and $\{F\}$ corresponds to the vector of applied nodal loads. Based on the two basic principles stated above, a finite element procedure consists of the following steps (Azizi, 2000) :

- Discretisation and selection of elements: The elements used in conjunction with any discretisation process are selected with the view to obtaining sufficiently precise values of the nodal unknowns.
- Selection of the stress-strain relationships: Most engineers prefer to use simple linear relationships, and in some cases related to homogeneous isotropic soils such assumptions can be justified. However, for more complex soil conditions corresponding to layered soils with different stiffness characteristics, the assumption of a unique stress-strain relationship can be erroneous. Non-linear stress-strain relationships usually lead to iterative procedures in computation, but reliable predictions of stresses and deformations can be obtained.
- Evaluation of element matrices: In finite element modeling, element matrices are first formulated in terms of local coordinates, so that for each element in the mesh, the element stiffness matrix and vector of nodal forces are established using an integral method.
- Assembly of elements matrices and introduction of boundary conditions: Once all elements` stiffness matrices are evaluated in local coordinates, a global stiffness matrix is then determined by first expanding each element matrix so that it is expressed in terms of all nodal variables of the entire mesh, then summing all matrices in a straightforward manner. The boundary conditions in terms of nodal forces are inserted into the global vector.
- Solution to nodal unknowns: This step consists of calculating the vector of nodal unknowns from global equation
- Computation of derived quantities, and analysis of results: Once primary unknowns are determined, the derived quantities can be established. The most important step in finite element modeling consists of analyzing the outcome of computation.

With the advent of computers, more sophisticated methods of analysis have been developed to predict the settlement and load distribution in a single pile. In general, there are three broad categories:

- (i) Load-Transfer Method: This method was first developed by Seed and Reese (1957), which used soil data measured from field tests on instrumented piles and laboratory tests on model piles to build the relationships between pile resistance and pile movement at various points along the pile. Because it is inherently assumed that the movement of the pile at any point is related only to the shear stress at that point and is independent of the stresses elsewhere on the pile, no proper account is taken of the continuity of the soil mass. Besides, precise load-transfer curve needs more instrumentations than for a normal pile load test.
- (ii) Elastic Method: Elastic-based analyses have been employed by several researchers, for example, Nair (1967), Poulos and Davis (1968), Randolph and Wroth (1978). In this method, the piles are divided into a number of uniformly-loaded elements and the soil acts as elastic solid, a solution is obtained by imposing compatibility between the displacements of the pile and the adjacent soil for each segment of the pile. The displacements of the pile are calculated by considering the compressibility of the pile under axial loading while the soil displacements are usually obtained by using Mindlin's equations. However, due to the limitation of the linear elastic soil model, pile-soil interaction in pile load test is always overestimated.
- (iii) Numerical Method: Finite element technique allows more variables to be considered in the problem. Ellison et al. (1971) have considered a multilinear soil stress-strain curve and have introduced special joint elements at the pile interface to allow for slip. Other investigators include Desai (1974) etc. The method involves discretizing of the pile and soil domains into a finite number of elements. Stiffness equations are formulated for each element and assembled together to give the global system. The appropriate constitutive models are selected to simulate the stress-strain behavior of soil so that soil inhomogeneity and nonlinearity can be studied in a rigorous manner. With the development of high

performance PC, some powerful FEM programs, such as PLAXIS, have been widely used in research, which made it possible that more factors such as 3D effects can be taken into consideration so that more realistic situations can be simulated.

2.3.1.1. Practical Aspects of Finite Element Modeling in Geotechnics. Finite element modeling of soil-structure interaction problems needs to be planned and undertaken carefully, so that any anomalies in the results can be spotted and remedied.

The first step consists of discretising the domain, in other words generating a mesh. A reasonable mesh should be refined near any applied load where the stress and displacement gradients are expected to be large, and around any geometric or material discontinuities. Also, the mesh needs to be refined around the areas where changes in material properties occur, as in the case of different soil layers, or around a soil-structure interface. In such cases, interface elements may be needed to simulate friction at the soil-structure interface, particularly when structure surface is relatively smooth.

The size of elements should be increased progressively around the refined portions of the mesh in a way that ensures a smooth transition from small to larger size elements. In any case, it is strongly advised to avoid using distorted elements. The reason being that, once loaded, each of these elements can potentially deform to the point where the stiffness matrix becomes singular thus causing numerical instability. Transitional elements are mainly used in conjunction with 3-D modeling.

The type of elements used in a finite element modeling depends on the type of problem to be analysed. Nevertheless, one has to bear in mind that the strains and stresses are derived from displacements. The calculation of strains and stresses is undertaken within each element at specific integration points. Once calculated at the integration points, the stresses are then extrapolated to the element nodes. Because each element has its own stiffness matrix, the stress discontinuity can be smoothed by taking the average value at the node. However, nodal averaging should not be used for stress and strain calculations at nodes connecting elements with different stiffness's. Numerical evidence shows that the

stress field is not affected by the stress discontinuity provided that a refined mesh with smaller size elements around these boundaries is used.

Geometric or material discontinuities constitute singularities which are unavoidable for a large number of soil-structure interaction problems. This can generate inaccurate stress fields near the singularities. However, these effects can be minimized by using a refined mesh around the singularity (Azizi, 2000).

2.3.2. Introduction to Plaxis

2.3.2.1. General. Plaxis is a finite element package that has been developed specifically for the analysis of deformation and stability in geotechnical engineering projects.

Geotechnical applications require advanced constitutive models for the simulation of the non linear, time dependent and anisotropic behaviour of soils and rock. In addition, since soil is a multi phase material, special procedures are required to deal with hydrostatic and non hydrostatic pore pressures in the soil. Although the modeling of the soil itself is an important issue, many projects involve the modeling of structures and the interaction between the structures and the soil (Brinkgreve, 2002).

The simple graphical input procedures enable a quick generation of complex finite element model, and the enhanced output facilities provide a detailed presentation of computational results. The calculation itself is fully automated and based on robust numerical procedures.

PLAXIS 8.2 is a finite element code used for the numerical simulation of pile load test in this thesis.

2.3.2.2. Model. For each new project to be analyzed it is important to create a geometry model first. A geometry model is a 2D representation of a real three-dimensional problem and consists of points, lines and clusters. A geometry model should include a representative division of the subsoil into distinct soil layers, structural objects, construction stages and loadings. The model must be sufficiently large so that the

boundaries do not influence the results of the problem to be studied. Points form the start and end of lines. Points can also be used for the positioning of anchors, point forces, point fixities and for local refinements of the finite element mesh. Lines are used to define the physical boundaries of the geometry, the model boundaries and discontinuities in the geometry such as walls or shells, separations of distinct soil layers or construction stages. Clusters are areas that are fully enclosed by lines. Within a cluster the soil properties are homogeneous. Actions related to clusters apply to all elements in the cluster (Brinkgreve, 2002).

In PLAXIS 8.2, plane strain model can be used for structures with an almost uniform cross section, corresponding stress state and loading scheme over a certain length perpendicular to the cross section. Displacements perpendicular to the cross section are assumed to be zero. Axisymmetric model can be used for circular structures with uniform radial cross section and loading scheme around the central axis, where deformation and stress state are assumed to be identical in any radial direction. To analyze the problem of pile, the axisymmetric model should be selected, which results in a two dimensional finite element model with only two translational degrees of freedom at each node (i.e. x- and y-direction).

2.3.2.3. Elements. During the generation of the mesh, clusters are divided into triangular elements. In PLAXIS 8.2, 6-node or 15-node triangular elements are available. The powerful 15-node element provides an accurate calculation of stresses and failure loads. In addition, 6-node triangles are available for a quick calculation of serviceability states. Considering the same element distribution the user should be aware that meshes composed of 15-node elements are actually much finer and much more flexible than meshes composed of 6-node elements, but calculations are also more time consuming. A 15-node element consists of 15 nodes and a 6-node triangle is defined by 6 nodes. The distribution of nodes over the elements is shown in Figure 2.25. Adjacent elements are connected through their common nodes. During a finite element calculation, displacements are calculated at the nodes. Nodes may be preselected for the generation of load-displacement curves. In contrast to displacements, stresses and strains are calculated at individual Gaussian integration points, also called stress points, rather than at the nodes. A 15-node

triangular element contains 12 stress points and a 6-node triangular elements contains 3 stress points as seen in Figure 2.25.

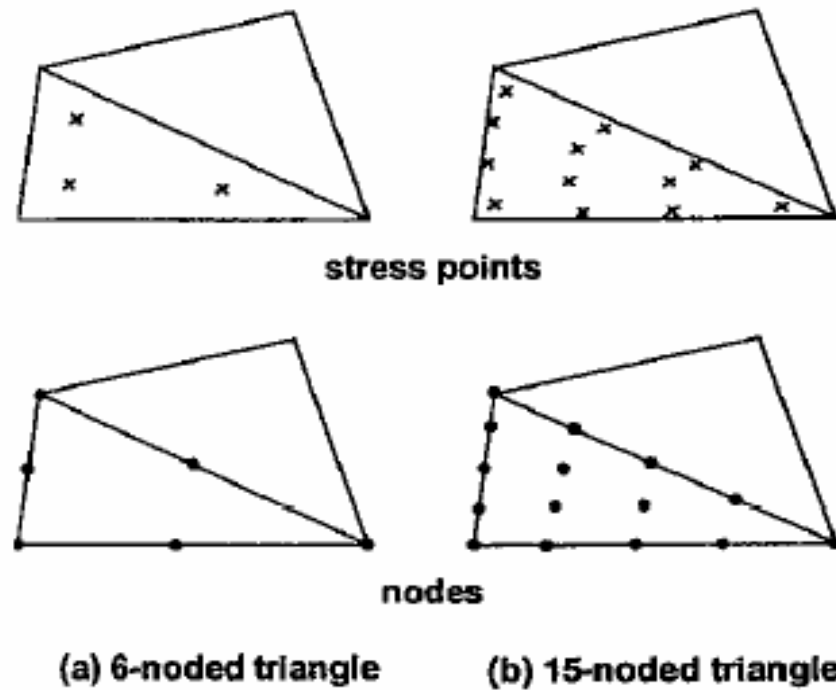


Figure 2.25. Nodes and Stress Points

Stress points may be preselected for the generation of stress paths or stress-strain diagrams (Brinkgreve, 2002).

2.3.2.4. Interfaces. Interfaces are used when modeling Soil-Structure interaction. Interfaces will be required to simulate the finite frictional resistance between the structure such as pile and adjacent soil. It allows relative displacement and separation between the structure and soil mass. Interfaces are composed of interface elements. When using 15-node soil elements, the corresponding interface elements are defined by five pairs of nodes, whereas for 6-node soil elements the corresponding interface elements are defined by three pairs of nodes. In the finite element formulation the coordinates of each node pair are identical, which means that the element has a zero thickness.

The stiffness matrix for interface elements is obtained by means of Newton Cotes integration. The position of these integration points coincides with the position of the node pairs. The 6-node interface elements use a 3-point Newton-Cotes integration, whereas the 10-node interface elements use 5-point Newton-Cotes integration.

The basic property of an interface element is the associated material data set for soil and interfaces. When interface element models the interaction between a pile and the soil, which is intermediate between smooth and fully rough. The roughness of the interaction is modeled by choosing a suitable value for the strength reduction factor in the interface (R_{inter}). This factor relates the interface strength (structure surface friction and adhesion) to the soil strength (friction angle and cohesion). An elastic-plastic model is used to describe the behaviour of interfaces for the modeling of soil-structure interaction. The Coulomb criterion is used to distinguish between elastic behaviour, where small displacements can occur within the interface, and plastic interface behaviour when permanent slip may occur (Yi, 2004).

For the interface to remain elastic the shear stress τ is given by:

$$|\tau| < \sigma_n \tan \phi_i + c_i \quad (2.2)$$

and for plastic behaviour τ is given by:

$$|\tau| = \sigma_n \tan \phi_i + c_i \quad (2.3)$$

where ϕ_i and c_i are the friction angle and cohesion (adhesion) of the interface, σ_n is the normal stress of the soil. The strength properties of interfaces are linked to the strength properties of a soil layer. Each data set has an associated strength reduction factor for interface (R_{inter}). The interface properties are calculated from the soil properties in the associated data set and the strength reduction factor by applying the following rules:

$$c_i (R_{inter} c_{soil}) \leq c_{soil} \quad (2.4)$$

$$\tan \varphi_i (=R_{\text{inter}} \tan \varphi_{\text{soil}}) \leq \tan \varphi_{\text{soil}} \quad (2.5)$$

2.3.2.5. Material Model. The behaviour of soil and other continua are simulated by various models supported by PLAXIS.

- i. Linear Elastic Model: This model represents Hooke's law of isotropic linear elasticity. The model involves two elastic stiffness parameters, i.e. Young's modulus, E , and Poisson's ratio, ν . As it involves only two input parameters, the linear elastic model is too crude to capture essential features of soil and rock behaviour. It is primarily used for stiff massive structural systems install in the soil, such as the test piles in this thesis.
- ii. Mohr-Coulomb Model: This well known model is usually used as a first approximation of soil behaviour, such as the soil profiles in this thesis. Due to its simplicity, it is highly popular and gives reasonable results. The model involves five parameters, i.e. Young's modulus, E , Poisson's ratio, ν , cohesion, c , internal friction angle, ϕ , and dilatancy angle, ψ .

For each layer one estimates a constant average stiffness. Due to this constant stiffness, computations tend to be relatively fast and one obtains a first impression of deformations. Besides the five model parameters mentioned above, initial soil conditions play essential role in most soil deformation problems. Initial horizontal soil stresses have to be generated by selecting proper K_0 values.

In real soils, the stiffness depends significantly on the stress level, which means that the stiffness generally increases with depth. The advanced M-C model in PLAXIS provides an option to account for the increase of the stiffness with depth. The Eincement is the increase of the Young's modulus per unit of depth (expressed in the unit of stress per unit depth). At the level given by the yref parameter, the stiffness is equal to the reference Young's modulus, E_{ref} . The actual value of Young's modulus in the stress points is obtained by Eq.2.6. However, during calculations a stiffness increasing with depth does not change as a function of the stress state (Brinkgreve, 2002).

$$E_{\text{actual}} = E_{\text{ref}} + (y_{\text{ref}} - y) E_{\text{increment}} \quad y < y_{\text{ref}} \quad (2.6)$$

Similarly, the increase of the cohesion with depth is accounted for in the M-C model in PLAXIS, as in Eq.2.7.

$$c_{\text{actual}} = c_{\text{ref}} + (y_{\text{ref}} - y) c_{\text{increment}} \quad y < y_{\text{ref}} \quad (2.7)$$

- iii. **Jointed Rock Model:** This is an anisotropic elastic-plastic model, especially meant to simulate the behaviour of rock layers involving a stratification and particular fault directions. Plasticity can only occur in a maximum of three shear directions. Each plane has its own strength parameters ϕ and c . The intact rock is considered to behave fully elastic with constant stiffness properties E and ν . Reduced elastic properties may be defined for the stratification direction (Brinkgreve, 2002).
- iv. **Hardening Soil Model:** This is an elastoplastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. Moreover, the model involves compression hardening to simulate irreversible compaction of soil under primary compression. This second-order model can be used to simulate the behaviour of sands and gravel as well as softer types of soil such as clays and silts.

When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. The observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola in the special case of a drained triaxial test.

The Hardening-Soil model has the following advantages of the others: Firstly by using the theory of plasticity rather than the theory of elasticity; secondly by including soil dilatancy and thirdly by introducing a yield cap. As for the Mohr-Coulomb model, limiting states of stress are described by means of the friction angle, ϕ , the cohesion, c , and the dilatancy angle, ψ . However, soil stiffness is described much more accurately by using three different input stiffnesses which are triaxial loading stiffness, E_{50} , the triaxial unloading stiffness, E_{ur} , and the oedometer loading stiffness, E_{oed} and more complicated parameters such as Poisson's ratio for unloading-reloading, ν_{ur} , coefficient of lateral stress

in normal consolidation, KoNC etc.. As average values for various soil types, we have $E_{ur} \approx 3 E_{50}$ and $E_{oed} \approx E_{50}$, but both very soft and very stiff soils tend to give other ratios of E_{oed} / E_{50} (Yi, 2004).

In contrast to the Mohr-Coulomb model, the Hardening-Soil model also accounts for stress-dependency of stiffness moduli, which means that all stiffnesses increase with pressure. Hence, all three input stiffness relate to a reference stress, being usually taken as 100 kPa.

- v. Soft Soil Model: The Soft Soil model is a Cam-Clay type model especially meant for primary compression of near normally-consolidated clay-type soils. The capabilities of this model are superseded by the Hardening Soil model (Brinkgreve, 2002).
- vi. Soft Soil Creep Model: This is a second order model formulated in the framework of viscoplasticity. The model can be used to simulate the time-dependent behaviour of soft soils like normally consolidated clays and peat. The model includes logarithmic compression.

All soils exhibit some creep and primary compression is thus followed by a certain amount of secondary compression. The latter is most dominant in soft soils. The Soft Soil Creep model has been developed for application to settlement problems of foundation, embankments, so on (Brinkgreve, 2002).

2.3.2.6. Material Model Summary. It is advised to use the Mohr-Coulomb model for a relatively quick and simple first analysis of the problem considered. When good soil data is lacking, there is no use in further more advanced analyses.

In many cases, one has good data on dominant soil layers, and it is appropriate to use the Hardening-Soil model in additional analysis. One seldom has test results from both triaxial and oedometer tests, but good quality data from one type of test can be supplemented by data from correlations and/or in situ testing.

Finally, a Soft-Soil-Creep analysis can be performed to estimate creep, i.e. secondary compression in very soft soils. The above idea of analyzing geotechnical problems with different soil models may seem costly, but it tends to pay off, firstly, due to the fact that the Mohr-Coulomb analysis is relatively quick and simple and secondly as the procedure tends to reduce errors (Brinkgreve, 2002).

2.3.2.7. Mesh Properties. PLAXIS allows for a fully automatic generation of finite element mesh. The generation of the mesh is based on a robust triangulation procedure, which results in “unstructured” meshes. These meshes may look disorderly, but the numerical performance of such meshes may yield better results than for regular structured meshes.

2.3.2.8. Staged Construction. PLAXIS allows for the option to change the geometrical configurations by activating and deactivating clusters or structural objects, which is convenient to simulate the installation of the pile as a material wish-in-place, for the purpose of the study in the following. Also the program allows an accurate and realistic simulation of the actual construction stages such as the loading changes of the reaction system during the process of pile loading. The material properties and pore pressure distribution can also be changed at each stage.

2.3.2.9. Undrained Analysis with Effective Parameters. PLAXIS allows specifying undrained behaviour in an effective stress analysis using effective model parameters. This can be achieved by transforming the inverted form of Hooke’s law in terms of the total stress rates and the undrained parameters E_u and ν_u according to Terzaghi’s principle, that is stresses in the soil are divided into effective stresses, σ' and pore pressures, σ_w :

$$\underline{\sigma} = \underline{\sigma'} + \underline{\sigma_w} \quad (2.8)$$

In which, $\underline{\sigma}$ is vector notation involving six different components:

$$\underline{\sigma} = (\sigma_{xx} \ \sigma_{yy} \ \sigma_{zz} \ \sigma_{xy} \ \sigma_{yz} \ \sigma_{zx})^T \quad (2.9)$$

A further distinction is made between steady state pore stress, p_{steady} , and excess pore stress, u_{excess} :

$$\sigma_w = u_{\text{steady}} + u_{\text{excess}} \quad (2.10)$$

Steady state pore pressures are considered to be input data, i.e. generated on the basis of phreatic levels or groundwater flow. Excess pore pressures are generated during plastic calculations for the case of undrained material behavior. Undrained material behaviour and the corresponding calculation of excess pore pressures are described below. Since the time derivative of the steady state component equals zero, it follows:

$$\dot{\sigma}_w = \dot{u}_{\text{excess}} \quad (2.11)$$

Considering slightly compressible water, the rate of pore pressure is written as:

$$\dot{\sigma}_w = K_w (\dot{\epsilon}_{xx}^e + \dot{\epsilon}_{yy}^e + \dot{\epsilon}_{zz}^e) / n \quad (2.12)$$

in which K_w is the bulk modulus of the water and n is the soil porosity. The inverted form of Hooke's law may be written in terms of the total stress rates and the undrained parameters E_u and ν_u where:

$$E_u = 2G(1 + \nu_u) \quad (2.13)$$

$$\nu_u = [\nu + \mu(1 + \nu)] / [1 + 2\mu(1 + \nu)] \quad (2.14)$$

$$\mu = K_w / 3nK \quad (2.15)$$

$$K = E / [3(1 - 2\nu)] \quad (2.16)$$

Hence, the special option for undrained behaviour in PLAXIS is such that the effective parameters G and ν are transferred into undrained parameters E_u and ν_u according to Eqs. 2.8~2.10.

Fully incompressible behaviour is obtained for $v_u=0.5$. However, it leads to singularity of the stiffness matrix. In fact, water is not fully incompressible although a realistic bulk modulus for water is very large. PLAXIS has taken a default v_u as 0.495 to avoid the numerical problems caused by an extremely low compressibility.

This special option to model undrained material behaviour on the basis of effective model parameters is available for all material models in the PLAXIS program. However, for such a project that in situ tests and laboratory test may have been performed to obtain undrained soil parameters, PLAXIS offers the option of an undrained analysis with direct input of the undrained shear strength (c_u or s_u) and $\phi=\phi_u=0^\circ$ in M-C model and H-S model.

3. CASE STUDIES

3.1. Introduction

Five case histories are presented in this chapter. For all five tests the allowable capacity and displacement values were compared with the numerical simulation of these five different case studies. First, second, fourth and fifth tests were conducted and simulated as axially static load tests. The third one was conducted as bidirectional static load test -O-cell test- on site, but it was also simulated numerically as static load test, so the comparison of O-cell test with static pile load test and equivalency and discrepancy of the test results between the two types of pile load test are demonstrated and analyzed.

3.2. Case Study 1

3.2.1. FEM Study on Case Study 1

The idealized finite element model of static pile load test is simulated with finite element analysis and illustrated in Figure 3.1 which is the model of conventional top load test. The test is simulated in Mohr Coulomb soil model with 15 node elements; the mesh is generated axisymmetrically, with the boundary of 26 meters wide and 52 meters deep. The test pile is in the same size of diameter of 0,8 meters and length of 26 meters. To ensure the precision of the calculation, the meshes of an area of 5 meters wide, 32 meters deep around the test pile are refined.

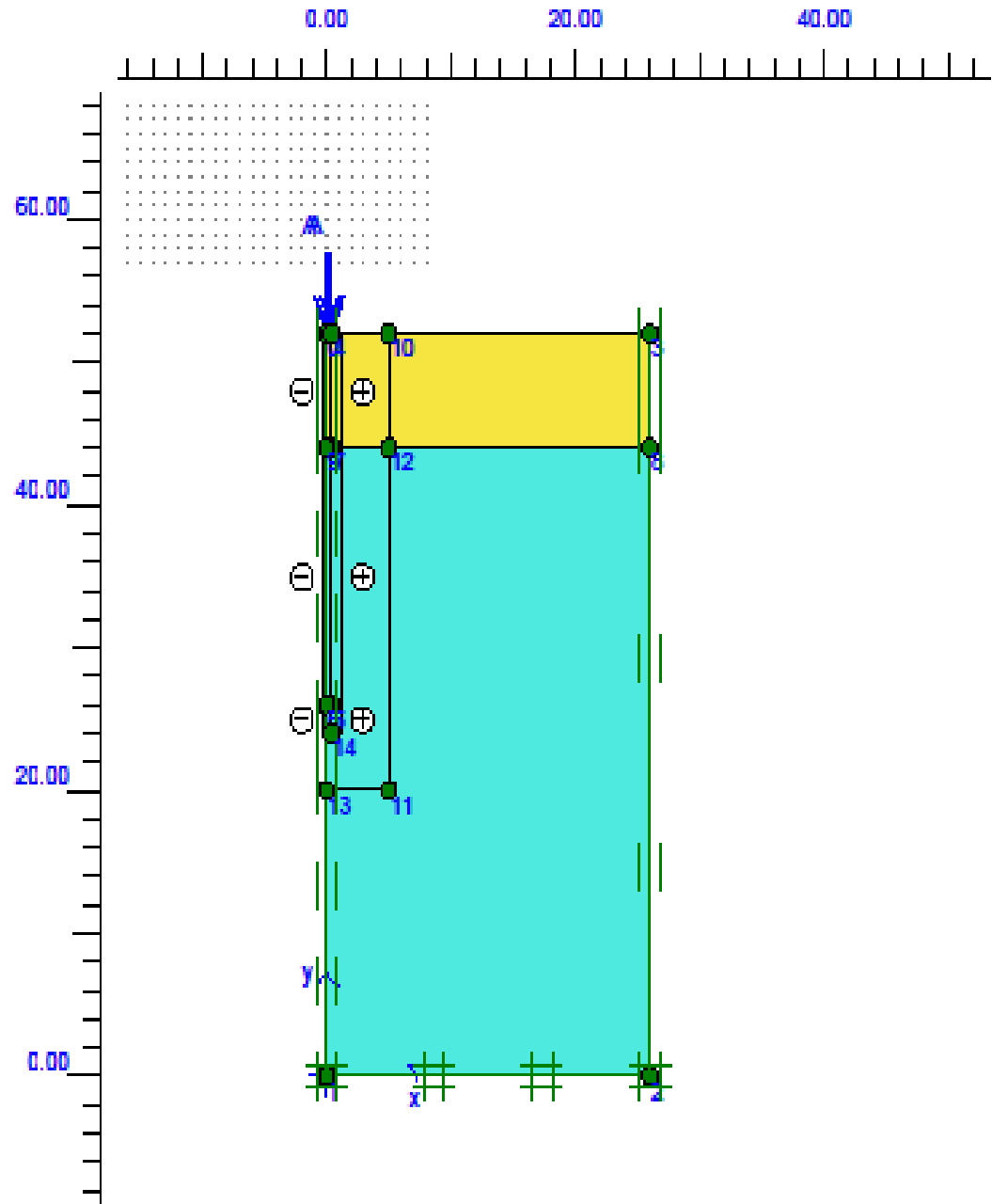


Figure 3.1. Finite element model of case1.

3.2.1.1. Soil Conditions and Design Parameters. Design parameters are obtained by incorporation of soil boring log which is seen in Figure 3.2 and Figure 3.3; and also by lab reports which is seen in Figure 3.4.

GEOTEM LTD.ŞTİ.		SONDAJ LOGU BORING LOG		SONDAJ NO / Boring : YSN-8															
				SAYFA NO / Page No : 1															
				SONDÖR / Driller : Yusuf ŞEN															
PROJE ADI / Project Name : OSMANGAZI-DEMİRTAŞ MEVKİİ (BURSA) 301 ADA 13 PARSEL ZEMİN VE TEMEL ETÜD SONDAJLARI																			
SONDAJ DERİNLİĞİ / Boring Depth (m) : 30,00			BAŞLAMA TARİHİ / Start Date : 23.02.2007																
SONDAJ KOTU / Elevation (m) : 88,90			BİTİŞ TARİHİ / Finish Date : 24.02.2007																
YERALTI SUYU / Groundwater (m) : 3,50			KOORDİNAT / Coordinate (N-S) y : 4457293																
MAKİNA TİPİ / YÖNTEMİ : PUNTEL 2-C / ROTARY			KOORDİNAT / Coordinate (E-W) x : 422068																
SONDAJ DERİNLİĞİ (m) Boring Depth	NUMUNE CİNSİ Sample Type	NUMUNE NO Sample No.	STANDART PENETRASYON DENEYİ Standard Penetration Test		ZEMİN SINIFI Soil Category														
			DARBE SAYISI Num.Of. Blows	GRAFİK Graph	ZEMİN TANIMLAMASI Soil Description														
			0-15 cm	15-30 cm	30-45 cm	N ₆₀	10	20	30	40	50		ZEMİN PROFİLİ Soil Profile	MANEYRA BOYU(m) Run	DAYANIKLILIK Sıvı Strength	AYRILMA Weathering	KIRIK/90 cm	KAROT % Core Recovery	RQD %
1	1.50 SPT	1	6	5	5	10							Bitkisel Toprak 0.50 m						
2	1.95 SPT	2	5	7	4	11													
3	3.00 SPT	2	5	7	4	11													
4	3.45 UD	1*																	
5	4.00 SPT	3	5	6	6	12													
6	4.50 SPT	3	5	6	6	12													
7	4.95 SPT	4	8	6	4	10													
8	6.00 SPT	4	8	6	4	10													
9	6.45 SPT	5	6	7	7	14													
10	7.50 SPT	5	6	7	7	14													
11	7.95 SPT	6	5	8	9	17													
12	9.00 SPT	6	5	8	9	17													
13	9.45 SPT	7	7	10	10	20													
14	10.50 SPT	7	7	10	10	20													
15	10.95 SPT	8	6	8	11	19													
16	12.00 SPT	8	6	8	11	19													
17	12.45 SPT	9	8	8	9	17													
18	13.50 SPT	9	8	8	9	17													
19	13.95 SPT	10	9	10	12	22													
20	15.00 SPT	10	9	10	12	22													
21	15.45 SPT	11	8	10	14	24													
22	16.50 SPT	11	8	10	14	24													
23	16.95 SPT	12	7	9	13	22													
24	18.00 SPT	12	7	9	13	22													
25	18.45 SPT	13	8	8	12	20													
26	19.50 SPT	13	8	8	12	20													
27	19.95 SPT	13	8	8	12	20													

SPT : Standart Pen. Deneyi Standard Pen. Test	K : Karot Numunesi Core Sample	Sondaj Mühendisi Drilling Engineer	İmza Sign	Kontrol Mühendisi Control Engineer	İmza Sign
D : Örselenmiş Numune Disturbed Sample	P : Presiyometre Deneyi Pressuremeter Test	Mustafa GÜLAY Jeoloji Müh.			
UD : Örselenmemiş Numune Undisturbed Sample	VST : Vane Deneyi Vane Shear Test				

Figure 3.2. Boring Log Page 1.

GEOTEM LTD.ŞTİ.		SONDAJ LOGU BORING LOG		SONDAJ NO / Boring : YSN-8																			
				SAYFA NO / Page No : 2																			
				SONDÖR / Driller : Yusuf ŞEN																			
PROJE ADI / Project Name : OSMANGAZI-DEMİRTAŞ MEVKİİ (BURSA) 301 ADA 13 PARSEL ZEMİN VE TEMEL ETÜD SONDAJLARI				BAŞLAMA TARİHİ / Start Date : 23.02.2007																			
SONDAJ DERİNLİĞİ / Boring Depth (m) : 30,00				SONDAJ KOTU / Elevation (m) : 88,90																			
YERALTI SUYU / Groundwater (m) : 3,50				KOORDİNAT / Coordinate (N-S) y : 4457293																			
MAKİNA TİPİ / YÖNTEMİ : PUNTEL 2-C / ROTARY				KOORDİNAT / Coordinate (E-W) x : 422068																			
SONDAJ DERİNLİĞİ (m) Boring Depth	NUMUNE CİNSİ Sample Type	NUMUNE NO Sample No.	STANDART PENETRASYON DENEYİ Standard Penetration Test					ZEMİN SINIFI Soil Category	ZEMİN TANIMLAMASI Soil Description	ZEMİN PROFİLİ Soil Profile	MANEVRİ BOYU (m) Run	DAYANIKLILIK Strength	AYRISMA Weathering	KIRIK/30 cm Fracture	KAROT % Core Recovery	RQD %							
			DARBE SAYISI Num.Of. Blows				GRAFIK Graph																
			0-15 cm	15-30 cm	30-45 cm	N ₃₀																	
21	21.00 SPT 21.45	14	6	7	9	16		CL	Kumlu KİL Yeşilimsi-kahverenkli, çakıllı, siltli, yer yer ince kum bantlı														
22	22.50 SPT 22.95	15	9	8	15	23																	
23	24.00 SPT 24.45	16	7	10	10	20																	
24	25.50 SPT 25.95	17	9	11	12	23																	
25	27.00 SPT 27.45	18	11	11	11	22																	
26	28.50 SPT 28.95	19	10	10	14	24																	
27	29.55 SPT 30.00	20	9	13	17	30																	
28																							
29																							
30																							
31							CL	Kumlu KİL Kahverenkli, çakıllı, siltli															
32																							
33																							
34																							
35																							
KIVAM DURUMU/STIFFNESS			SİKLİLİK/DENSITY			DAYANIMLILIK/STRENGTH			AYRISMA/WEATHERING														
N=0-3 Çok Yumuşak N=3-4 Yumuşak N=4-8 Orta Kattı N=8-15 Katı N=15-30 Çok Katı N>30 Sert			V=Soft M=Stiff V=Very Stiff			N=0-4 Çok Gevrek N=4-10 Gevrek N=10-30 Orta Sıkı N=30-50 Sıkı N>50 Çok Sıkı			V=Loose M=Loose M=Dense V=Dense			I.Dayanimsız II.Orta Dayanimsız III.Orta Zayıf IV.Zayıf V.Cok Zayıf			Strong M.Strong M.Weak Weak V.Weak			I.Taze II.Az Ayrışmış III.Orta D.Ayrışmış IV.Cok Ayrışmış V.Tam Ayrışmış			Fresh Slightly W. Mod.Weath Highly W. Comp Weath		
ORANLAR/Proportions			ORANLAR/Proportions			KIRIKLAR/30cm/FRACTIONS/30 cm			KAYA KALİTESİ TANIMI-RQD														
%5 < Pek Az %5-15 Az %15-35 Çok > %35 Ve			Slightly Little Very And			%5 < Pek Az %5-10 Az %10-30 Çok Very			Slightly Little Very			<1 Seyrek 1-2 Orta 2-10 Sık 10-20 Çok Sıkı >20 Parçalı			Wide (W) Moderate (M) Close (C) Intense (I) Crushed (Cr)			% 0-25 Çok Zayıf %25-50 Zayıf %50-75 Orta %75-90 İyi %90-100 Çok İyi			V.Poor Poor Fair Good Excellent		
SPT : Standart Pen. Deneyi D : Örselenmiş Numune UD : Örselenmemiş Numune			K : Karot Numunesi P : Presiymetre Deneyi VST : Vane Deneyi			Sondaj Mühendisi Drilling Engineer			İmza Sign			Kontrol Mühendisi Control Engineer			İmza Sign								
						Mustafa GÜLAY Jeoloji Müh.																	

Figure 3.3. Boring Log Page 2.

JEOTASARIM 2000

ZEMİN MEKANİĞİ ve YAPI MALZEMELERİ

LABORATUARLARI

BELGE NO 49-50 ANKARA MERKEZ: EMEK KÜÇÜK SAN. SİT. 646. SK. NO:7 OSTİM/ANKARA Tel: (312) 3954875 Fax: 395 25 17
 SAKARYA ŞUBE: TIĞCILAR MAH. AKÇA SK. NO:1/A ADAPAZARI /SAKARYA Tel: (264) 278 77 01 Fax: 2787771

BELGE NO 49-50

DENEY TALEP EDEN					GEOTEM MÜHENDİSLİK					NUMUNENİN GELİŞ TARİHİ					28,02,07		
PROJE ADI					301 Ada 13 Parsel					DENEY TARİHİ					28,02-19,03,		
NUMUNE YERİ					BURSA / OSMANGAZI					RAPOR TARİHİ					19,03,07		
SAYFA NUMARASI					1/60					KAYIT/RAPOR NO					07-0225		
LABORATUAR DENEYLERİ SONUÇ RAPORU																	
Sondaj No	Numune Tipi	Derinlik (m)	Özgül Ağırlık (Gs)	Doğal B.H.A (γn) gr/cm³	Su İçeriği (w) %	Elek Analizi		Atterberg Limitleri			Zemin Tipi	Tek Eksenli Basınç		Üç Eksenli Basınç (UU)		Odömetre	
						No.4 Kalan %	No.200 Geçen %	LL %	PL %	PI %		cu kg/cm2	qu kg/cm2	c kg/cm2	φ (o)	Şişme Yüzdesi (%)	Şişme Basıncı kg/cm²
YSN-6	SPT	3.00			29.99	0.70	76.67	40	19	21	CL						
YSN-6	UD	5.50	2.728	1.88	33.80	0.41	94.88	56	27	29	CH	0.39	0.77				
YSN-6	SPT	7.50			30.95	0.26	83.59	39	17	22	CL						
YSN-6	SPT	13.50			29.57	0.00	82.94	41	20	21	CL						
YSN-6	SPT	16.50			29.62	0.00	85.60	41	19	22	CL						
YSN-6	SPT	24.00			27.49	0.60	71.58	43	19	24	CL						
YSN-6	SPT	29.55			27.87	0.47	70.63	40	18	22	CL						
YSN-8	SPT	3.00			19.50	0.19	80.28	39	20	19	CL						
YSN-8	UD	4.00	2.713	1.98	22.43	0.17	86.16	47	21	26	CL			0.86	5		
YSN-8	SPT	4.50			23.58	0.00	89.31	45	22	23	CL						
YSN-8	SPT	9.00			23.74	0.00	88.89	47	22	25	CL						
YSN-8	SPT	15.00			29.66	0.20	69.70	39	18	21	CL						
YSN-8	SPT	21.00			29.28	0.31	67.94	41	18	23	CL						
YSN-8	SPT	28.50			29.73	0.27	66.31	40	19	21	CL						
YSN-9	SPT	6.00			24.28	0.00	88.96	58	21	37	CH						
YSN-9	SPT	9.00			27.13	1.99	70.62	46	17	29	CL						
YSN-9	UD	10.00	2.556	1.76	15.49	3.17	9.55		NP		SM						
YSN-9	SPT	12.00			23.90	0.00	80.59	40	17	23	CL						
YSN-9	SPT	18.00			26.19	0.00	80.91	44	19	25	CL						
YSN-9	SPT	21.00			25.57	0.00	73.59	42	17	25	CL						
YSN-9	SPT	24.00			24.69	0.00	78.74	45	16	29	CL						
YSN-13	SPT	1.50			21.97	0.38	81.31	48	20	28	CL						
YSN-13	UD	5.50	2.719	1.88	20.86	0.41	71.34	38	17	21	CL	0.44	0.87				
YSN-13	SPT	6.00			22.84	1.37	86.04	38	18	20	CL						
YSN-13	SPT	12.00			22.55	1.60	84.91	47	18	29	CL						
YSN-13	SPT	15.00			29.13	0.56	77.77	45	19	26	CL						
YSN-13	SPT	18.00			29.24	0.00	78.91	48	20	28	CL						

Figure 3.4. Lab Reports.

According to the boring log and lab reports (YSN8) , the Layer 1 is taken from the ground level +0,00 to -8,00 m and the Layer 2 is taken from – 8,00 m to – 52,00 m. Ground water level starts from -3.5 meters.

For soil layer1; because the water content of the layer tends to have values near the plastic limit values, undrained cohesion was assumed as 100 kPa. Young` s Modulus` assumption was made according to the formula (Bowles, 1997)

$$E= (100 \text{ to } 500) c_u \quad (3.1)$$

as 15,000 kN/m².

For soil layer2, in order to get undrained cohesion the following formula (Terzaghi & Peck, 1967) was used,

$$c_u= 6.25N \quad (3.2)$$

and the undrained cohesion was found as 90 kPa. Young` s Modulus` assumption was made according to the formula given by Bowles (1997) as $E = c_u * 250 = 22,500 \text{ kN/m}^2$.

Poisson ratio is generally taken between 0.3 and 0.35 for soils as drained parameters in undrained analysis and 0.2 for concrete in Plaxis Teaching Manuals. Saturated soil`s poisson ratio 0.5-or values around 0.5- was used by simulation; however , finite element code gave error during the calculation phase consistent with the teaching manual, so for both layers it was assumed 0.33. In addition, Young` s Modulus of concrete was assumed as 25,000,000 kN/m² according to the formula given by TS500,

$$E=5000 \times \sqrt{f_{ck}} \quad (3.3)$$

because C25 type of concrete was used in the pile.

Table 3.1 lists the parameters of material parameters according to the boring log and lab reports, seen by Figure 3.2, Figure 3.3 and Figure 3.4.

Table 3.1. Material Properties of the FEM Model.

Name	Depth	Type	Model	γ_{unsat}	γ_{sat}	ν	E_{ref}	c_{ref}
	m			kN/m^3	kN/m^3	-	kN/m^2	kN/m^2
Soil layer 1	0-8	Undrained	M-C	19	20	0.33	15000	100
Soil layer 2	8-52	Undrained	M-C	19	20	0.33	22500	90
Concrete	n/a	Elastic	L-E	24	24	0.2	2.50E+07	-

The interface element is applied between piles and soil, the strength reduction factor R_{inter} is assumed to be 0.7. The definition of parameters can be referred to Section 2.3.2.

3.2.1.2. Results. In Figure 3.5 and Figure 3.6 total displacements of case 1 are given. As can be seen there, maximum deformation observed is 17.74 mm. The deformations observed indicates that the soil up to 10 m is effected by the movement. As can be seen in the following Figure 3.7, maximum vertical deformation observed is the same as total displacement value 17.74 mm which means there isn't any distinctive horizontal displacement.

In the following Figure 3.8 and Figure 3.9 displacement behaviors of numerically simulated and site measured case 1 under loading are given respectively. These two data are combined on the same template to show the similarities and differences between them which is given in Figure 3.10. The calculated vertical settlement is approximately three times bigger than the measured vertical displacement. Shape of the mobilized displacement curve appears in good agreement with the measured one. The overall displacement values of the predicted displacement curve ,especially in the final loading stage, tends to be overestimated for measurement curve, which is because of the stiffer displacement behavior of the predicted case under loading.

In Appendix A, additional numerical simulation results are given.

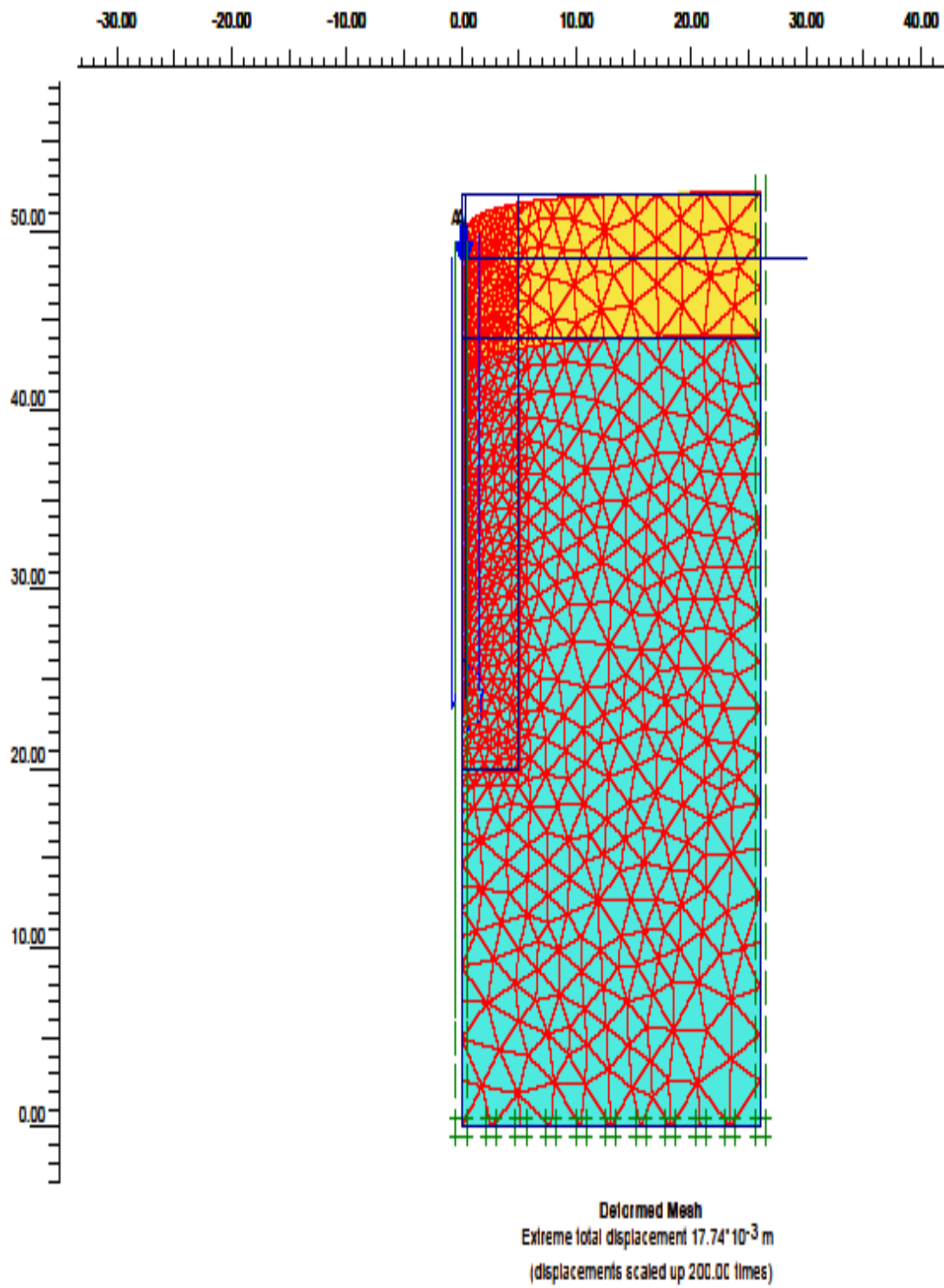


Figure 3.5. Deformed mesh of case1.

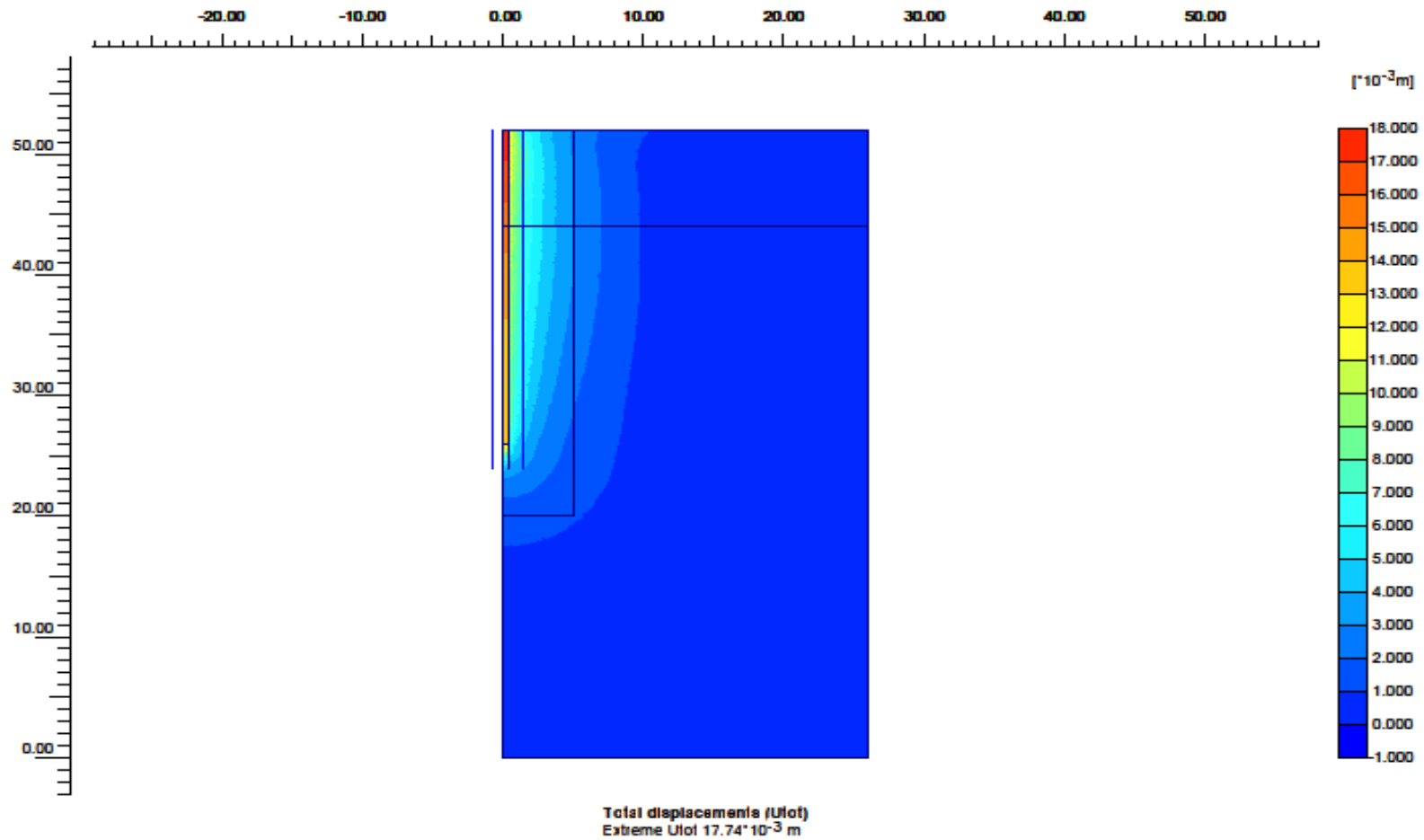


Figure 3.6. Total displacements of case1.

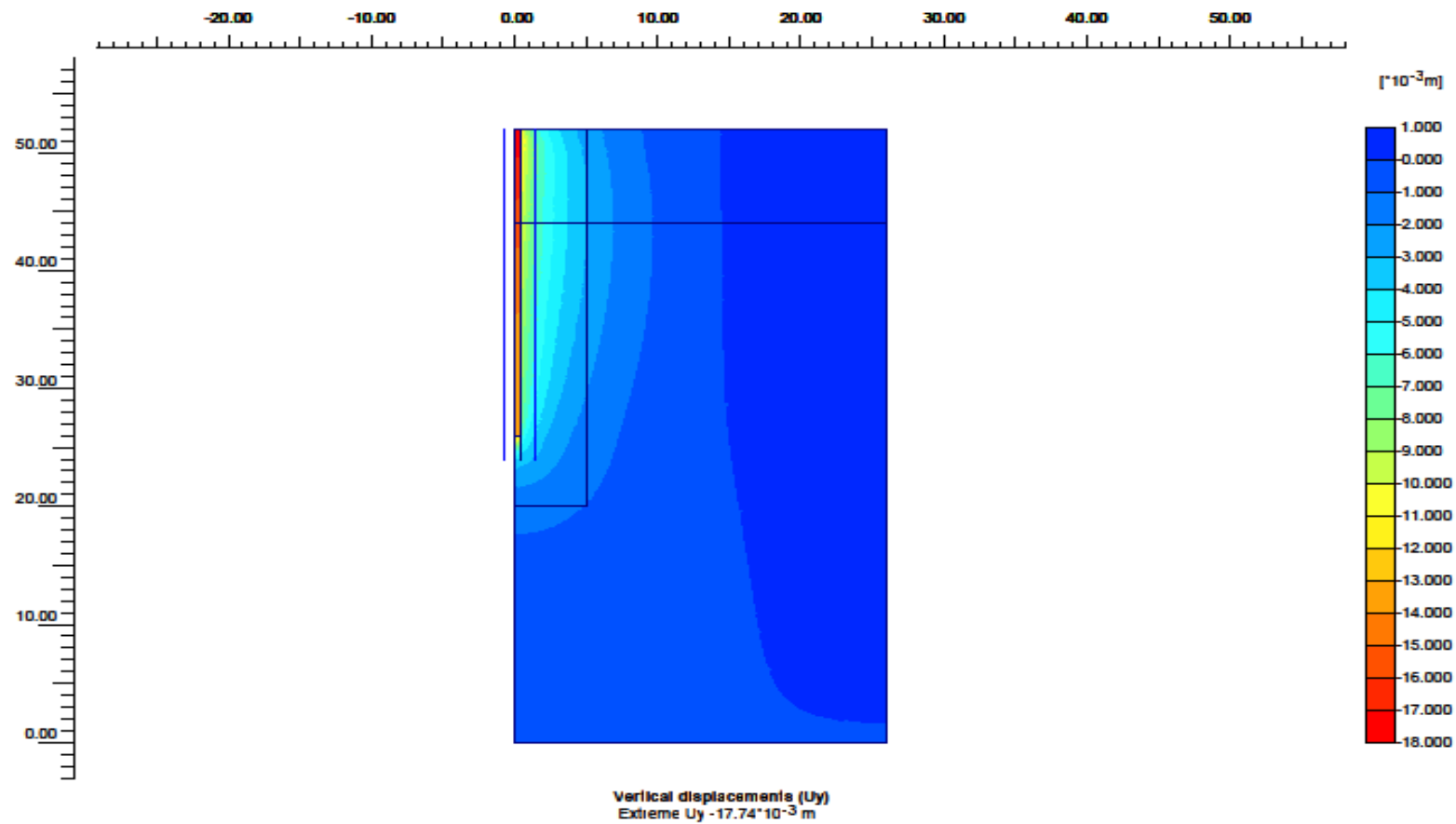


Figure 3.7. Vertical displacements of case1.

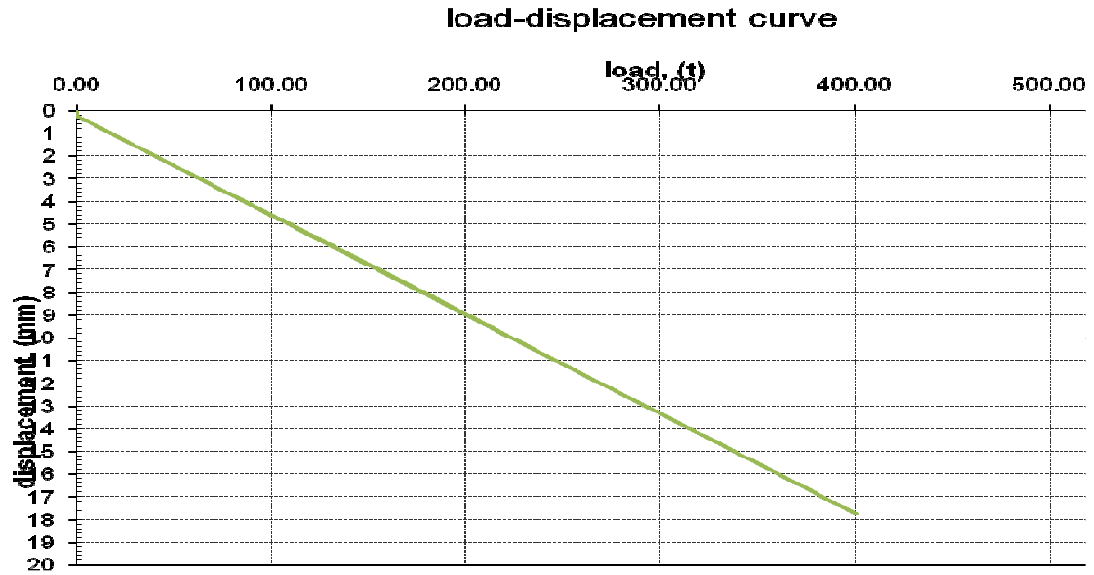


Figure 3.8. Load(tons) versus Displacement(mm) curve of case1.

3.2.1.3. Site Test Results. In Appendix A, additional site measured experimental data are given.

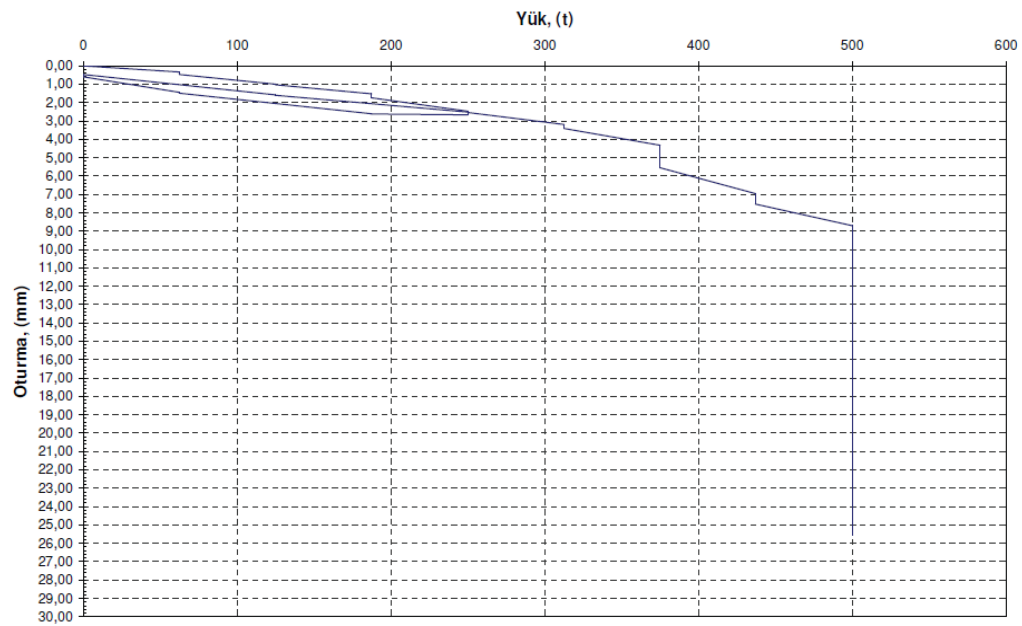


Figure 3.9. Load versus Displacement curve of case1.

3.2.1.4. Site Measured and Numerically Simulated Load versus Displacement Curve of Case1. Both curves are given in the following figure



Figure 3.10. Site measured(blue) and numerically simulated (green) load versus displacement curve of case 1.

3.2.1.5. Stress Distribution Along the Pile Shaft. Stress distribution is given in the following figure.

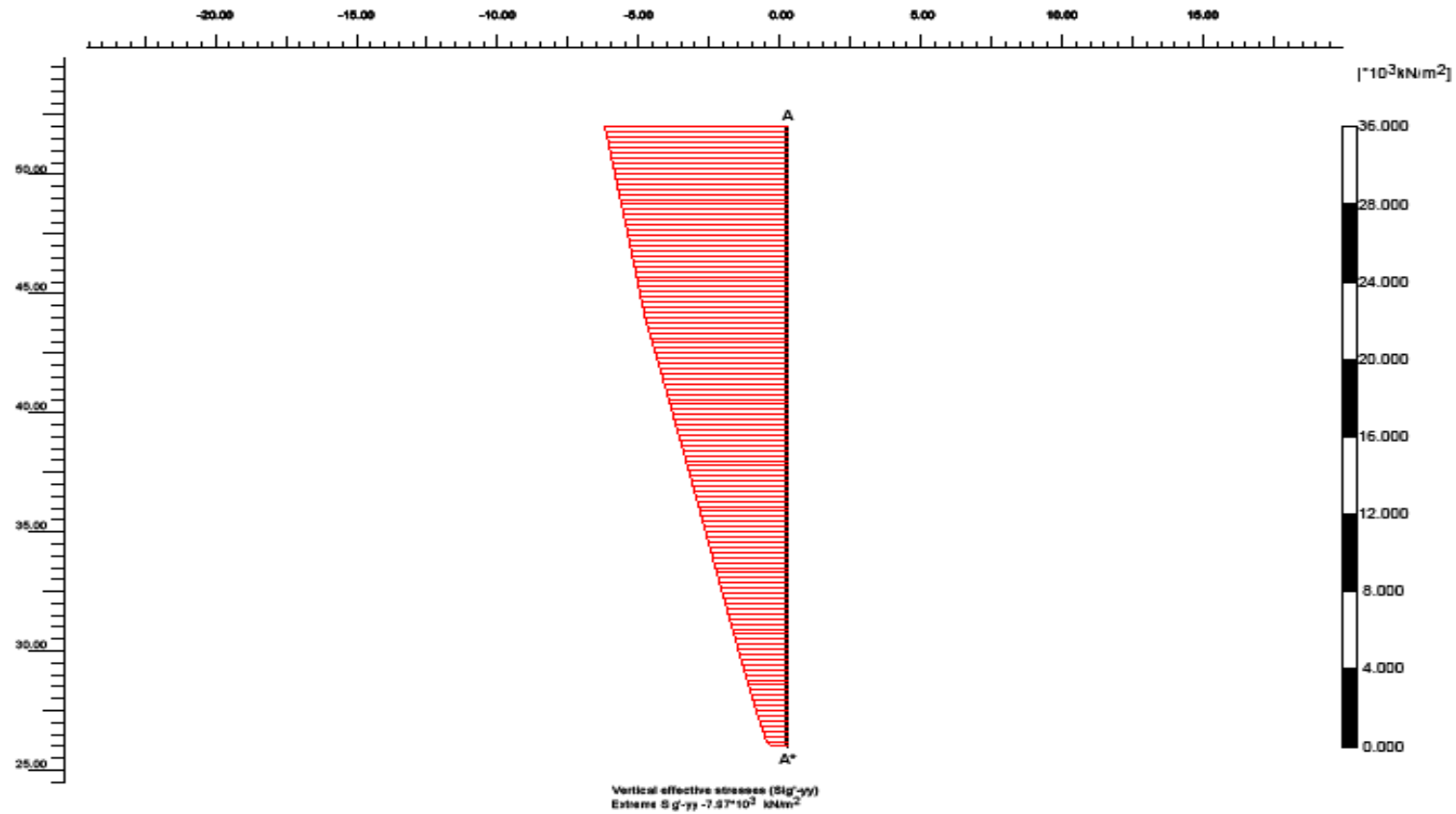


Figure 3.11. Vertical stress distribution along the pile shaft cross-section from the top to the bottom.

Obtained from Figure 3.11, in – 8 meters stress is -6204.29 kN/m^2 and in – 26 meters it is -678.824 kN/m^2 and -7973.72 kN/m^2 at the top.

Combining the data from Figure 3.11 gives the curve given in Figure 3.12 in which decreasing stress distribution with depth can be seen.

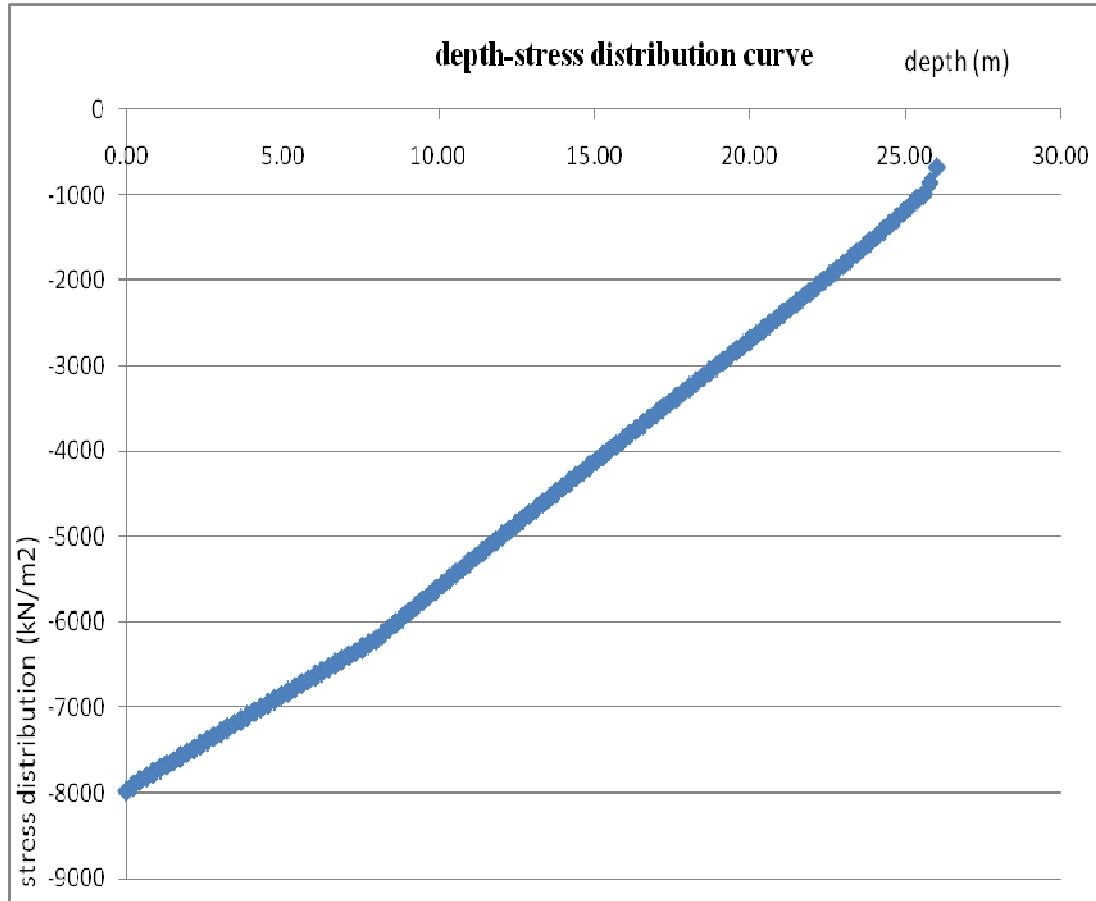


Figure 3.12. Depth(m) versus Stress(kN/m^2) distribution curve of pile shaft cross-section from the top to the bottom.

Stress change within a specific depth can be obtained from the Figure 3.12 and used to calculate cohesion within this specific depth by dividing the stress change to frictional resistance area. The result of the calculations is given in Table 3.2, in which the difference between calculated and assumed cohesion values can be seen. This difference stems from that I assumed an average cohesion value for a specific depth with the help of literature data while effective stress was increasing within depth.

Table 3.2. Calculated and assumed cohesion values within depth.

depth	stress change	$c_{\text{calculated}}$	c_{assumed}
m	kN/m^2	kN/m^2	kN/m^2
0-8	7963.72-6204.29	44	100
8-26	6204.29-678.824	61	90

3.3. Case Study 2

3.3.1. FEM Study on Case Study 2

The idealized finite element model of static pile load test is illustrated in Figure 3.13 which is the model of conventional top load test. The test is simulated in Mohr Coulomb soil model with 15 node elements; the mesh is generated axisymmetrically, with the boundary of 28 meters wide and 56 meters deep. The test pile is in the same size of diameter of 0,8 meters and length of 28 meters. To ensure the precision of the calculation, the meshes of an area of 5 meters wide, 36 meters deep around the test pile are refined.

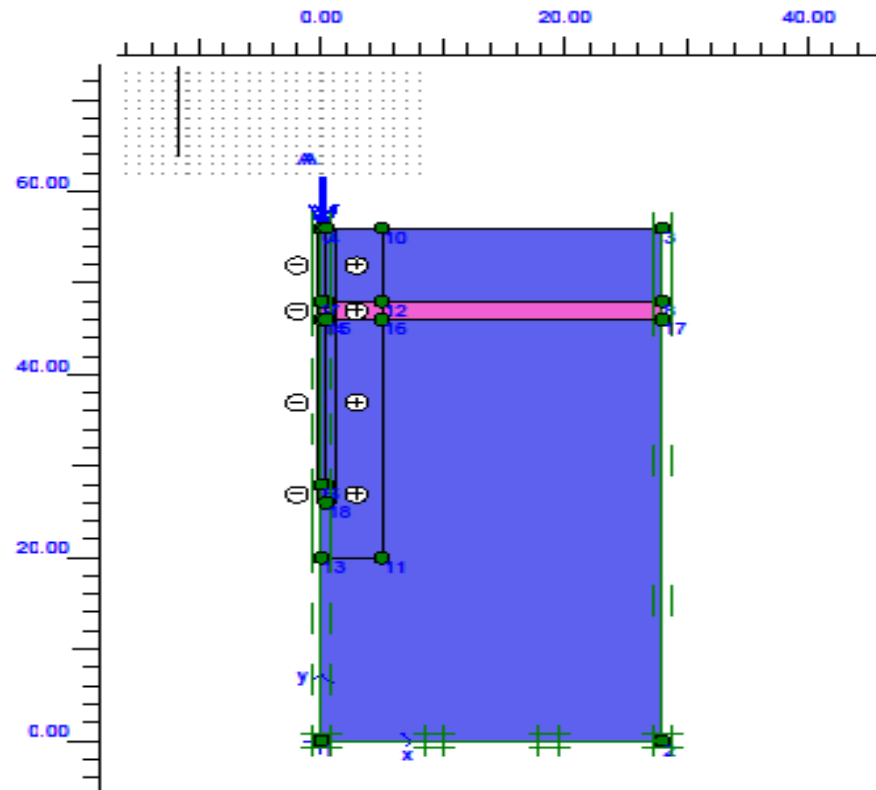


Figure 3.13. Finite element model of case2.

 JAS										SONDAJ NO / BORING NO : SK 1 SAYFA NO / PAGE NO : 1 SONDÖR / DRILLER : Suat GENÇ											
SONDAJ LOGU / BORING LOG																					
PROJE ADI / PROJECT NAME : SİNPAS SONDAJ YERİ / BORING LOCATION : DEMİRTAŞ/BURSA KİLOMETRE / KILOMETER : Muhafaza Borusu Derinliği / Casing Depth : 15,00 m SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07 SONDAJ KOTU / ELEVATION : Koordinat/Coordinate(N-S) X : YER ALTI SUYU / GRANDWATER : m Koordinat/Coordinate(N-S) Y :																					
DERİNLİK(m) DEPTH(m)		MANEVR RUN	NUMUNE QİSİ SAMPLE TYPE	STANDART PENETRASYON DENEYİ STANDART PENETRATION TEST				N ₆₀	GRAFIK GRAPH						JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION	PROFİL PROFILE	DAYANIMLILIK STRENGTH	AYRILMA WEATHERING	KIRIK/30 cm FRACTURE/30 cm	KAROT % CORE RECOVERY %	ROD % ROD %
				DARBE SAYISI NUMBER OF BLOWS																	
				0-15 cm	15-30 cm	30-45 cm			10	20	30	40	50	60							
0,50															Dolgu Malzemesi						
1,70		1,50													Az Siltli KİL Siyahimsi renkli, orta katı kıvamlı, Az Siltli KİL						
			SPT-1	3	3	4	7														
		1,95																			
		3,00																			
			SPT-2	5	5	7	12														
		3,45																			
		4,50													Az Çakıllı Kumlu Siltli KİL Sarımsı renkli, orta katı-katı-çok katı kıvamlı, seyrek çakıllı, Az Çakıllı Kumlu Siltli KİL						
			SPT-3	7	8	10	18														
		4,95																			
		6,00																			
			SPT-4	5	3	7	10														
		6,45																			
7,00																					
		7,50																			
			SPT-5	7	7	11	18														
		7,95																			
		9,00													Az Kili Siltli KUM Siyahimsi renkli, orta sıkı - sıkı, Az Kili Siltli KUM						
			SPT-6	11	18	26	44														
		9,45																			
10,00																					
DAYANIMLILIK-STRENGTH				AYRILMA-WEATHERING				İNCİ DANELİ-FINE GRAINED				İRİ DANELİ-COARSE GRAINED									
I DAYANIMLI STRONG				I TAZE FRESH				N=0-2 ÇOK YUMUŞAK V.SOFT				N=0-4 Çok Ganyak V.LOOSE									
II ORTA DAYANIMLI M.STRONG				II AZ AYRIŞMIŞ S.WEATHERED				N=3-4 YUMUŞAK SOFT				N=5-10 Ganyak LOOSE									
III ORTA ZAYIF M.WEAK				III ORTA DERAYL M.WEATHERED				N=5-8 ORTA KATI M.STIFF				N=11-30 Orta Sıkı M.DENSE									
IV ZAYIF WEAK				IV ÇOK AYRIŞMIŞ H.WEATHERED				N=9-15 KATI STIFF				N=31-50 Sıkı DENSE									
V ÇOK ZAYIF V.WEAK				V TAMAMEN AYR. C.WEATHERED				N=15-30 ÇOK KATI V.STIFF				N=50 Çok Sıkı V.DENSE									
N=30 SERT HARD																					
KAYA KALİTESİ ROD %				KIRIK-FRACTURE/ 30 cm				ORANLAR-PROPORTIONS													
% 0-25 ÇOK ZAYIF V.POOR				<1 SEYREK WIDE(W)				% 5- PEK AZ SLIGHTLY				% 5- PEK AZ SLIGHTLY									
% 25-50 ZAYIF POOR				1-2 ORTA MODERATE(M)				% 5-15 AZ LITTLE				% 5-20 AZ LITTLE									
% 50-75 ORTA FAIR				2-10 SIK CLOSE(C)				% 15-35 ÇOK VERY				% 20-50 ÇOK VERY									
% 75-90 İYİ GOOD				10-20 ÇOK SIKI INTENSE(I)				% 35 > VE AND													
% 90-100 ÇOK İYİ EXCELLENT				>20 PARÇALI CRUSHED(Cr)																	
SPT : Standart Penetrasyon Deneyi				K : Karot Numunesi				JEOLJİ MÜHENDİSİ				İMZA									
SPT : Standart Penetrasyon Test				K : Core Sample				DRILLING ENGINEER				SING									
D : Örselenmiş Numune				P : Presiyometre Deneyi				Hacı BALIK													
D : Disturbed Sample				P : Pressuremeter Test				Jeoloji Mühendisi													
UD : Örselenmemiş Numune				VST : Vano Deneyi				Oda Sicil No:6794													
UD : Undisturbed Sample				VST : Vano Shear Test																	

Figure 3.14. Boring Log Page 1.

										SONDAJ NO / BORING NO : SK 22 SAYFA NO / PAGE NO : 2 SONDÖR / DRILLER : Suat GENÇ																			
SONDAJ LOGU / BORING LOG																													
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SONDAJ YERİ / BORING LOCATION : DEMİRTAŞ/BURSA																													
KİLOMETRE / KILOMETER :										Muhafaza Borusu Derinliği / Casing Depth : 15,00 m																			
SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m										Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07																			
SONDAJ KOTU / ELEVATION :										Kordinat/Coordinate(N-S) X :																			
YER ALTI SUYU / GRANDWATER : m										Kordinat/Coordinate(N-S) Y :																			
DERİNLİK(m) DEPTH(m)		MANEVRA RUN		NUMUNE QİNSİ SAMPLE TYPE		DARBE SAYISI NUMBER OF BLOWS				GRAFIK GRAPH				JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION				PROFİL PROFILE		DAYANIMLILIK STRENGTH		AYRISMA WEATHERING		KIRIK/30 cm FRACTURE/30 cm		KAYI CORE RECOVERY %		ROD %	
						0-15 cm 15-30 cm 30-45 cm				N ₆₀ 10 20 30 40 50 60																			
10,50				SPT-7		5 6 9 15																							
10,95																													
12,00				SPT-8		6 7 11 18																							
12,45																													
13,50																													
13,95				SPT-9		6 5 8 13																							
15,00		15,00																											
				SPT-10		5 6 8 14																							
15,45																													
16,50																													
				SPT-11		7 9 11 20																							
16,95																													
18,00																													
				SPT-12		4 5 8 13																							
18,45																													
19,50																													
				SPT-13		6 6 8 14																							

 JAS LTD. ŞTİ.										SONDAJ NO / BORING NO : SK 1 SAYFA NO / PAGE NO : 3 SONDÖR / DRILLER : Suat GENÇ									
SONDAJ LOGU / BORING LOG																			
PROJE ADI / PROJECT NAME : SİMPAŞ																			
SONDAJ YERİ / BORING LOCATION : DEMİRTAŞ/BURSA																			
KİLOMETRE / KILOMETER :										Muhafaza Borusu Derinliği / Casing Depth : 15,00 m									
SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m										Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07									
SONDAJ KOTU / ELEVATION :										Koordinat/Coordinate(N-S) X :									
YER ALTI SUYU / GRANDWATER :										Koordinat/Coordinate(N-S) Y :									
STANDART PENETRASYON DENEYİ STANDART PENETRATION TEST																			
DERİNLİK(m) DEPTH(m)	MANEVR RUN	NUMUNE QMS/ SAMPLE TYPE	DARBE SAYISI NUMBER OF BLOWS			N ₆₀	GRAFIK GRAPH					JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION	PROFİL PROFILE	DAYANIMLILIK STRENGTH	AYRISMA WEATHERING	KIRIK/30 cm FRACTURE/30 cm	KAROT % CORE RECOVERY %	ROD % ROD %	
			0-15 cm	15-30 cm	30-45 cm		10	20	30	40	50								60
21,00												Az Kumlu Siltli KİL Gimsi renkli, katı - çok katı kvrmlı, Az Kumlu Siltli KİL							
	SPT-14		6	7	9	16													
21,45																			
22,50		SPT-15	10	7	12	19													
22,95																			
24,00																			
24,50		SPT-16	6	8	8	16													
24,45																			
25,50																			
		SPT-17	8	9	11	20													
25,95																			
27,00																			
		SPT-18	6	6	12	18													
27,45																			
28,50																			
		SPT-19	8	13	19	32													
28,95																			
DAYANIMLILIK-STRENGTH					AYRISMA-WEATHERING					İNCE DANELİ-FINE GRAINED					İRİ DANELİ-COARSE GRAINED				
I DAYANIMLI STRONG					I TAZE FRESH					N=2 ÇOK YUMUŞAK V.SOFT					N=0-4 Çok Gariyok V.LOOSE				
II ORTA DAYANIMLI M.STRONG					II AZ AYRIŞMIŞ S.WEATHERED					N=3-4 YUMUŞAK SOFT					N=5-10 Gariyok LOOSE				
III ORTA ZAYIF M.WEAK					III ORTA DERLAYR. M.WEATHERED					N=5-8 ORTA KATI M.STIFF					N=11-30 Orta Siki M.DENSE				
IV ZAYIF WEAK					IV ÇOK AYRIŞMIŞ H.WEATHERED					N=9-15 KATI STIFF					N=31-60 Siki DENSE				
V ÇOK ZAYIF V.WEAK					V TAMAMEN AYR. C.WEATHERED					N=16-30 ÇOK KATI V.STIFF					N=50 Çok Siki V.DENSE				
										N=30 SERT HARD									
KAYA KALİTESİ RQD %					KIRIK-FRACTURE/ 30 cm					ORANLAR-PROPORTIONS									
% 0-25 ÇOK ZAYIF V.POOR					<1 SEYREK WIDE(W)					% 5- PEK AZ SLIGHTLY					% 5- PEK AZ SLIGHTLY				
% 25-50 ZAYIF POOR					1-2 ORTA MODERATE(M)					% 5-15 AZ LITTLE					% 5-20 AZ LITTLE				
% 50-75 ORTA FAIR					2-10 SIK CLOSE(C)					% 15-35 ÇOK VERY					% 20-50 ÇOK VERY				
% 75-90 İYİ GOOD					10-20 ÇOK SIK INTENSE(I)					% 35 > VE AND									
% 90-100 ÇOK İYİ EXCELLENT					>20 PARÇALI CRUSHED(Cr)														
SPT : Standart Penetrasyon Deneyi K : Karot Numunesi										JEOLOJİ MÜHENDİSİ					İMZA				
SPT : Standart Penetrasyon Test K : Core Sample										DRILLING ENGINEER					SING				
D : Örselenmiş Numune P : Presiyometre Deneyi										Hacı BALIK									
D : Disturbed Sample P : Presiyometre Test										Jeoloji Mühendisi									
UD : Örselenmemiş Numune VST : Vane Deneyi										Oda Sicil No:6794									
UD : Undisturbed Sample VST : Vane Shear Test																			

Figure 3.16. Boring Log Page 3.

 JAS LTD.ŞTİ.										SONDAJ NO / BORING NO : SK 1 SAYFA NO / PAGE NO : 3 SONDÖR / DRILLER : Suat GENÇ																													
SONDAJ LOGU / BORING LOG																																							
PROJE ADI / PROJECT NAME : SİMPAŞ																																							
SONDAJ YERİ / BORING LOCATION : DEMİRTAŞ/BURSA																																							
KİLOMETRE / KILOMETER :										Muhafaza Borusu Derinliği / Casing Depth : 15,00 m																													
SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m										Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07																													
SONDAJ KOTU / ELEVATION :										Koordinat/Coordinate(N-S) X :																													
YER ALTI SUYU / GRANDWATER : m										Koordinat/Coordinate(N-S) Y :																													
DERİNLİK(m) DEPTH(m)		MANEVR RUN		NUMERİK SAMPLE TYPE		STANDART PENETRASYON DENEYİ STANDART PENETRATION TEST				JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION						PROFİL PROFILE		DAYANIMLILIK STRENGTH		AYRISMA WEATHERING		KIRILMA/30 cm FRACTURE/30 cm		KAYIP % CORE RECOVERY %		ROD % ROD %													
						DARBE SAYISI NUMBER OF BLOWS																																	
						0-15 cm 15-30 cm 30-45 cm N ₆₀				10 20 30 40 50 60																													
30,45				SPT-20		9 13 15 28																																	
33,00																																							
33,45				SPT-21		9 11 13 24																																	
36,00																																							
36,45				SPT-22		9 10 15 25																																	
37,00																																							
39,00																																							
39,45				SPT-23		33 41 42 83																																	
40,00																																							
DAYANIMLILIK-STRENGTH										AYRISMA-WEATHERING										İNCE DANELİ-FINE GRAINED										İRİ DANELİ-COARSE GRAINED									
I DAYANIMLI STRONG										I TAZE FRESH										N-0-2 ÇOK YUMUŞAK V.SOFT										N-0-4 Çok Gariyak V.LOOSE									
II ORTA DAYANIMLI M.STRONG										II AZ AYRIŞMIŞ S.WEATHERED										N-3-4 YUMUŞAK SOFT										N-5-10 Gariyak LOOSE									
III ORTA ZAYIF M.WEAK										III ORTA DERLAYR. M.WEATHERED										N-5-8 ORTA KATI M.STIFF										N-11-30 Orta Siki M.DENSE									
IV ZAYIF WEAK										IV ÇOK AYRIŞMIŞ H.WEATHERED										N-9-15 KATI STIFF										N-31-60 Siki DENSE									
V ÇOK ZAYIF V.WEAK										V TAMAMEN AYR. C.WEATHERED										N-16-30 ÇOK KATI V.STIFF										N-60 Çok Siki V.DENSE									
KAYA KALİTESİ ROD %										KIRIK-FRACTURE/ 30 cm										ORANLAR-PROPORTIONS																			
% 0-25 ÇOK ZAYIF V.POOR										≤1 SEYREK WIDE(W)										% 5- PEK AZ SLIGHTLY																			
% 25-50 ZAYIF POOR										1-2 ORTA MODERATE(M)										% 5-15 AZ LITTLE										% 5- PEK AZ SLIGHTLY									
% 50-75 ORTA FAIR										2-10 SIK CLOSE(C)										% 15-35 ÇOK VERY										% 5-20 AZ LITTLE									
% 75-90 İYİ GOOD										10-20 ÇOK SIK INTENSE(I)										% 35 > VE AND										% 20-50 ÇOK VERY									
% 90-100 ÇOK İYİ EXCELLENT										>20 PARÇALI CRUSHED(Cr)																													
SPT : Standart Penetrasyon Deneyi										K : Karot Numunesi										JEOLJİ MÜHENDİSİ										İMZA									
SPT : Standart Penetrasyon Test										K : Core Sample										DRILLING ENGINEER										SİNG									
D : Örselenmiş Numune										P : Pressiometre Deneyi																													
D : Disturbed Sample										VST : Vano Deneyi																													
UD : Örslenmemiş Numune										VST : Vano Shear Test																													
UD : Undisturbed Sample																				Hacı BALIK																			
																				Jeoloji Mühendisi																			
																				Oda Sicil No:6794																			

Figure 3.17. Boring Log Page 4.

 JAS LTD. ŞTİ.										SONDAJ NO / BORING NO : SK 1 SAYFA NO / PAGE NO : 3 SONDÖR / DRILLER : Suat GENÇ																													
SONDAJ LOGU / BORING LOG																																							
PROJE ADI / PROJECT NAME : SİNPAS																																							
SONDAJ YERİ / BORING LOCATION : DEMİRTAŞ/BURSA																																							
KİLOMETRE / KILOMETER :										Muhafaza Borusu Derinliği / Casing Depth : 15,00 m																													
SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m										Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07																													
SONDAJ KOTU / ELEVATION :										Kordinat/Coordinate(N-S) X :																													
YER ALTı SUYU / GRANDWATER : m										Kordinat/Coordinate(N-S) Y :																													
DERİNLİK(m) DEPTH(m)	MANEVR RUN	NUMUNE ÇİNSİ SAMPLE TYPE	STANDART PENETRASYON DENEYİ STANDART PENETRATION TEST				N ₆₀	GRAFIK GRAPH						JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION	PROFİL PROFILE	DAYANIMLILIK STRENGTH	AYRISMA WEATHERING	KIRIK/30 cm FRACTURE/30 cm	KAYIPI % LOSS %	KURTULUŞ % RECOVERY %																			
			0-15 cm	15-30 cm	30-45 cm	45-60 cm		60-75 cm	75-90 cm	90-105 cm	105-120 cm	120-135 cm	135-150 cm																										
42,00																																							
42,30		SPT-24	11	13	17	30																																	
42,45																																							
45,00																																							
45,45		SPT-25	19	20	23	43																																	
48,00																																							
48,45		SPT-26	14	21	31	52																																	
DAYANIMLILIK-STRENGTH										AYRISMA-WEATHERING										İNCE DANELİ-FINE GRAINED										KABA DANELİ-COARSE GRAINED									
I DAYANIMLI STRONG										I TAZE FRESH										N:0-2 ÇOK YUMUŞAK V.SOFT										N:0-4 ÇOK GARIYAK V.LOOSE									
II ORTA DAYANIMLI M.STRONG										II AZ AYRIŞMIŞ S.WEATHERED										N:3-4 YUMUŞAK SOFT										N:5-10 GARIYAK LOOSE									
III ORTA ZAYIF M.WEAK										III ORTA DERLAYR M.WEATHERED										N:5-8 ORTA KATI M.STIFF										N:11-30 ORTA SIKI M.DENSE									
IV ZAYIF WEAK										IV ÇOK AYRIŞMIŞ H.WEATHERED										N:9-15 KATI STIFF										N:31-50 SIKI DENSE									
V ÇOK ZAYIF V.WEAK										V TAMAMEN AYRIL C.WEATHERED										N:15-30 ÇOK KATI V.STIFF										N:50 ÇOK SIKI V.DENSE									
KAYA KALİTESİ RQD %										KIRIK-FRACTURE/ 30 cm										ORANLAR-PROPORTIONS																			
% 0-25 ÇOK ZAYIF V.POOR										<1 SEYREK WIDE(W)										% 5- PEK AZ SLIGHTLY										% 5- PEK AZ SLIGHTLY									
% 25-50 ZAYIF POOR										1-2 ORTA MODERATE(M)										% 5-15 AZ LITTLE										% 5- AZ LITTLE									
% 50-75 ORTA FAIR										2-10 SIK CLOSE(C)										% 15-35 ÇOK VERY										% 20-50 ÇOK VERY									
% 75-90 İYİ GOOD										10-20 ÇOK SIK INTENSE(I)										% 35- VE AND																			
% 90-100 ÇOK İYİ EXCELLENT										>20 PARÇALI CRUSHED(Cr)																													
SPT : Standart Penetrasyon Deneyi										K : Karot Numunesi										JEOLJİ MÜHENDİSİ										İMZA									
SPT : Standart Penetrasyon Test										K : Core Sample										DRILLING ENGINEER										SİNG									
D : Örselenmiş Numune										P : Presiyometre Deneyi										Hacı BALIK																			
D : Disturbed Sample										P : Presiyometre Test										Jeoloji Mühendisi																			
UD : Örselenmemiş Numune										VST : Vane Deneyi										Oda Sicil No:6794																			
UD : Undisturbed Sample										VST : Vane Shear Test																													

Figure 3.18. Boring Log Page 5.

		SONDAJ NO / BORING NO : SK 1													
		SAYFA NO / PAGE NO : 3													
SONDAJ LOGU / BORING LOG		SONDÖR / DRILLER : Suat GENÇ													
PROJE ADI / PROJECT NAME : SİNPAS															
SONDAJ YERİ / BORING LOCATION : DEMİRTAS/BURSA															
KİLOMETRE / KILOMETER :		Muhafaza Borusu Derinliği / Casing Depth : 15,00 m													
SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m		Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07													
SONDAJ KOTU / ELEVATION :		Kordinat/Coordinate(N-S) X :													
YER ALTI SUYU / GRANDWATER : m		Kordinat/Coordinate(N-S) Y :													
DERİNLİK(m) DEPTH(m)	MANEVR RUN	STANDART PENETRASYON DENEYİ STANDARD PENETRATION TEST				JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION	PROFİL PROFILE	DAYANIMLILIK STRENGTH	AYRISMA WEATHERING	KIRIK/30 cm FRACTURE/30 cm	KAROT % CORE RECOVERY %	ROD % ROD %			
		NUMUNE QNSİ SAMPLE TYPE	DARBE SAYISI NUMBER OF BLOWS										GRAFIK GRAPH		
		0-15 cm	15-30 cm	30-45 cm	N ₆₀	10 20 30 40 50 60									
51,00															
	SPT-27	15	25	42	67										
51,45															
54,00															
	SPT-28	13	25	31	56										
54,45															
57,00															
	SPT-29	15	28	36	64										
57,45															
DAYANIMLILIK-STRENGTH				AYRISMA-WEATHERING				İNCİ DANELİ-FINE GRAINED				İRİ DANELİ-COARSE GRAINED			
I DAYANIMLI STRONG				I TAZE FRESH				N-0-2 ÇOK YUMUŞAK V.SOFT				N-0-4 Çok Ganyak V.LOOSE			
II ORTA DAYANIMLI M.STRONG				II AZ AYRIŞMIŞ S.WEATHERED				N-3-4 YUMUŞAK SOFT				N-5-10 Ganyak LOOSE			
III ORTA ZAYIF M.WEAK				III ORTA DELAYR. M.WEATHERED				N-5-8 ORTA KATI M.STIFF				N-11-30 Orta Siki M.DENSE			
IV ZAYIF WEAK				IV ÇOK AYRIŞMIŞ H.WEATHERED				N-9-15 KATI STIFF				N-31-60 Siki DENSE			
V ÇOK ZAYIF V.WEAK				V TAMMEN AYR. C.WEATHERED				N-15-30 ÇOK KATI V.STIFF				N-60 Çok Siki V.DENSE			
KAYA KALİTESİ ROD %				KIRIK-FRACTURE/ 30 cm				ORANLAR-PROPORTIONS							
% 0-25 ÇOK ZAYIF V.POOR				<1 SEYREK WIDE(W)				% 5- PEK AZ SLIGHTLY				% 5- PEK AZ SLIGHTLY			
% 25-50 ZAYIF POOR				1-2 ORTA MODERATE(M)				% 5-15 AZ LITTLE				% 5-20 AZ LITTLE			
% 50-75 ORTA FAIR				2-10 SIK CLOSE(C)				% 15-35 ÇOK VERY				% 20-50 ÇOK VERY			
% 75-90 İYİ GOOD				10-20 ÇOK SIK INTENSE(I)				% 35 > VE AND							
% 90-100 ÇOK İYİ EXCELLENT				>20 PARÇALI CRUSHED(Cr)											
SPT : Standart Penetrasyon Deneyi K : Karot Numunesi				SPT : Standart Penetrasyon Test K : Core Sample				JEOLJİ MÜHENDİSİ				İMZA			
D : Örselenmiş Numune P : Presiyometre Deneyi				D : Disturbed Sample P : Presuremeter Test				DRILLING ENGINEER				SING			
UD : Örselenmemiş Numune VST : Vane Deneyi				UD : Undisturbed Sample VST : Vane Shear Test				Hacı BALIK							
								Jeoloji Mühendisi							
								Oda Sicil No:6794							

Figure 3.19. Boring Log Page 6.

JAS										SONDAJ NO / BORING NO : SK 1										
										SAYFA NO / PAGE NO : 3										
SONDAJ LOGU / BORING LOG										SONDÖR / DRILLER : Suat GENÇ										
PROJE ADI / PROJECT NAME : SİNPAS																				
SONDAJ YERİ / BORING LOCATION : DEMİRTAŞ/BURSA																				
KİLOMETRE / KILOMETER :										Muhalaza Borusu Derinliği / Casing Depth : 15,00 m										
SONDAJ DERİNLİĞİ / BORING DEPTH : 60,40 m										Baş. Ve Bitiş Tarihi/Start and Finish Date : 03.11.07										
SONDAJ KOTU / ELEVATION :										Koordinat/Coordinate(N-S) X :										
YER ALTı SUYU / GRANDWATER : m										Koordinat/Coordinate(N-S) Y :										
DERİNLİK(m) DEPTH(m)	MANEVR RUN	NUMUNE QNS SAMPLE TYPE	STANDART PENETRASYON DENEYİ STANDARD PENETRATION TEST										JEOTEKNİK TANIMLAMA GEOTECHNICAL DESCRIPTION	PROFİL PROFILE	DAYANIMLIK STRENGTH	AYRISMA WEATHERING	KIRILMA FRACTURE	KAROT % RECOVERY %	ROD %	
			DARBE SAYISI NUMBER OF BLOWS				GRAFIK GRAPH													
			0-15 cm	15-30 cm	30-45 cm	N ₆₀	10	20	30	40	50	60								
	60,00	SPT-30	19	41	50/10										Az Çakıllı Kumlu Siltli Kil					
															Kuyu Sonu: 60,40 m.					
DAYANIMLIK-STRENGE					AYRISMA-WEATHERING					İNCE DANELİ-FINE GRAINED					İRİ DANELİ-COARSE GRAINED					
I	DAYANIMLI	STRONG	I	TAZE	FRESH	N=02	ÇOK YUMUŞAK	V.SOFT	N=04	Çok Gıcık	V.LOOSE									
II	ORTA DAYANIMLI	M.STRONG	II	AZ AYRIŞMIŞ	S.WEATHERED	N=34	YUMUŞAK	SOFT	N=5-10	Gıcık	LOOSE									
III	ORTA ZAYIF	M.WEAK	III	ORTA DEĞAYIRL	M.WEATHERED	N=5-8	ORTA KATI	M.STIFF	N=11-30	Orta Siki	M.DENSE									
IV	ZAYIF	WEAK	IV	ÇOK AYRIŞMIŞ	H.WEATHERED	N=9-15	KATI	STIFF	N=31-60	Siki	DENSE									
V	ÇOK ZAYIF	V.WEAK	V	TAMAMEN AYIRL	C.WEATHERED	N=15-30	ÇOK KATI	V.STIFF	N=50	Çok Siki	V.DENSE									
						N=30	SERT	HARD												
KAYA KALİTESİ ROD %					KIRIK-FRACTURE/ 30 cm					ORANLAR-PROPORTIONS										
% 0-25	ÇOK ZAYIF	V.POOR	<1	SEYREK	WIDE(W)	% 5c	PEK AZ	SLIGHTLY	% 5c	PEK AZ	SLIGHTLY									
% 25-50	ZAYIF	POOR	1-2	ORTA	MODERATE(M)	% 5-15	AZ	LITTLE	% 5-20	AZ	LITTLE									
% 50-75	ORTA	FAIR	2-10	SİK	CLOSE(C)	% 15-35	ÇOK	VERY	% 20-50	ÇOK	VERY									
% 75-90	İYİ	GOOD	10-20	ÇOK SIK	INTENSE(I)	% 35 >	VE	AND												
% 90-100	ÇOK İYİ	EXCELLENT	>20	PARÇALI	CRUSHED(Cr)															
SPT : Standart Penetrasyon Deneyi SPT : Standart Penetrasyon Test D : Örstenmiş Numune D : Disturbed Sample UD : Örstenmemiş Numune UD : Undisturbed Sample										K : Karot Numunesi K : Core Sample P : Prossiyometre Deneyi P : Prossurometre Test VST : Vano Deneyi VST : Vano Shear Test										
JEOLOJİ MÜHENDİSİ DRILLING ENGINEER										İMZA SİNG										
Hacı BALIK Jeoloji Mühendisi Oda Sicil No:6794																				

Figure 3.20. Boring Log Page 7.

3.3.1.1. Soil Conditions and Design Parameters. Design parameters are obtained by incorporation of soil boring log which is seen in Figure 3.14, Figure 3.15, Figure 3.16, Figure 3.17, Figure 3.18, Figure 3.19 and Figure 3.20.

According to the boring log the Layer 1 is taken from the ground level +0,00 to -8,00 m , the Layer 2 is taken from – 8,00 m to – 10,00 m and Layer 3 which has the same properties with Layer 1 is taken from -10,00 m to -56,00 m. Ground water level starts from -16 meters.

For soil layer1; N is assumed as 14.5 and undrained cohesion was found as 90 kPa according to the formula (3.2). Young` s Modulus` assumption was made according to the formula (3.1) and found 30,000 kN/m².

For soil layer2, N is assumed as 31 and undrained cohesion was found as 190 kPa according to the formula (3.2). Young` s Modulus` assumption was made according to the formula given by Bowles, 1997,

$$E=500(N+15) \quad (3.4)$$

and found 25,000 kN/m².

Poisson ratio is generally taken between 0.3 and 0.35 for soils as drained parameters in undrained analysis and 0.2 for concrete in Plaxis Teaching Manuals. Saturated soil`s poisson ratio 0.5-or values around 0.5- was used by simulation; however , finite element code gave error during the calculation phase consistent with the teaching manual, so for all layers it was assumed 0.33. Young` s Modulus of concrete was assumed as 25,000,000 kN/m² according to the formula (3.3) because C25 type of concrete was used in the pile.

Table 3.3 lists the parameters of material parameters according to the boring log , seen by Figure 3.14, Figure 3.15, Figure 3.16, Figure 3.17, Figure 3.18, Figure 3.19, and Figure 3.20.

Table 3.3. Material Properties of the FEM Model.

Name	Depth	Type	Model	γ_{unsat}	γ_{sat}	ν	E_{ref}	c_{ref}
	m			kN/m ³	kN/m ³	-	kN/m ²	kN/m ²
Soil layer 1	0-8	Undrained	M-C	17	18	0.33	30,000	90
Soil layer 2	8-10	Undrained	M-C	17	18	0.33	26,000	190
Soil layer 3	10-56	Undrained	M-C	17	18	0.33	30,000	90
Concrete	n/a	Elastic	L-E	24	24	0.2	2.50E+07	-

The interface element is applied between piles and soil, the strength reduction factor R_{inter} is assumed to be 0.7. The definition of parameters can be referred to Section 2.3.2.

3.3.1.2. Results. In Figure 3.21 and Figure 3.22 total displacements of case 2 are given. As can be seen there, maximum deformation observed is 41.74 mm. The deformations observed indicates that the soil up to 8 m is effected by the movement. As can be seen in the following Figure 3.23, maximum vertical deformation observed is the same as total displacement value 41.74 mm which means there isn't any distinctive horizontal displacement.

In the following Figure 3.24 and Figure 3.25 displacement behaviors of numerically simulated and site measured case 2 under loading are given respectively. These two data are combined on the same template to show the similarities and differences between them which is given in Figure 3.26. The shape of the mobilized displacement curve appear in very good agreement with the measured one especially the small load stages. The subsequent loading stages especially before and after last unloading stage of the predicted displacement curve tends to be overestimated a small amount for measurement curve. In contrast, in the final loading stage towards to failure, measurement curve is higher than the predictions.

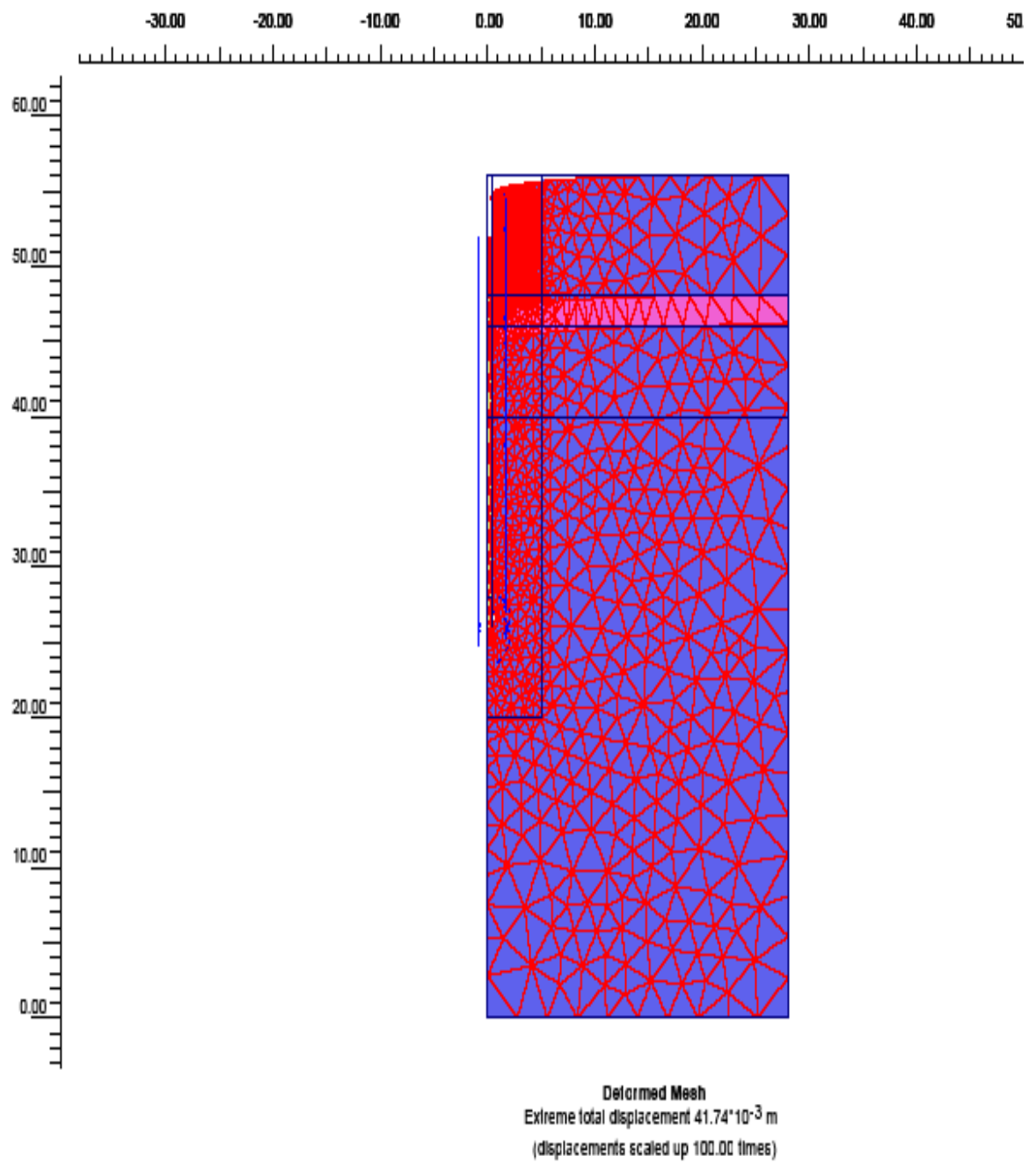


Figure 3.21. Deformed mesh of case2.

In Appendix B, additional numerical simulation results are given.

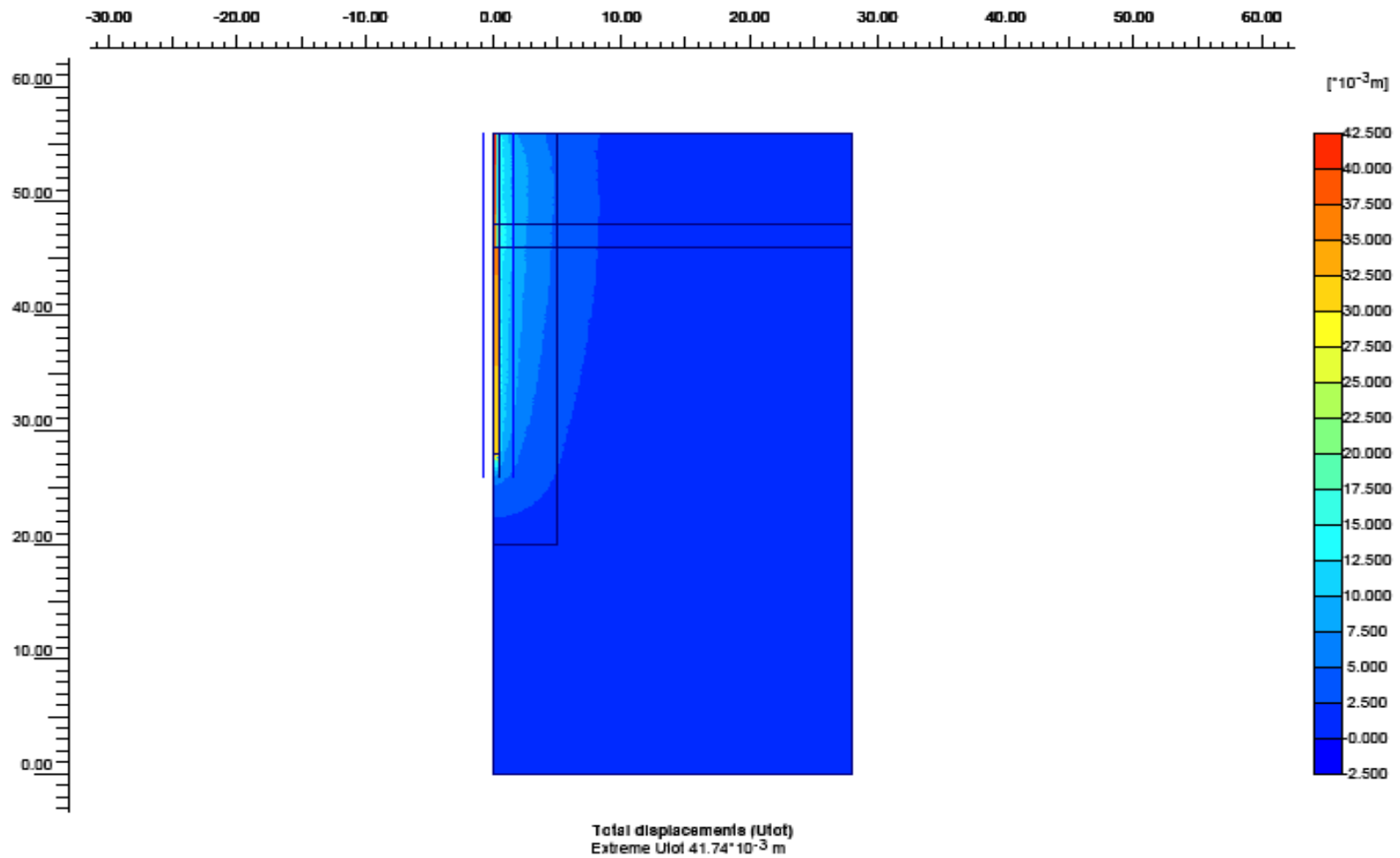


Figure 3.22. Total displacements of case2.

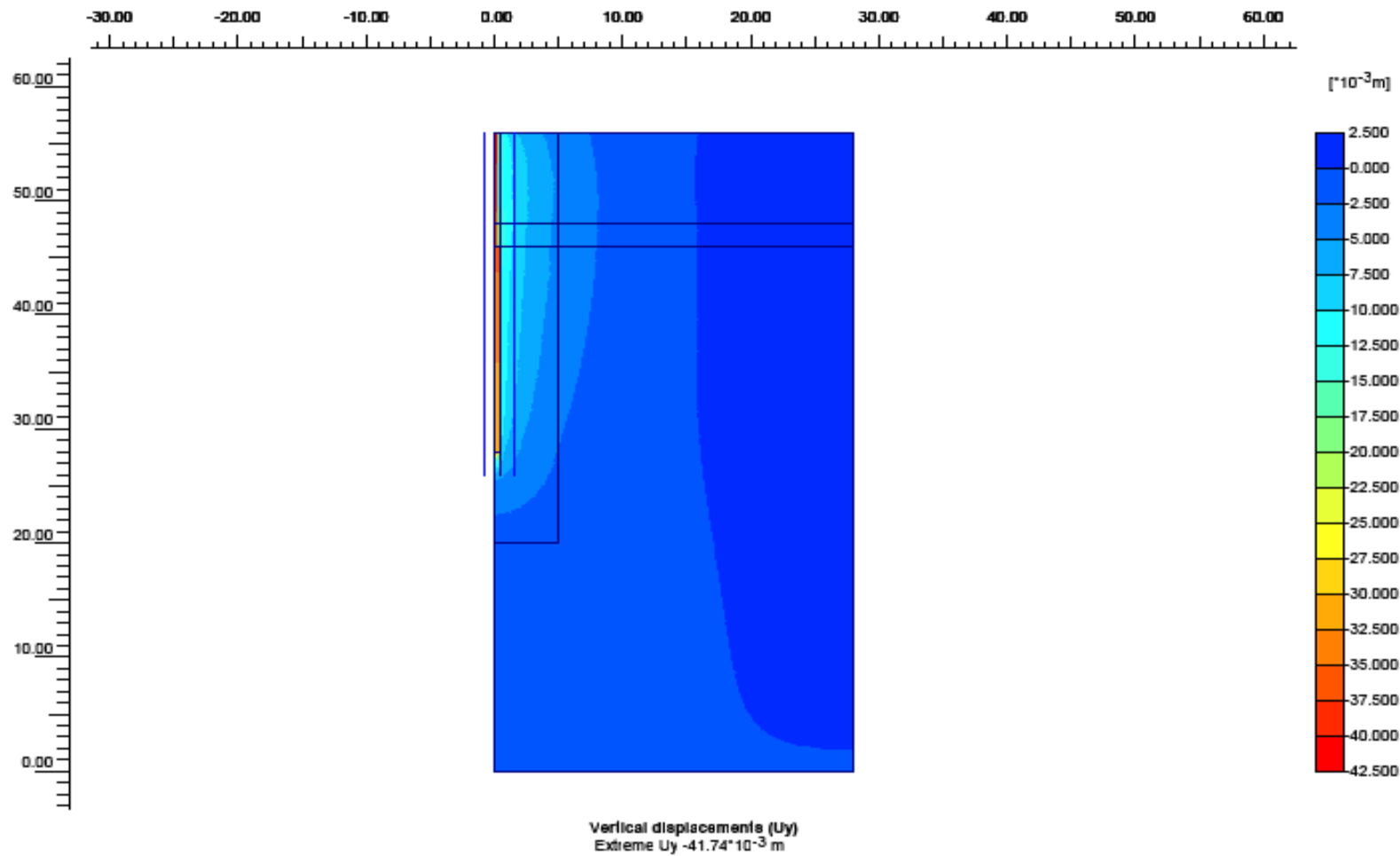


Figure 3.23. Vertical displacements of case2.

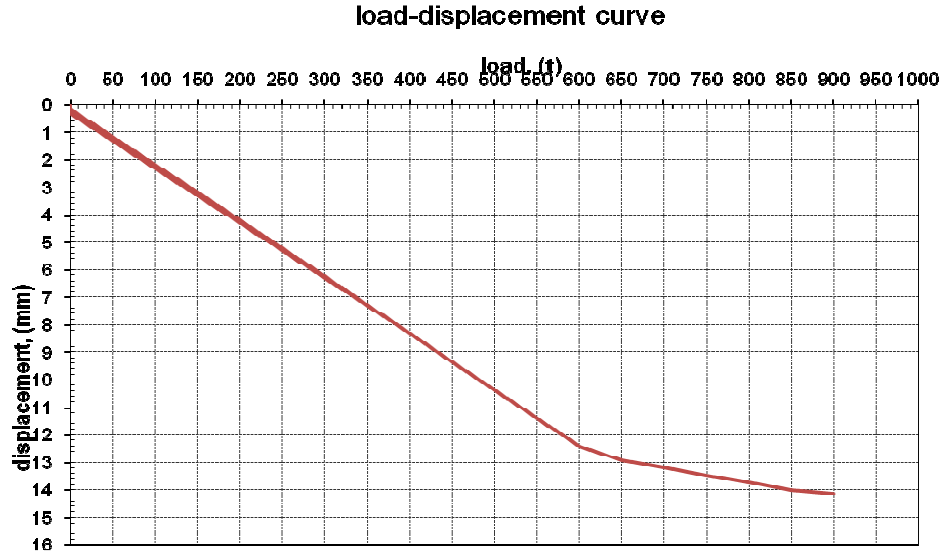


Figure 3.24. Load versus Displacement curve of case2.

3.3.1.3. Site Test Results. In Appendix B, additional site measured experimental data are given.

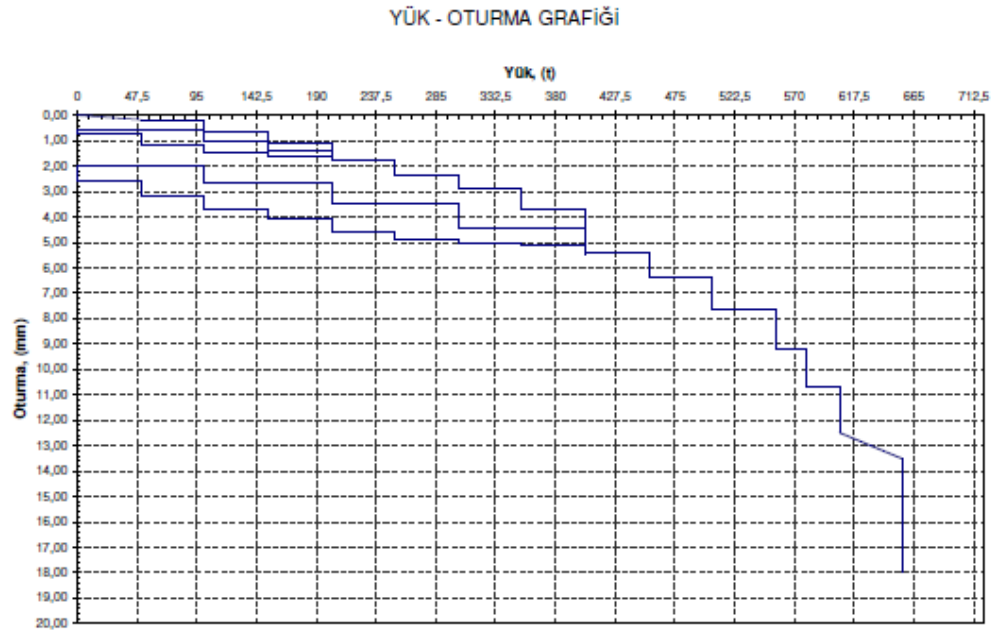


Figure 3.25. Load versus Displacement Curve of case2.

3.3.1.4. Site Measured and Numerically Simulated Load versus Displacement Curve of Case2. Both curves are given in the following figure.

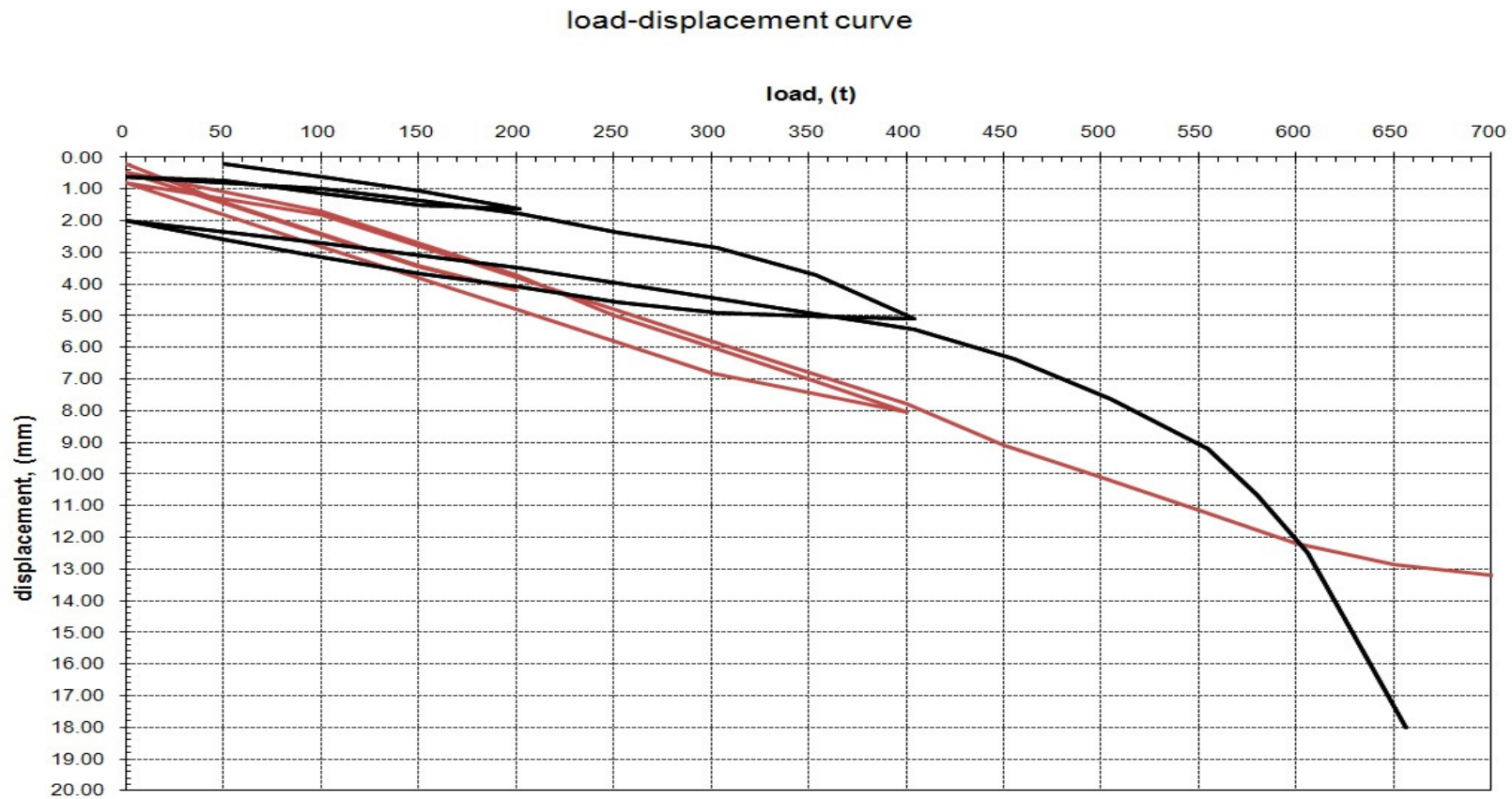


Figure 3.26. Site measured(black) and numerically simulated(red) load versus displacement curve of case2.

3.3.1.5. Stress Distribution Along the Pile Shaft. Stress distribution is given in the following figure.

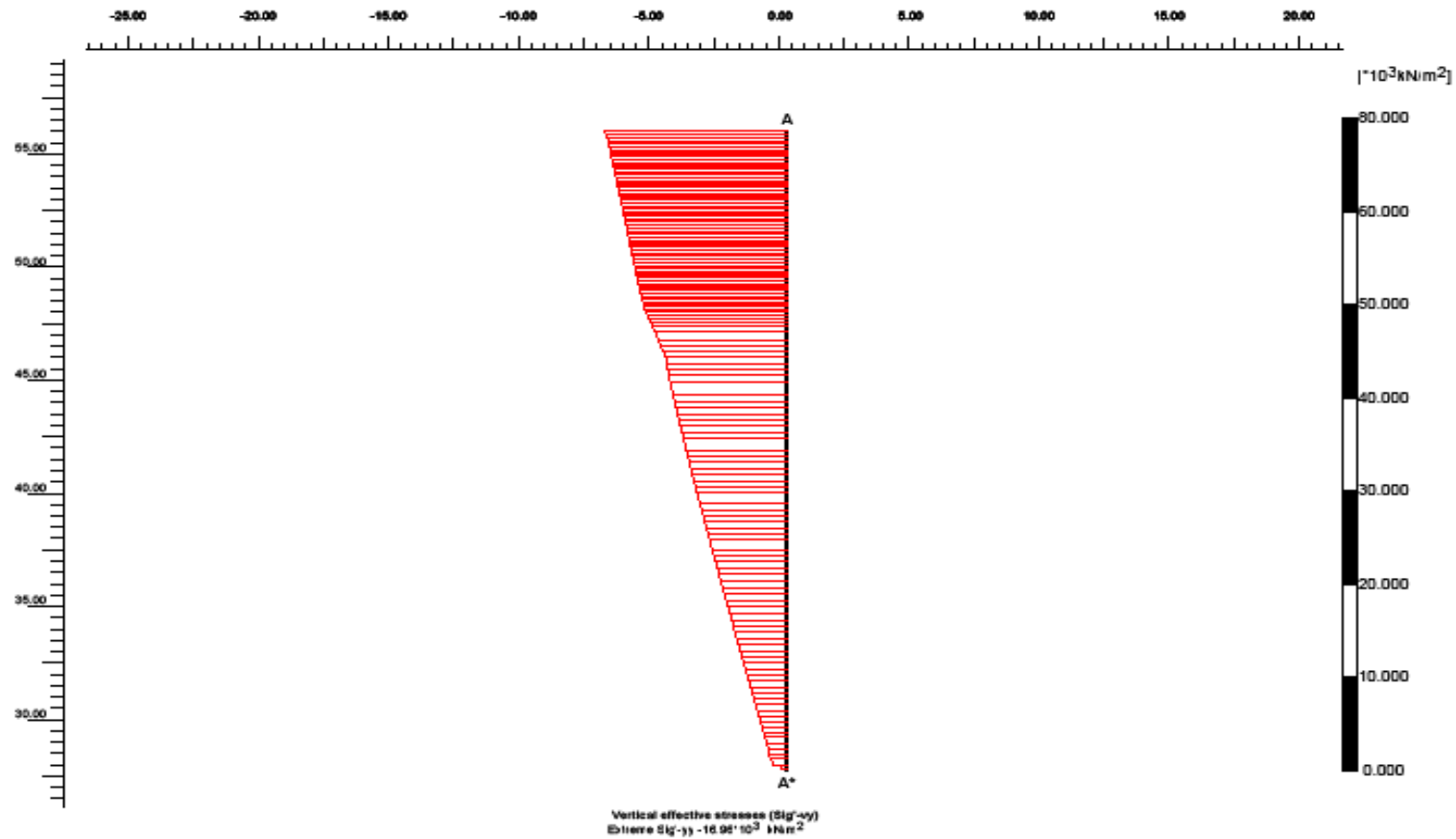


Figure 3.27. Vertical stress distribution along the pile shaft cross-section from the top to the bottom.

Obtained from Figure 3.27 in – 8 meters stress is $-9,420.76 \text{ kN/m}^2$ and in – 10 meters it is $-8,572.94 \text{ kN/m}^2$, in – 28 meters -407.085 and $-12,967.4 \text{ kN/m}^2$ at the top.

Combining the data from Figure 3.27 gives the curve given in Figure 3.28 in which decreasing stress distribution with depth can be seen.

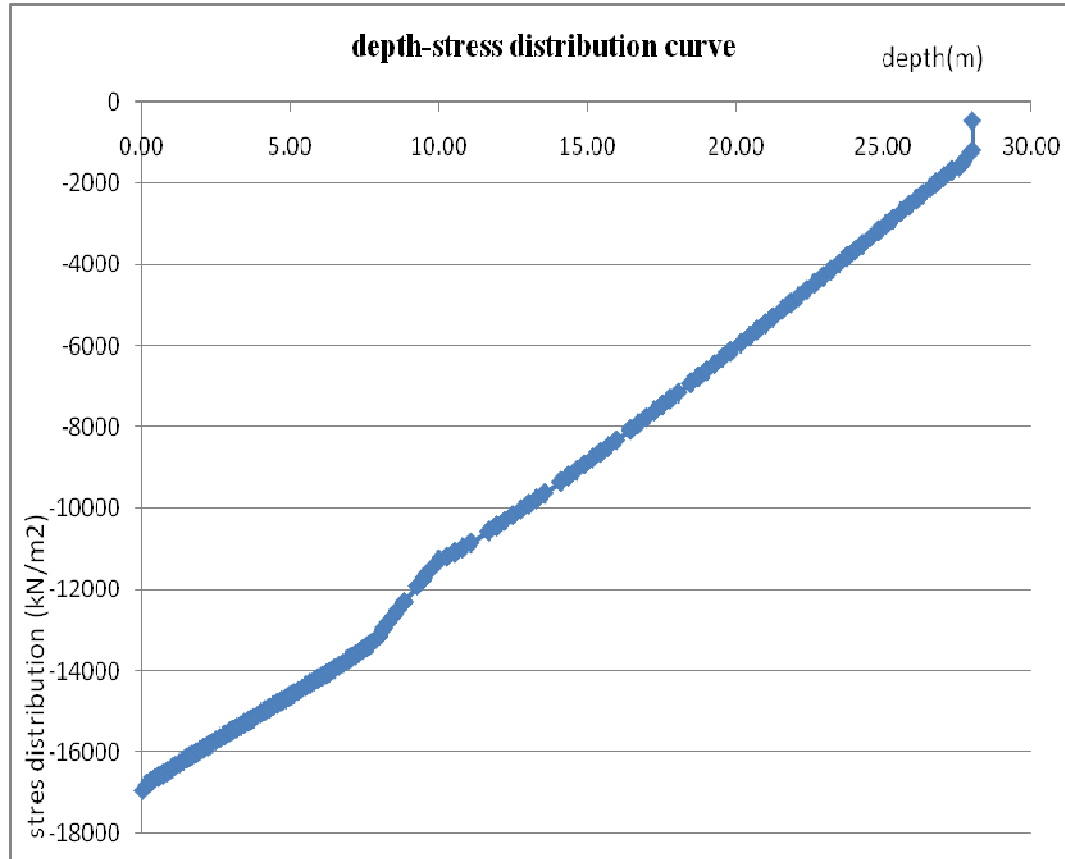


Figure 3.28. Depth(m) versus Stress(kN/m^2) distribution curve of pile shaft cross-section from the top to the bottom.

Stress change within a specific depth can be obtained from the Figure 3.28 and used to calculate cohesion within this specific depth by dividing the stress change to frictional resistance area. The result of the calculations is given in Table 3.4, in which the difference between calculated and assumed cohesion values can be seen. This difference stems from that I assumed an average cohesion value for a specific depth with the help of literature data while effective stress was changing within depth.

Table 3.4. Calculated and assumed cohesion values within depth.

depth	stress change	$c_{\text{calculated}}$	c_{assumed}
m	kN/m ²	kN/m ²	kN/m ²
0-8	12967.4-9420.76	88	90
8-10	9420.76-8572.94	84	190
10-28	8572.94-407.085	94	90

3.4. Case Study 3

3.4.1. General

Osterberg Cell (O-cell) testing has been carried out on one 1200 mm diameter bored pile constructed at the Sky Tower site in Istanbul, Turkey. The test has been performed as a single level bi-directional static load test. The objective of the test was to obtain and verify the capacity of the piles used in the geotechnical design and verify that the piles can meet the acceptance criteria as specified.

The pile information is summarized as follows:

Table 3.5. Pile Information.

Qty	Test Type	Pile Size	Aproximate Bore Length	Required Test Load
1	Compression O-Cell	φ 1200 mm	30.0 m	40.0 MN

3.4.1.1. Single Level O-Cell Test on Piles. One Osterberg Cell assembly (using two Ocells of 510 mm diameter) was installed in the test pile. The O-Cell assembly location in the test pile can be seen in Figure 3.29 as agreed with LOADTEST.

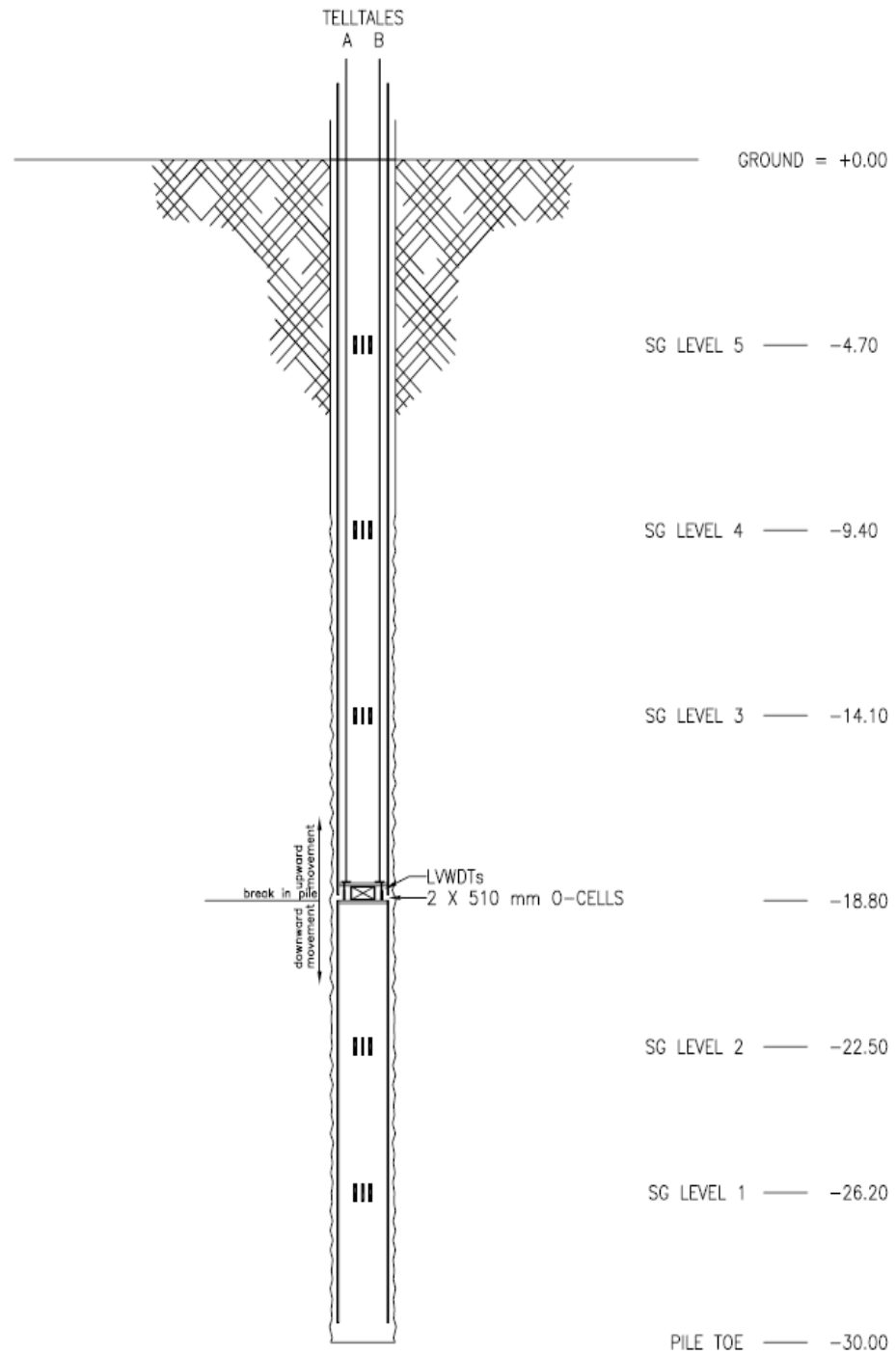


Figure 3.29. Schematic Section of Test Pile.

The O-cell assembly was fitted with top and bottom steel bearing plates (50 mm thick and of dimensions to correspond with the dimensions of the reinforcing cage). Details

can be seen in Figure 3.30, Figure 3.31 and Figure 3.32. The bearing plates were welded to the reinforcing steel cage as recommended by LOADTEST and approved by the Engineer. Any horizontal reinforcement was terminated immediately above and below the O-cell assembly. Vertical reinforcement terminated above and below the O-cell assembly and welded to the bearing plates.

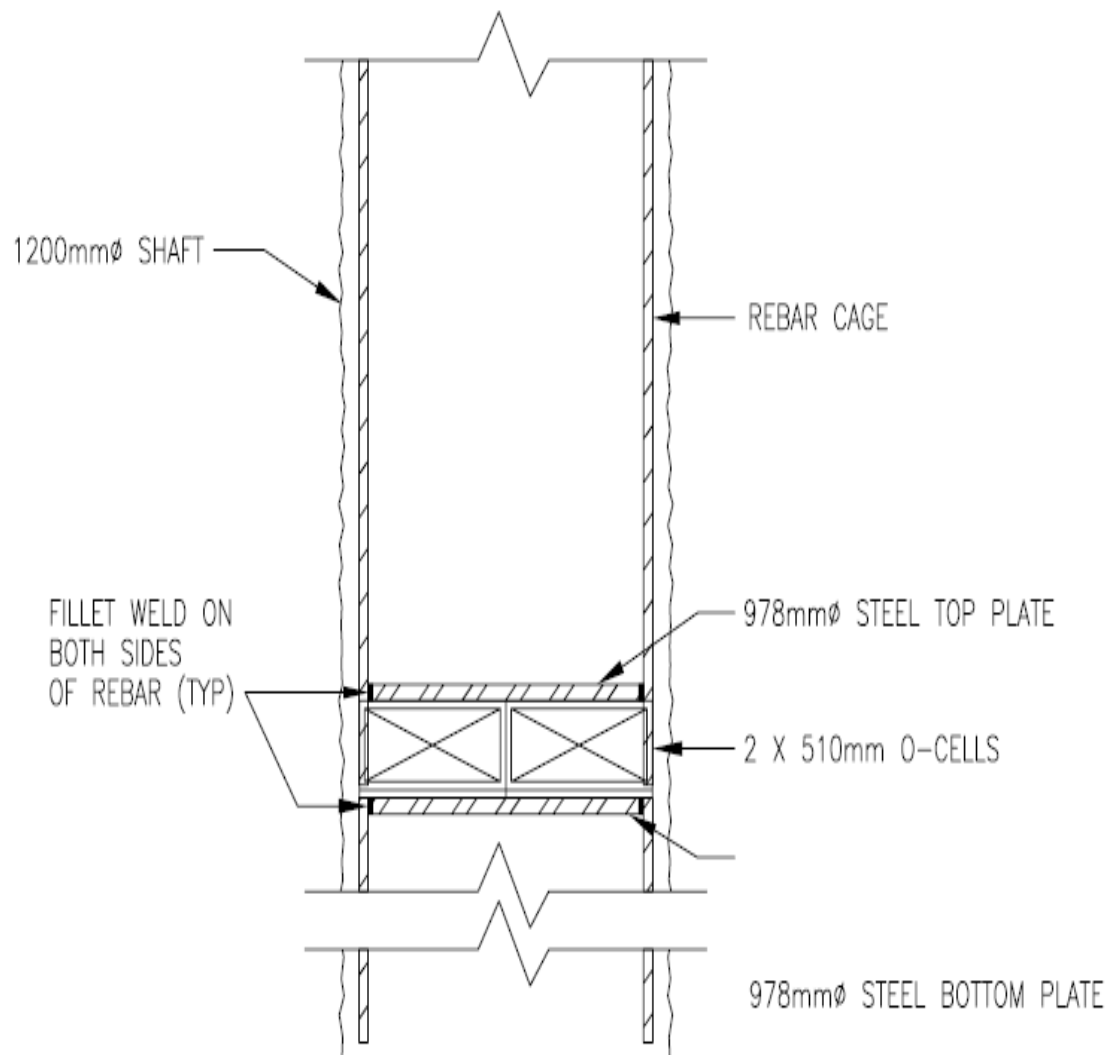
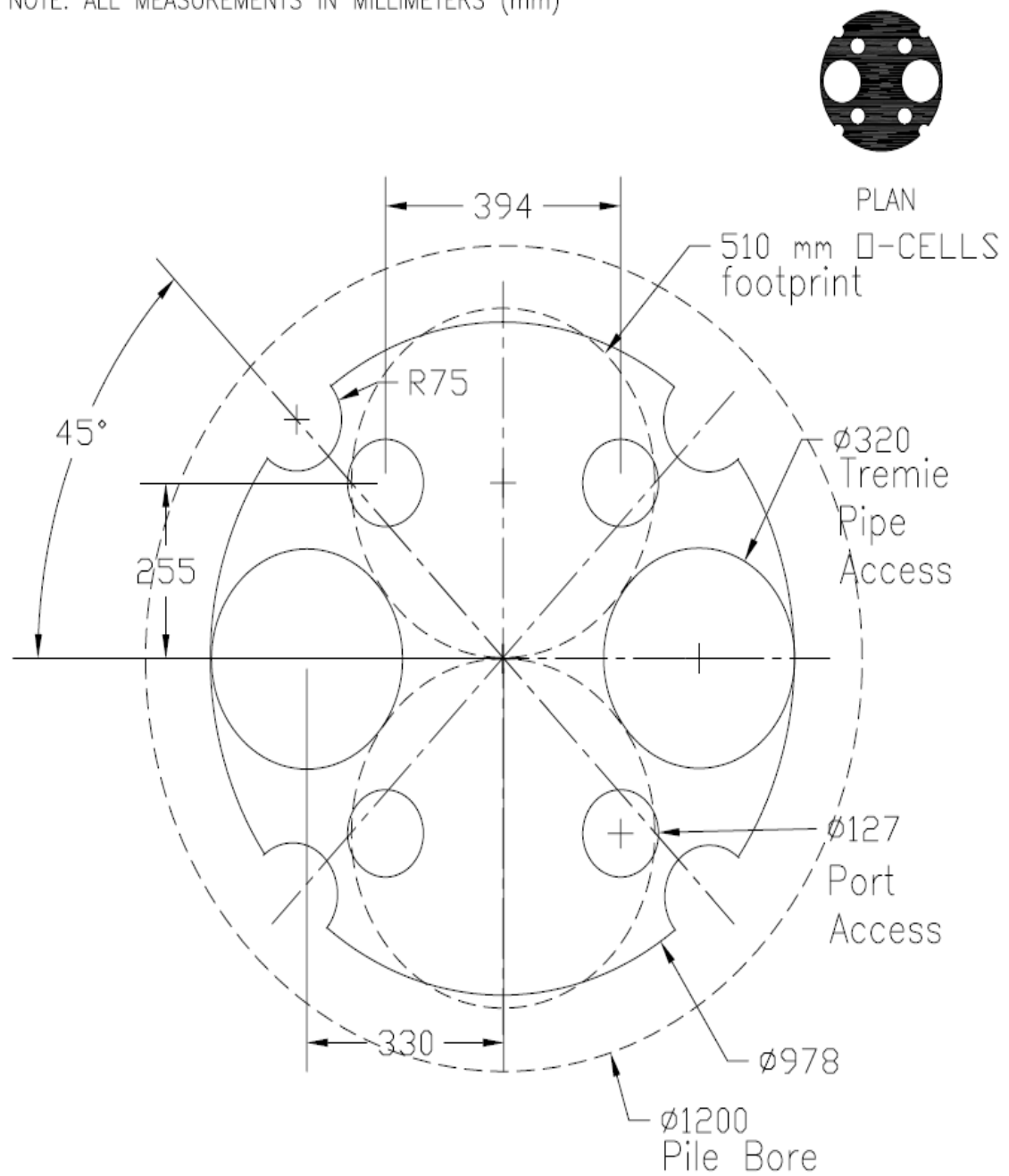


Figure 3.30. Conceptual Sketch of O-Cell Assembly.

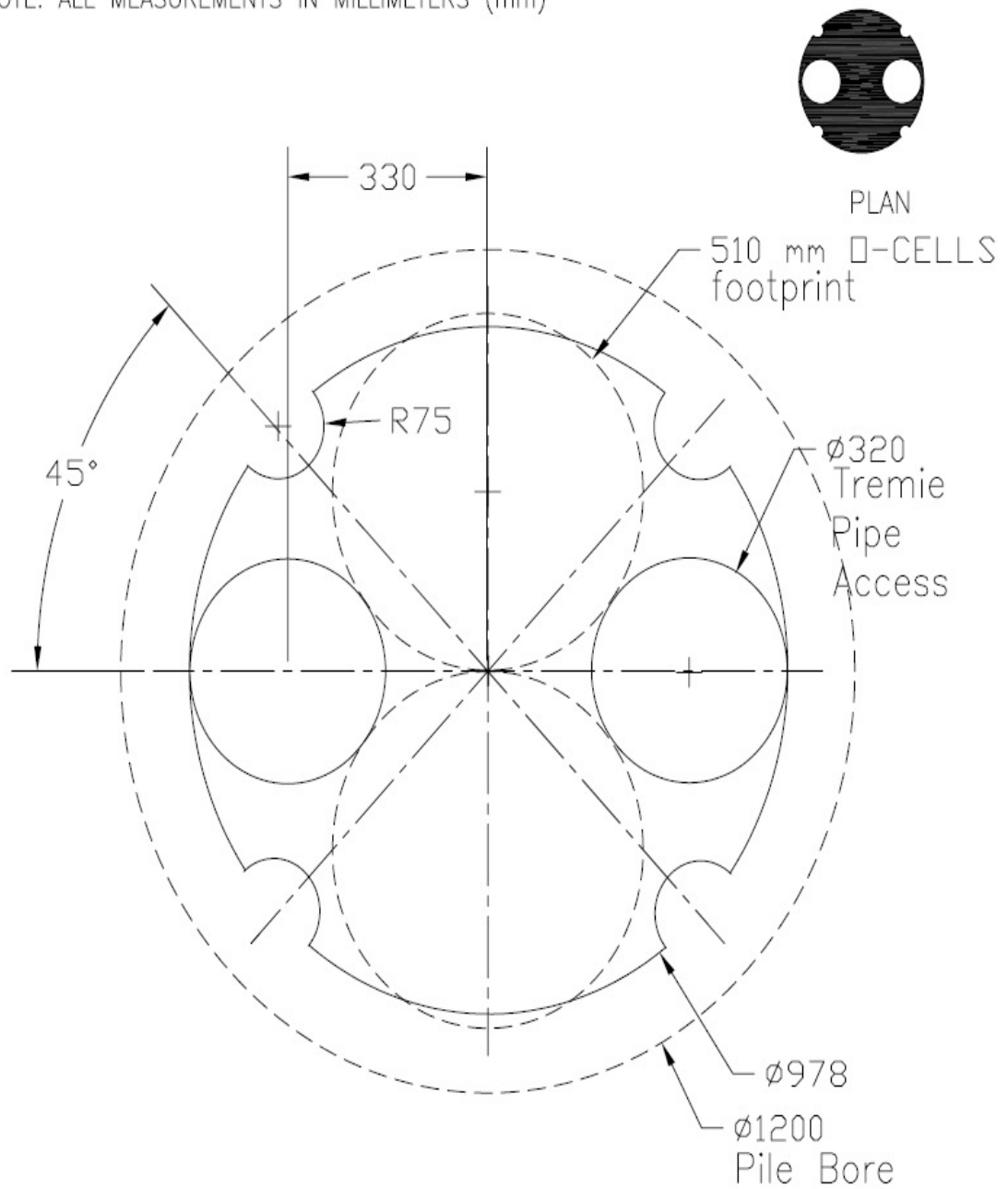
NOTE: ALL MEASUREMENTS IN MILLIMETERS (mm)



1 PER SHAFT WITH 2 X 510mm O-CELLS
978mmØ X 50mm THICK STEEL PLATE
(OR INSIDE DIA OF REBAR CAGE)

Figure 3.31. Top Plate Layout.

NOTE: ALL MEASUREMENTS IN MILLIMETERS (mm)



1 PER SHAFT WITH 2 X 510mm O-CELLS
 978mmø X 50mm THICK STEEL PLATE
 (OR INSIDE DIA OF REBAR CAGE)

Figure 3.32. Bottom Plate Layout.

3.4.1.2. O-Cell Positioning. The following table summarizes how the test pile was instrumented with O-cells and their positioning in the proposed test pile:

Table 3.6. Test Pile Instrumentation.

Pile Type	Bored
Number of Tests	1
Test type	Preliminary
Geometry(mm)	1200
Max Test load requirement(MN)	40.0 MN
Ground level elevation(m)	0.0
Concreted length(m)	30.0
O-Cells elevation(m)	-18.8
O-Cell quantities and diameters	2 x 510 mm
Concrete cover(mm)	75
Inside dimensions of rebar cage(mm)	978
Tremie pipe max OD(mm)	<300
Size and numbers of vertical rebar	22 T26
Size of link reinforcement	T10

A general schematic section of the test pile is included in Figure 3.29.

3.4.1.3. Instrumentation. Two traditional telltale casing and rod extensometers extending from the top of the O-cell assembly to top of pile were used to allow for measurement of the pile compression above the O-cell assembly.

Four 150 mm stroke Linear Vibrating Wire Displacement Transducers (LVWDTs Geokon Model 4450) fixed between the bottom and top steel bearing plates to directly measure O-cell expansion.

Two traditional telltale casing and rod extensometers or embedded compression telltales the O-cell and fixed at pile toe were used to allow for measurement of the pile compression below the O-cell assembly and actual pile toe displacement. The telltales were fitted with expansion sleeves to allow for expansion of the O-cell assembly where necessary.

Strain gauges were used to assess the force distribution within the pile shaft above and below the O-cell assembly. Suggested strain gauge elevations have been included in

Figure 3.29. Three strain gauges were fixed at 120° from each other at each level. The proposed elevations of the strain gauges are shown in the Schematic Section of Test Pile drawings. Geokon Model 4911-4 sister bar vibrating wire strain gauges were utilized.

3.4.2. Loading Schedule

The proposed loading schedule is included

Table 3.7. Proposed Loading Schedule.

Increment No.			Pressure		Gross O-Cell Load (MN)	Percent of available test load	Load Holding Criteria	
			(psi)	(Mpa)			Min Holding Time (min)	Rate of Creep to be obtained
1	L-	0	0	0	0	0.00%	10	n/a
1	L-	1	825	5.7	3.3	8.50%	30	* <0.1 mm/20-min and rate of creep slowing down or 1% of total displacement per hour or maximum 3 hour hold
1	L-	2	1650	11.4	6.7	17.10%	30	
1	L-	3	2475	17.1	10	25.60%	30	
1	L-	4	3300	22.8	13.3	34.20%	30	
1	L-	5	4125	28.4	16.7	42.70%	30	
1	L-	6	4950	34.1	20	51.30%	30	
1	L-	7	5775	39.8	23.3	59.80%	30	
1	L-	8	6600	45.5	26.7	68.40%	30	
1	L-	9	7425	51.2	30	76.90%	30	
1	L-	10	8250	56.9	33.4	85.50%	30	
1	L-	11	9075	62.6	36.7	94.00%	30	
1	L-	12	9900	68.3	40	102.60%	30	
**	1	L-	13	10725	73.9	43.4	111.10%	30
***	1	L-	14	11550	79.6	46.7	119.70%	30
***	1	L-	15	12375	85.3	50	128.20%	30
***	1	U-	1	8000	55.2	32.3	82.90%	10
	1	L-	2	6000	41.4	24.3	62.20%	10
	1	U-	3	4000	27.6	16.2	41.40%	10
	1	U-	4	2000	13.8	8.1	20.70%	10
	1	U-	5	0	0	0	0.00%	60

The loading schedule describes a single loading cycle up to test load or up to the maximum O-cell capacity in 12 steps. Each load was held for a minimum of 30 minutes or until the prescribed settlement rate is achieved, subject to a maximum duration of 3 hours. The required settlement rate to be met was 0.1mm per 20 minute interval.

Settlement rate was measured as the combined displacement of upward and downward movements measured directly by the expansion transducers monitoring the O-cell or by the difference between top and bottom plate measured using telltale rods. This is the most sensitive location for the measurements and the most immune to external influences.

The unloading was performed in 5 equal load steps from the maximum load applied and held for approximately 10 minutes. The recovery at zero load was monitored for a 1 hour period.

3.4.3. Field Measurements

The expansion of the O-cell assembly is directly measured by using Linear Vibrating Wire Displacement Transducers (LVWDTs Geokon Model 4450). The LVWDTs are fixed between the top and bottom bearing plates. Measurements were recorded to the nearest 0.01 mm division.

The upward vertical movement of the top of the pile was measured using two (2) LVDTs or LVWDTs attached to a reference beam set to measure traditional telltale rods set in the top of the pile. Details of the reference beam arrangement can be found in Figure 2.9. These LVDTs or LVWDTs are read to the nearest 0.01mm division. The stems of the indicators are plumbed to ensure accurate vertical measurement.

Pile compression under load was measured using a minimum of two (2) LVDTs or LVWDTs attached to the top of the pile and set over traditional telltale rods extending from top of O-cell to top of pile. These LVDTs or LVWDTs are read to the nearest 0.01mm division. The stems of the indicators are plumbed to ensure accurate vertical measurement.

Pile toe displacement under load was measured using LVDTs or LVWDTs attached at the top of the pile and set over the traditional telltale rod casing extending from pile toe to top of pile or directly by embedded compression telltales. Deducting measured compression from the downward O-cell movement provides for actual pile toe movement.

Unit shaft friction parameters were assessed along the pile shaft using Geokon Model 4911-4 strain gauges.

Where a conventional reference beam was used, with its supports located at over 3 pile diameters from the test pile, an automated digital survey level (Leica NA2002, NA3000 or NA3003) was used to monitor the reference beam for any movement during testing. Any top of pile movements with respect to the reference beam was assessed and corrected for variation in vertical position of the reference system.

All the LVDTs, LVWDTs and VWSGs were connected directly to a Data Electronics Geologger 515 or 615 data logger. The logger was, in turn, connected to a laptop computer. This arrangement allows LVDTs/LVWDTs vibrating wire readings to be recorded simultaneously and stored automatically at 30 second intervals during the test (depending on the number of instruments being read). The use of a data logger allowed for greater accuracy than manual readout box methods. All readings were rounded up or down to the nearest 0.01mm for reporting. In addition the loggers automatically recorded the temperature of the pile concrete and external ambient temperature during testing.

3.4.4. FEM Study on Case Study 3

The idealized finite element model of O-Cell pile load test and static load test are illustrated in Figure 3.33 and Figure 3.34. Figure 3.33 is the model of O-Cell test where the O-Cell is located 11 m above the pile toe; Figure 3.34 is the model of conventional top load test. Both of the tests are simulated in the same Mohr Coulomb soil model with 15 node elements; the meshes are generated axisymmetrically, with the boundary of 26 meters wide and 60 meters deep. The test piles are in the same size of diameter of 1,20 meters and length of 30 meters. To ensure the precision of the calculation, the meshes of an area of 5 meters wide, 40 meters deep around the test pile are refined.

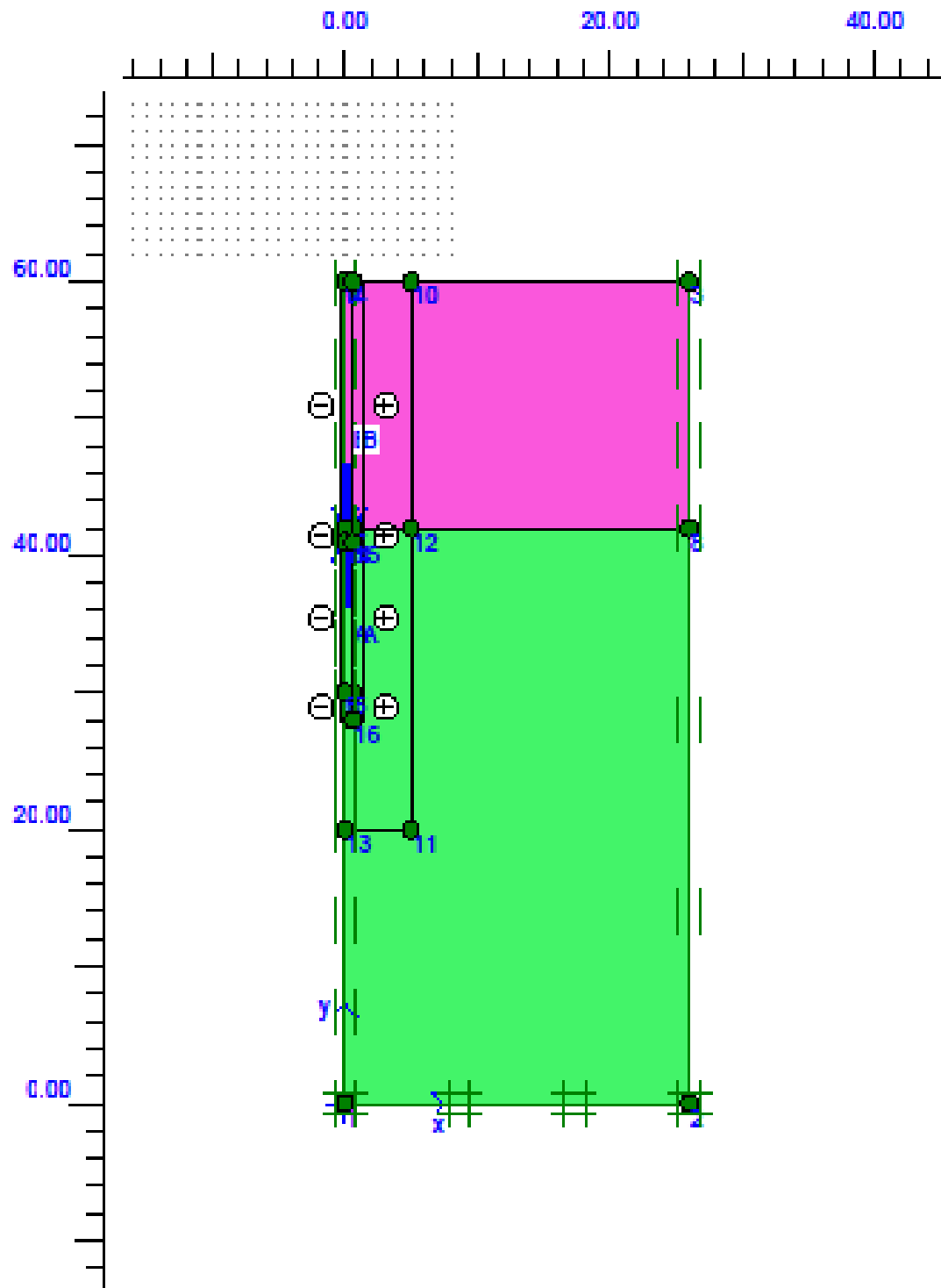


Figure 3.33. Finite element model of case3(O-Cell).

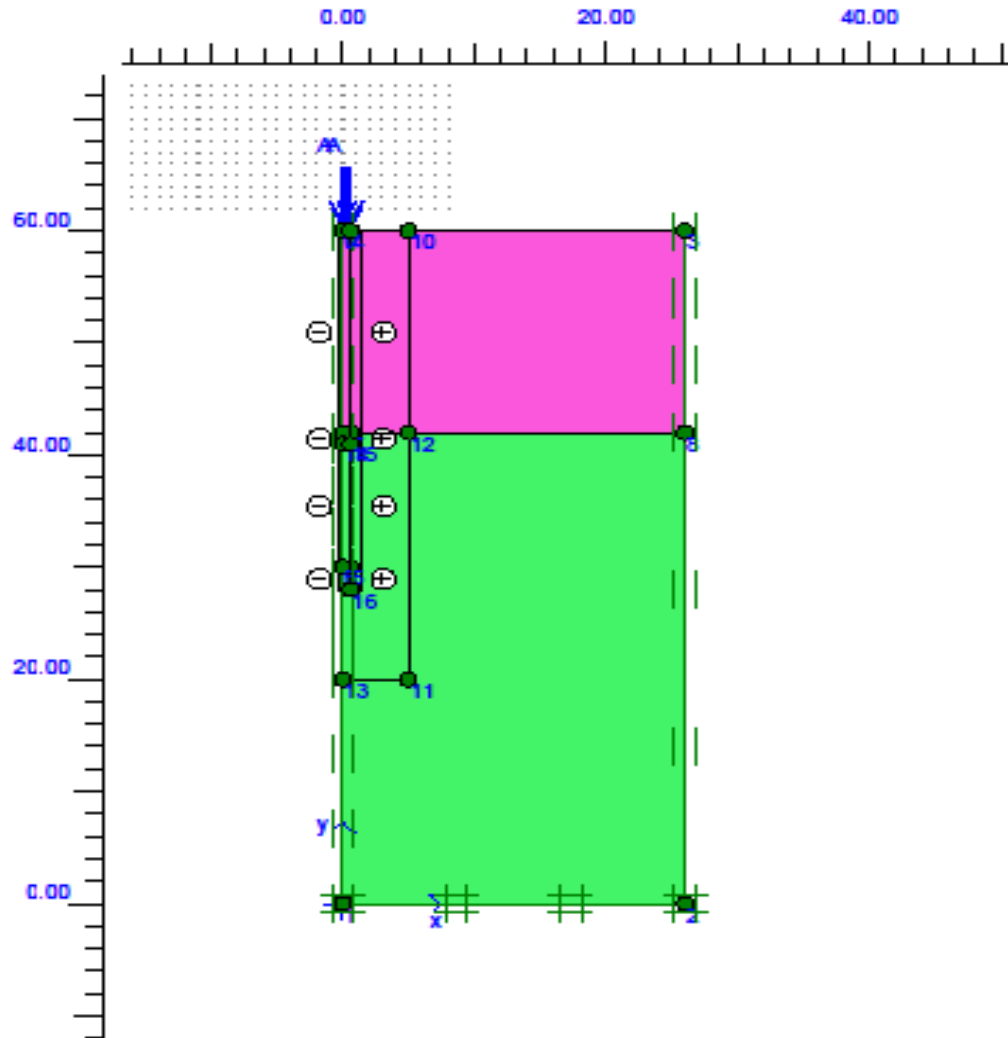


Figure 3.34. Finite element model of case3(static).

3.4.4.1. Soil Conditions and Design Parameters. Design parameters are obtained by incorporation of soil boring log which is seen in Figure 3.35 and Figure 3.36.

According to the boring log ,the Layer 1 is taken from the ground level +0,00 to - 18,00 m and the Layer 2 is taken from – 18,00 m to – 60,00 m. Ground water level starts from - 40 meters apart from mesh area.

For soil layer1; undrained cohesion was assumed as 350 kPa. Young` s Modulus was assumed as 150,000 kN/m².

For soil layer2; undrained cohesion was assumed as 600 kPa. Young` s Modulus was assumed as 4,000,000 kN/m²

These assumptions were made because soil profile where the pile was built, was a rock strata. Therefore, parameters was used as high as possible.

Poisson ratio is generally taken between 0.3 and 0.35 for soils as drained parameters in undrained analysis and 0.2 for concrete in Plaxis Teaching Manuals. Because ground water level is apart from mesh area for both layers it was assumed 0.3. In addition, Young` s Modulus of concrete was assumed as 34,000,000 kN/m² according to the formula (3.3) , because C35 type of concrete was used in the pile.

Table 3.8 lists the parameters of material parameters according to the boring log, seen by Figure 3.35 and Figure 3.36.

Table 3.8. Material Properties of the FEM Model.

Name	Depth	Type	Model	γ_{unsat}	γ_{sat}	ν	E_{ref}	c_{ref}
	m			kN/m ³	kN/m ³	-	kN/m ²	kN/m ²
Soil layer 1	0-18	Undrained	M-C	21	23	0.3	150000	350
Soil layer 2	18-60	Undrained	M-C	21	23	0.3	400000	600
Concrete	n/a	Elastic	L-E	24	24	0.2	3.40E+07	-

The interface element is applied between piles and soil, the strength reduction factor R_{inter} is assumed to be 0,7. The definition of parameters can be referred to Section 2.3.2.

The test procedures of these methods are simulated by FEM model. The respective results of the tests are compared in the following sections.

İŞVEREN Carried Out For : Akdeniz İnşaat		SONDAJ YERİ/ Boring Location :		DERİNLİK Depth (m)	SONDAJ No Borehole No	SAYFA Page
İDARE Directorate :		KOORDİNAT-Y : 964.382 m		30	A BLOK 1	1 / 2
		KOORDİNAT-X : 4813.067 m		YER ALTI SUYU / Ground Water		
PROJE : Atışehir Projesi Geoteknik Project : Danışmanlık		ZEMİN KOTU / G. Elev. : 101.265 m		DERİNLİK Depth (m)	TARİH Date	AÇIKLAMA Explanation
		BAŞ. TARİHİ / Start Date : 28.06.2009				
MAKİNE TİPİ / Machine Type : Dişmec		BİT. TARİHİ / Finish Date : 30.06.2009				
SONDAJ TİPİ / Boring Type : Rotary		DEL. ÇAPI / Hole Diam. : 76 mm				

Sondaj Derinliği Boring Depth (m)	Numune Tipi / Sample Type	MANİPÜLASYON BOYU (Run)	Müh. Boru / Casing	STANDARD PENETRASYON DENEYİ (Standard Penetration Test)				GRAFIK (Graph)	KAYA ÖZELLİKLERİ (Rock Characteristic)					LİTOLOJİ (LITHOLOGY)	AÇIKLAMALAR (EXPLANATION)	
				DARBE SAYISI (Number of Blows)					TCR %	SCR %	RCR %	DAYANIMLILIK Strength	AYRISMA / Weathering			KIRILMA / Fracture (30cm)
				0-15 cm	15-30 cm	30-45 cm	N									
0.00																
1.50	SPT			50/8				R	80	0	0					
1.98									40	0	0					
3.00	SPT			50/7				R	67	-	-					
3.07																
4.50	SPT			50/8				R	80	0	0					
4.98									67	0	0					
6.00									100	0	0					
7.50									67	0	0					
9.00									80	0	0					
10.50									73	0	0					
12.00																
13.00																

DAYANIMLILIK / Strength			AYRISMA / Weathering			İNCE DANELİ / Fine Grained			KIRILMA / Fracture			ORANLAR / Proportions		
I	DAYANIMLI	Strong	I	TAZİ	Fresh	N : 0-2	ÇOK YUMUŞAK	V. Soft	N : 0-4	ÇOK GÜÇLÜ	V. Loose	N : 0-4	ÇOK GÜÇLÜ	V. Loose
II	ORTA DAYANIMLI	M. Strong	II	AZ AYRISMA	Slightly W.	N : 3-4	YUMUŞAK	Soft	N : 5-10	GÜÇLÜ	Loose	N : 5-10	GÜÇLÜ	Loose
III	ORTA ZAYIF	M. Weak	III	ORTA AYRISMA	Md. W.	N : 5-8	ORTA KATI	M. SET	N : 11-20	ORTA GÜÇLÜ	M. Dense	N : 11-20	ORTA GÜÇLÜ	M. Dense
IV	ZAYIF	Weak	IV	ÇOK AYRISMA	Highly W.	N : 9-15	KATI	SEF	N : 21-40	SİĞİRCİ	Dense	N : 21-40	SİĞİRCİ	Dense
V	ÇOK ZAYIF	V. Weak	V	TÜMEYİCİ AYRISMA	Comp. W.	N : 16-30	ÇOK KATI	V. SET	N : 41-60	ÇOK GÜÇLÜ	V. Dense	N : 41-60	ÇOK GÜÇLÜ	V. Dense

KAYA KALİTESİ TANIMI - RQD			KIRILMA - 30 cm / Fractures			ORANLAR / Proportions		
% 0-25	ÇOK ZAYIF	V. Poor	1	SEYİRCİ	Wd. (W)	% 5	PK AZ	Slightly
% 26-50	ZAYIF	Poor	1-2	ORTA AYRISMA	Medium (M)	% 5-10	AZ	Loose
% 51-75	ORTA	Fair	3-10	SEYİRCİ	Wd. (W)	% 11-20	ÇOK	Very
% 76-90	İYİ	Good	10-20	ÇOK SEYİRCİ	Wd. (W)	% 21-30	VE	And
% 91-100	ÇOK İYİ	Excellent	30	PARÇALI	Crushed (C)			

SPT	Standard Penetration Test	K	Next Number	SAHA MÜHENDİSİ (Site Engineer)	KONTROL MÜHENDİSİ (Checked By)
D	Disturbed Sample <td>P</td> <td>Pressuremeter Test <td>İSİM (Name) : Ramazan Meluğh</td> <td>Gökmen S. BAKLAR</td> </td>	P	Pressuremeter Test <td>İSİM (Name) : Ramazan Meluğh</td> <td>Gökmen S. BAKLAR</td>	İSİM (Name) : Ramazan Meluğh	Gökmen S. BAKLAR
UD	Undisturbed Sample <td>VS</td> <td>Van Shear Test <td>İMZA (Signature) :</td> <td></td> </td>	VS	Van Shear Test <td>İMZA (Signature) :</td> <td></td>	İMZA (Signature) :	

Figure 3.35. Boring Log Page 1.

3.4.4.2. Results.

i. O-Cell Test:

In Figure 3.37 and Figure 3.38 total displacements of case 3 are given. As can be seen there, maximum deformation observed is 16.27 mm. The deformations observed indicates that the soil up to 7 m is effected by the movement. As can be seen in the following Figure 3.39, maximum vertical deformation observed is the same as total displacement value 16.27 mm which means there isn't any distinctive horizontal displacement.

In the following Figure 3.40 and Figure 3.41 displacement behaviors of numerically simulated and site measured top and bottom of O-Cell displacement behavior under loading are given respectively. These two data are combined on the same template to show the similarities and differences between them which is given in Figure 3.42. The shape of the mobilized displacement curve appear in very good agreement with the measured one. For bottom of O-Cell movement; in low load stages the predicted displacement curve tends to be overestimated a small amount for the measurement curve ; however, after 20 MN loading measured displacement gets higher than the predicted one. In addition, for top of O-Cell movement; in all load stages the predicted displacement curve tends to be overestimated for measurement curve; however this small displacement difference starts to increase after 20 MN.

In Appendix C, additional numerical simulation results are given.

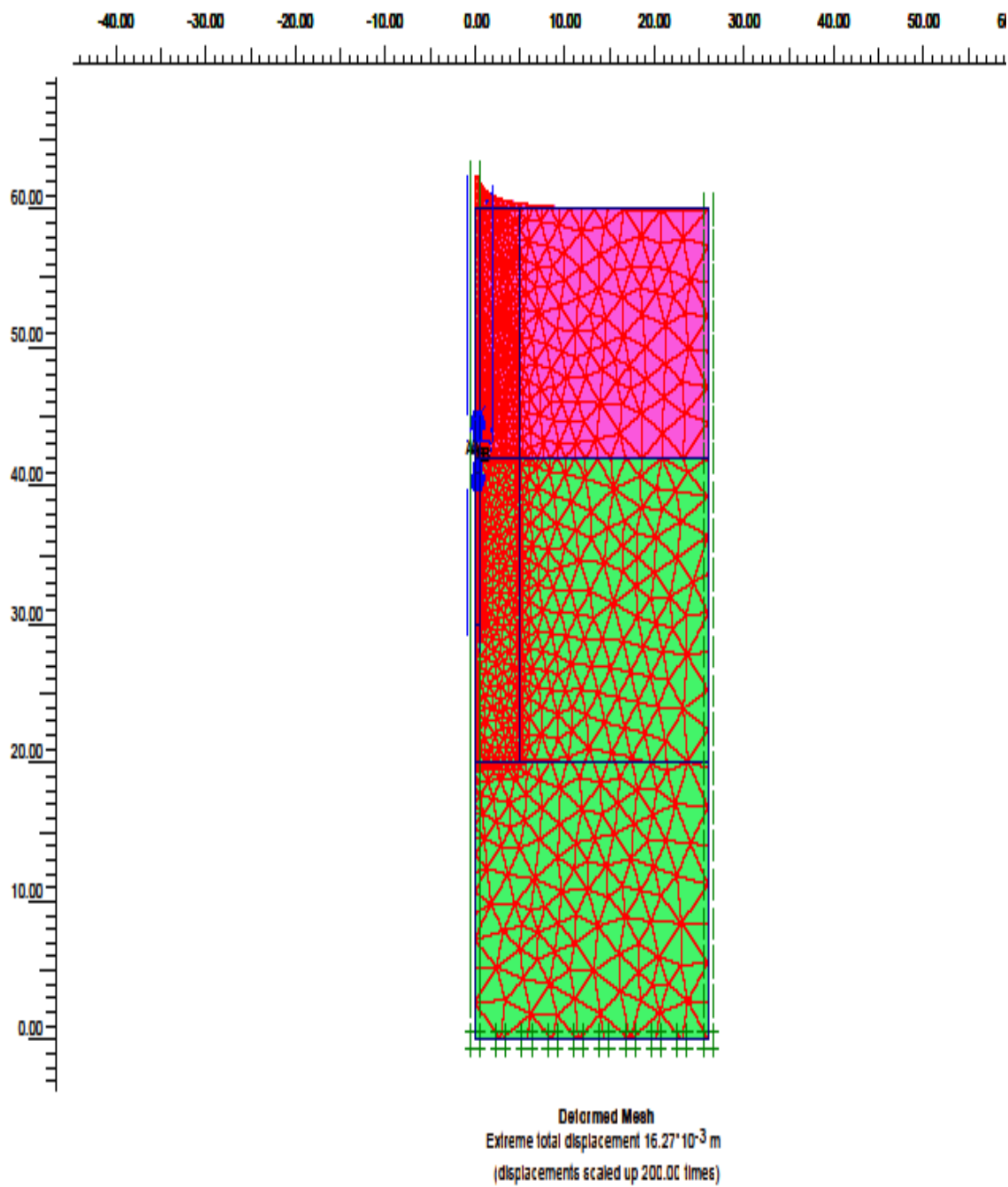


Figure 3.37. Deformed mesh of case3.

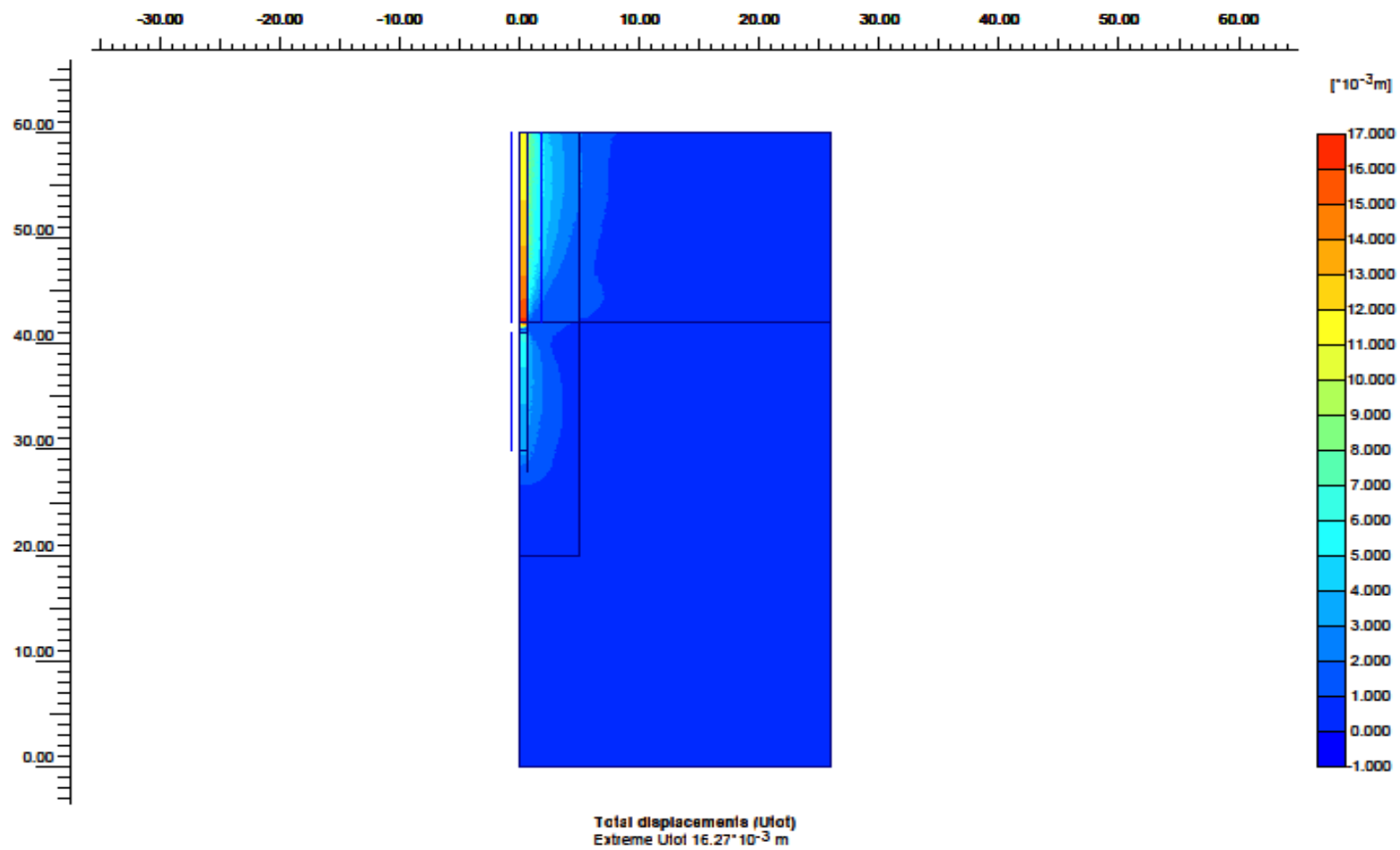


Figure 3.38. Total displacements of case3.

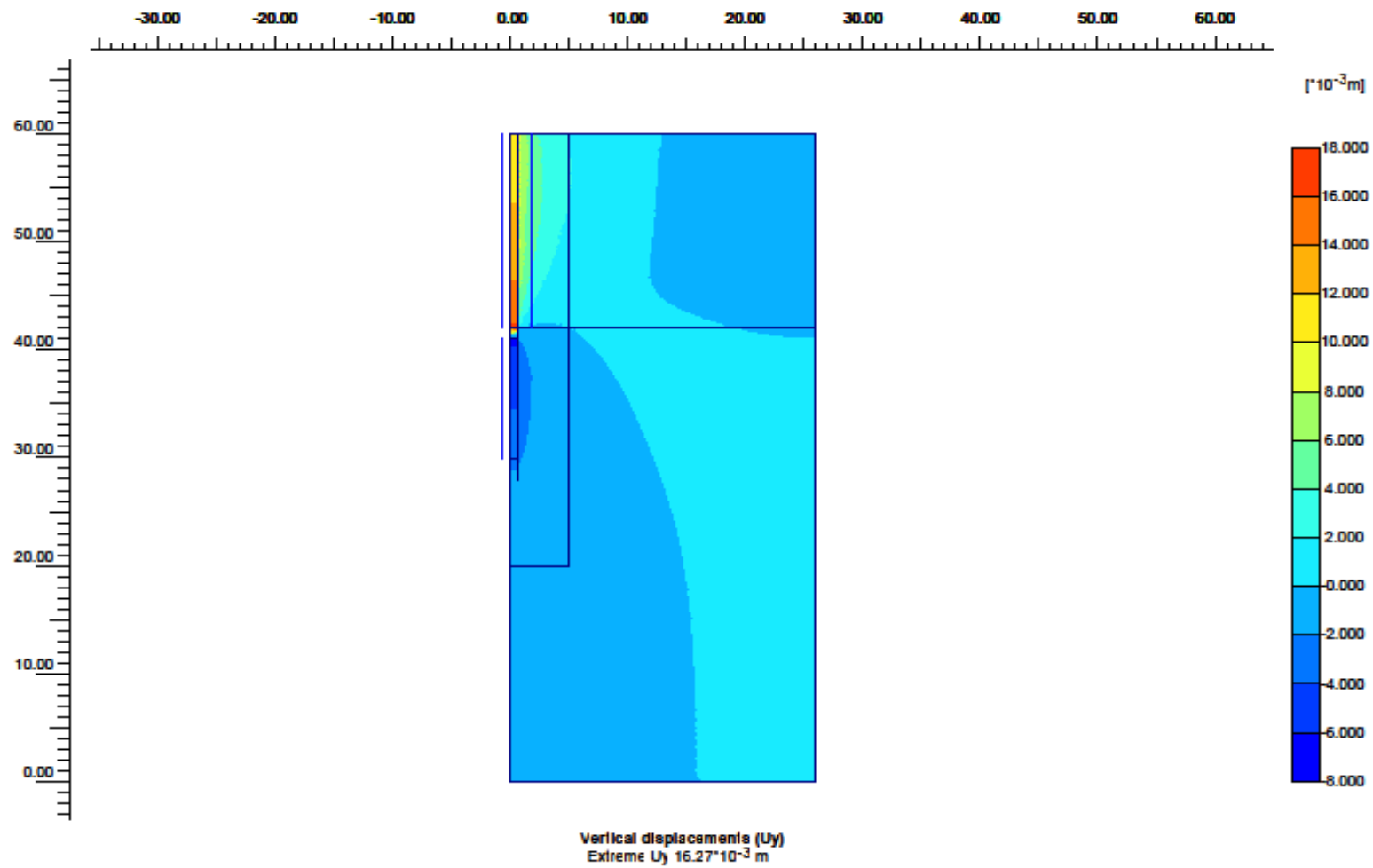


Figure 3.39. Vertical displacements of case3.

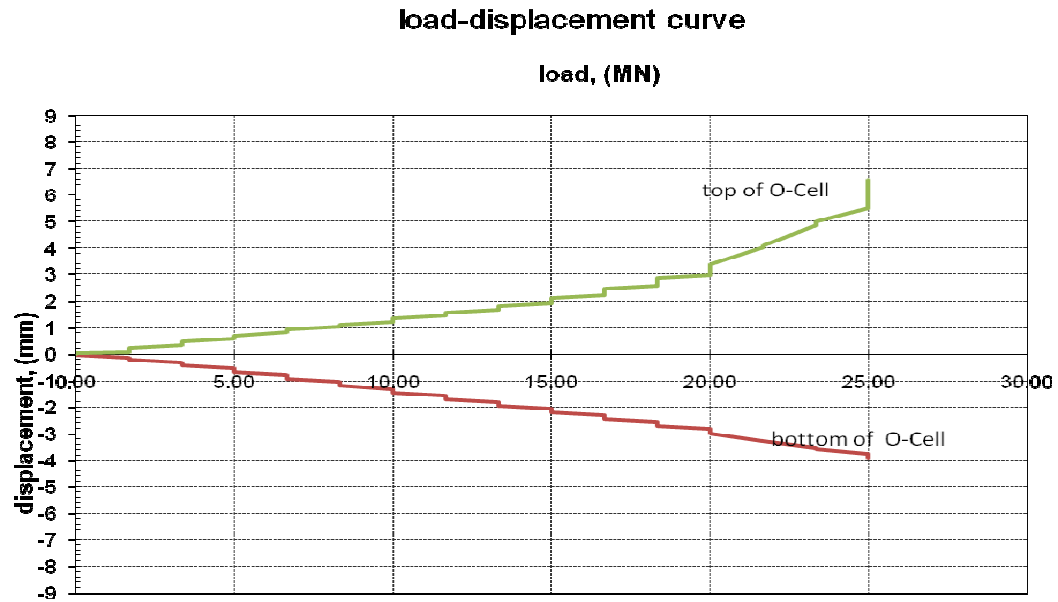


Figure 3.40. Load versus Displacement curve of case3.

3.4.4.3. Site Test Results. In Appendix C , additional site measured experimental data are given.

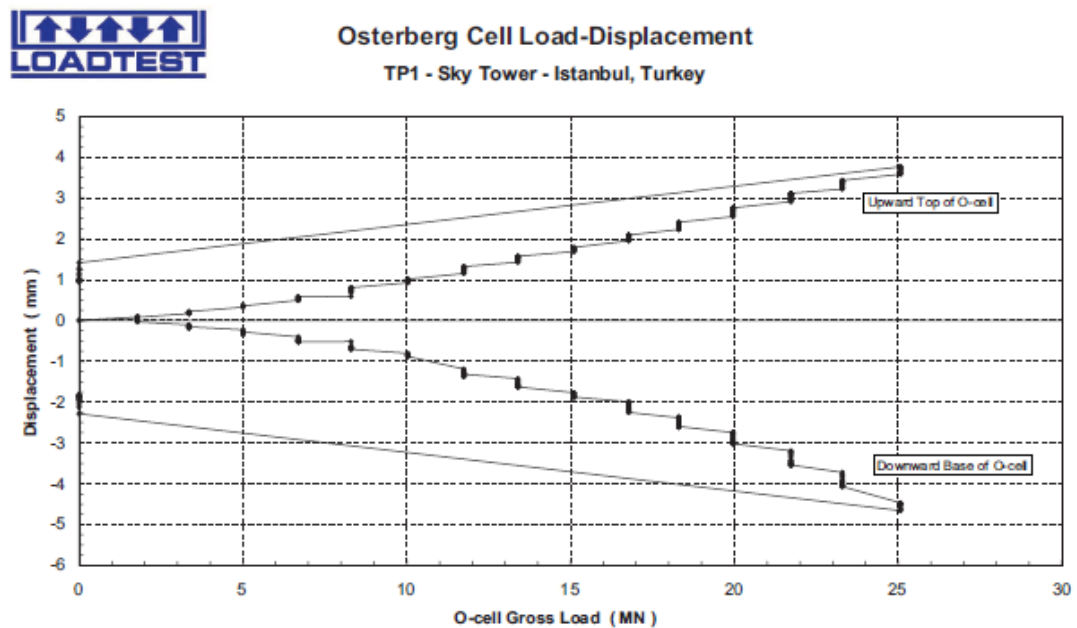


Figure 3.41. Load versus Displacement curve of case3.

3.4.4.4. Site Measured and Numerically Simulated Load versus Displacement Curve of Case3. Both curves are given in the following figure

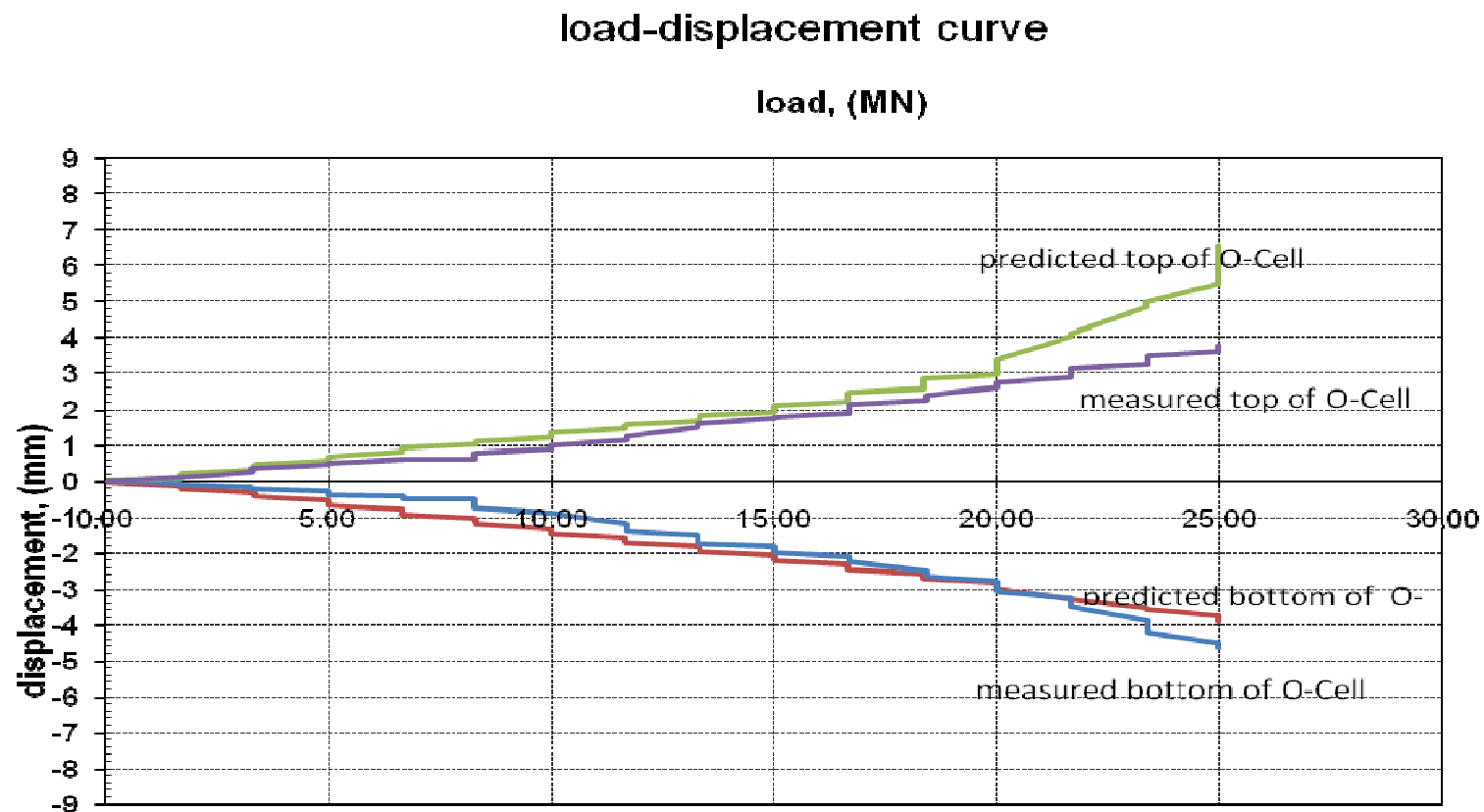


Figure 3.42. Site measured(blue and purple) and numerically simulated(red and green) load versus displacement curve of case3.

3.4.4.5. Stress Distribution Along the Pile Shaft. Stress distribution is given in the following figure.

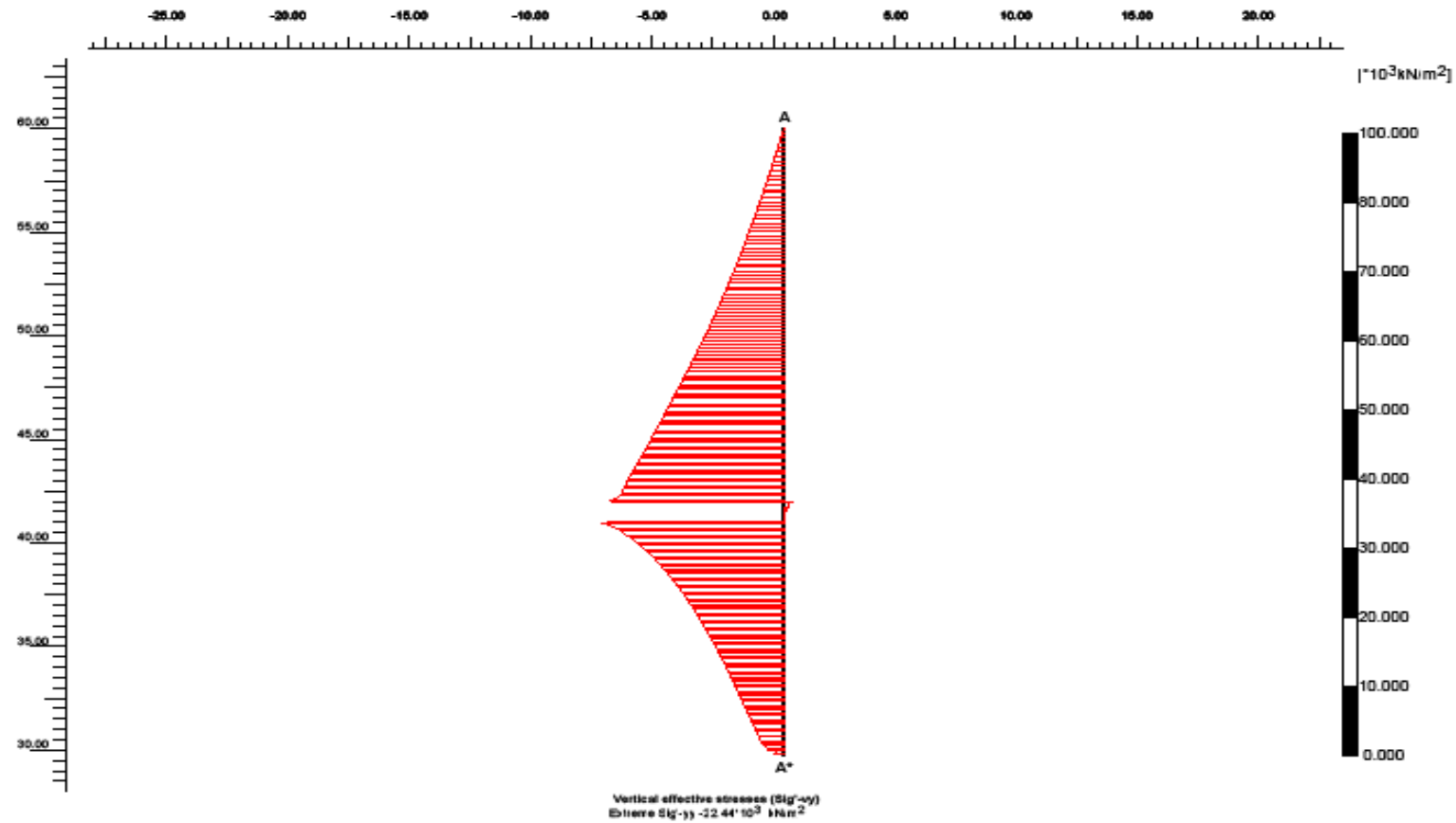


Figure 3.43 Vertical stress distribution along the pile shaft cross-section from the top to the bottom.

Obtained from Figure 3.43 in – 18 meters stress is $-21,026 \text{ kN/m}^2$ and in – 19 meters it is $-21,508 \text{ kN/m}^2$, in – 30 meters $-1,103.09$ and -209.44 kN/m^2 at the top.

Combining the data from Figure 3.43 gives the curve given in Figure 3.44 in which decreasing stress distribution with depth at the bottom of O-Cell and increasing stress distribution with depth at the top of O-Cell can be seen.

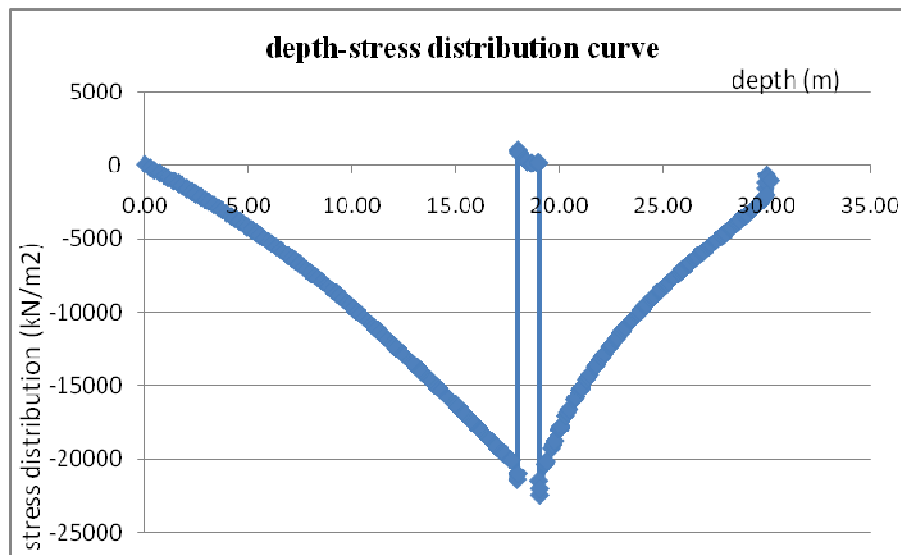


Figure 3.44. Depth(m) versus Stress(kN/m^2) distribution curve of pile shaft cross-section from the top to the bottom.

Stress change within a specific depth can be obtained from the Figure 3.44 and used to calculate cohesion within this specific depth by dividing the stress change to frictional resistance area. The result of the calculations is given in Table 3.9, in which the difference between calculated and assumed cohesion values can be seen. This difference stems from that I assumed an average cohesion value for a specific depth with the help of literature data while effective stress was changing within depth.

Table 3.9. Calculated and assumed cohesion values within depth.

depth m	stress change kN/m^2	$C_{\text{calculated}}$ kN/m^2	C_{assumed} kN/m^2
0-18	209.44-21,026	347	350
19-30	21,508.7-1,103.09	557	600

ii. Static Test

In Figure 3.45 and Figure 3.46 total displacements of case 3 are given. As can be seen there, maximum deformation observed is 27.93 mm. The deformations observed indicates that the soil up to 7 m is effected by the movement. As can be seen in the following Figure 3.47 maximum vertical deformation observed is the same as total displacement value 27.93 mm which means there isn't any distinctive horizontal displacement.

In the following Figure 3.48 displacement behavior of numerically simulated displacement behavior of static case3 under loading is given which is combined with the pile top displacement behavior of O-Cell case under loading in Figure 3.51. Investigation of graph is given in 3.4.4.7.

In Appendix C, additional numerical simulation results are given.

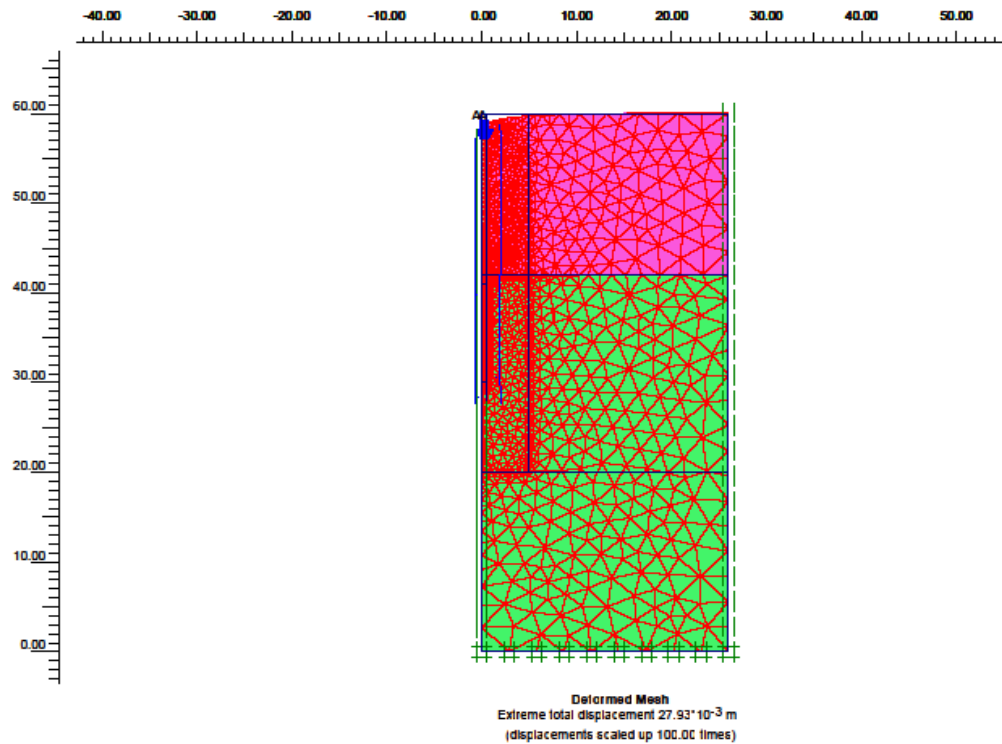


Figure 3.45. Deformed mesh of case3(static).

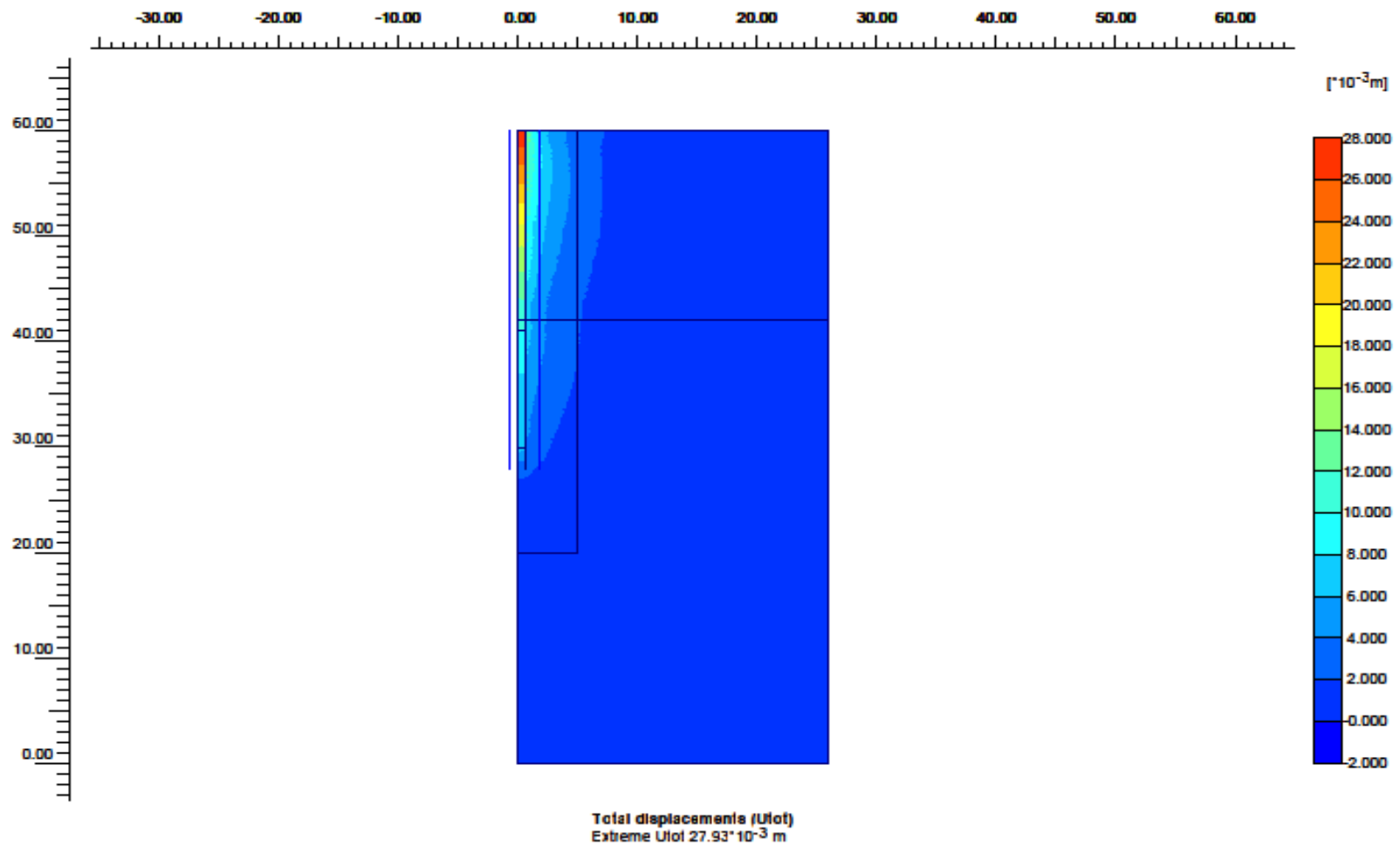


Figure 3.46. Total displacements of case3(static).

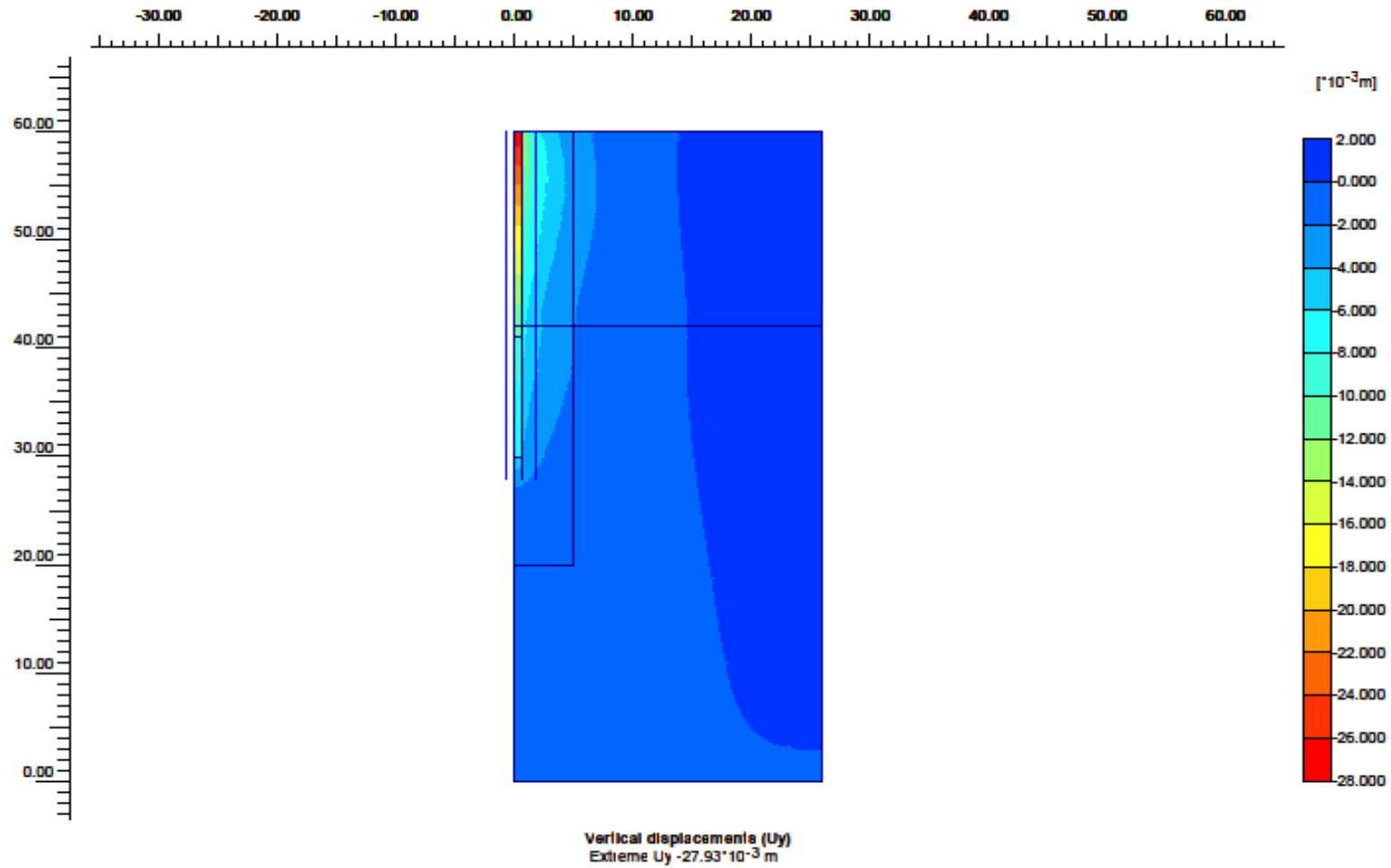


Figure 3.47. Vertical displacements of case3(static).

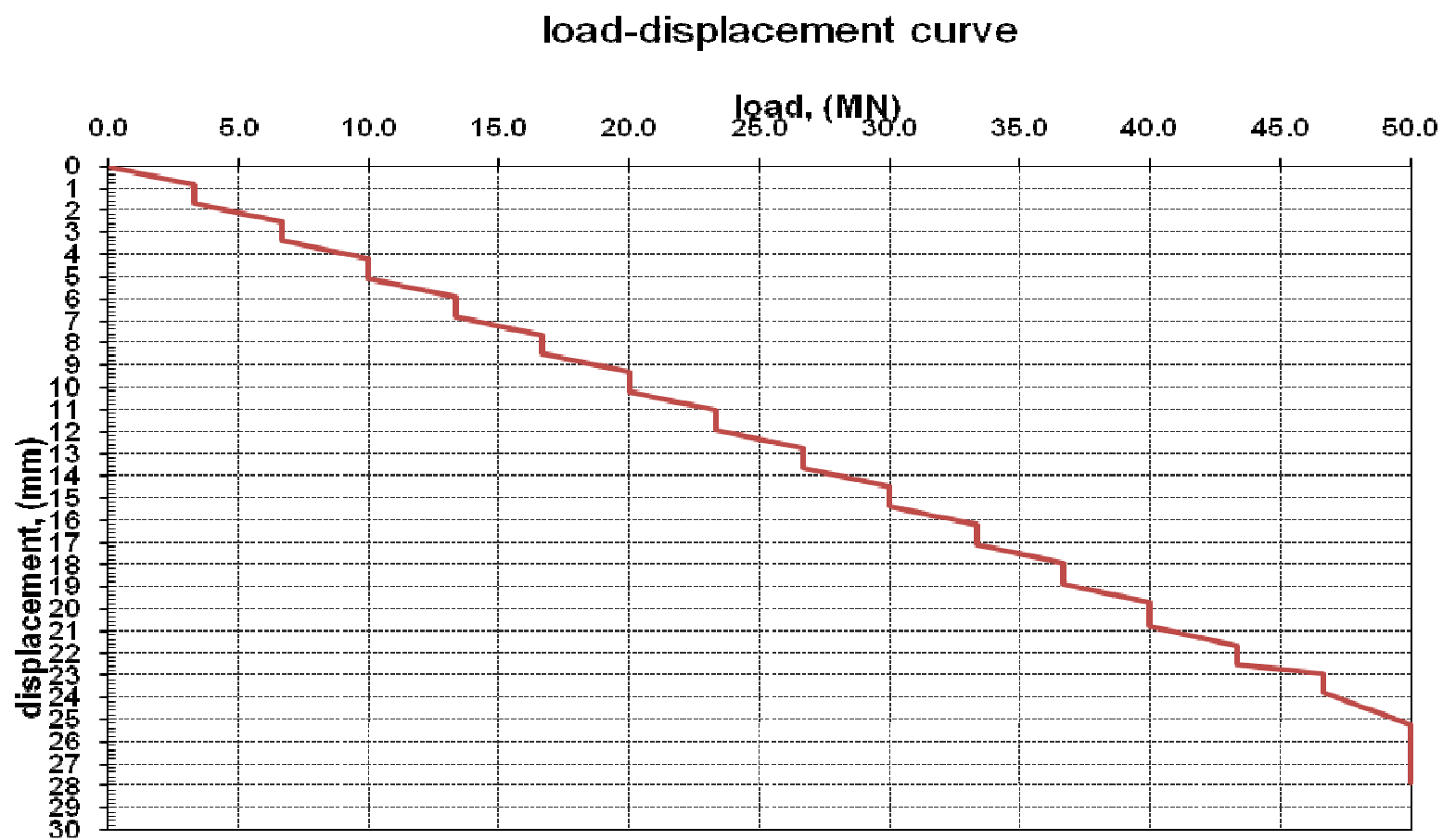


Figure 3.48. Load versus Displacement curve of case3(static).

3.4.4.6. Stress Distribution Along the Pile Shaft. Stress distribution is given in the following figure.

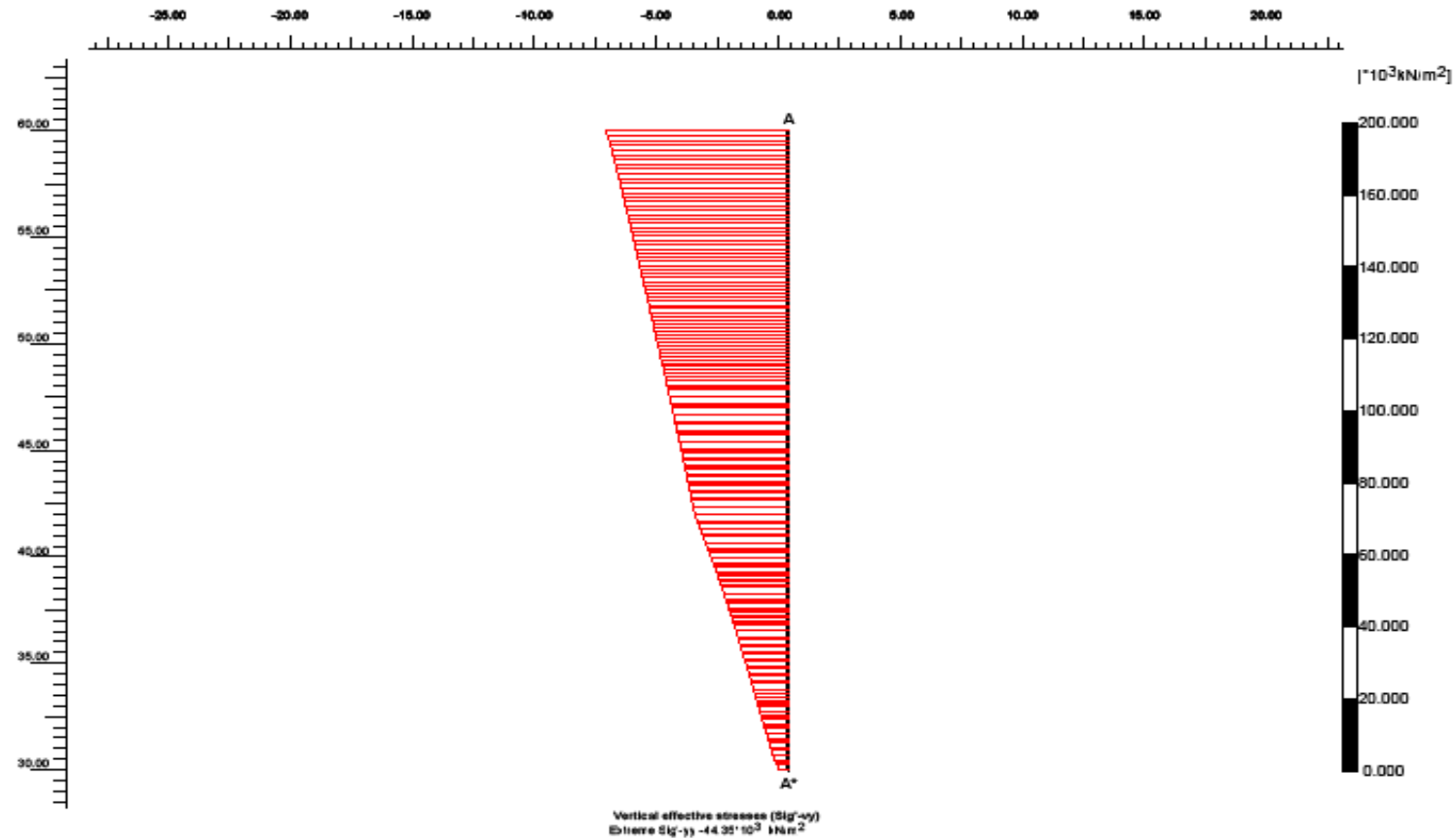


Figure 3.49. Vertical stress distribution along the pile shaft cross-section from the top to the bottom.

Obtained from Figure 3.49 in – 18 meters stress is $-22,709.6 \text{ kN/m}^2$ and in – 19 meters it is $-20,721.7 \text{ kN/m}^2$, in – 30 meters -661.759 and $-44,350.9 \text{ kN/m}^2$ at the top.

Combining the data from Figure 3.49 gives the curve given in Figure 3.50 in which decreasing stress distribution with depth can be seen.

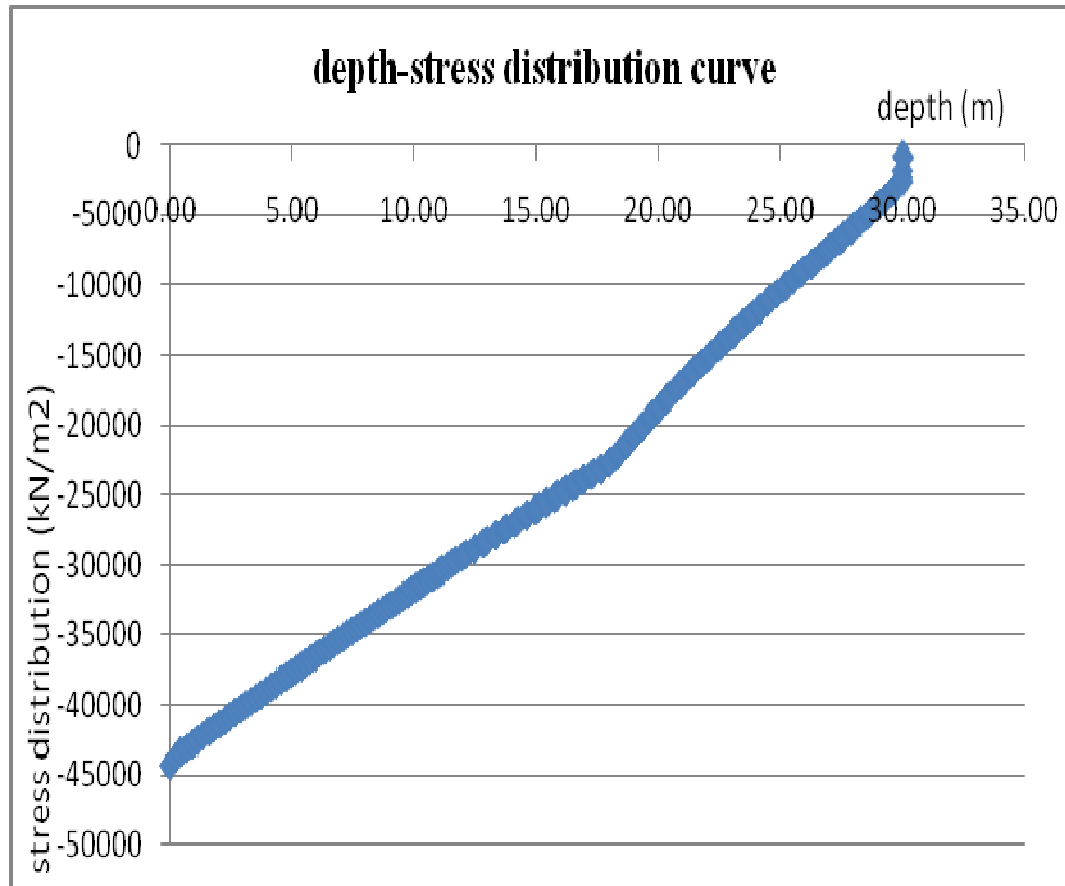


Figure 3.50. Depth(m) versus Stress(kN/m²) distribution curve of pile shaft cross-section from the top to the bottom.

Stress change within a specific depth can be obtained from the Figure 3.44 and used to calculate cohesion within this specific depth by dividing the stress change to frictional resistance area. The result of the calculations is given in Table 3.10, in which the difference between calculated and assumed cohesion values can be seen. This difference stems from that I assumed an average cohesion value for a specific depth with the help of literature data while effective stress was changing within depth.

Table 3.10. Calculated and assumed cohesion values within depth.

depth	stress change	$c_{\text{calculated}}$	c_{assumed}
m	kN/m ²	kN/m ²	kN/m ²
0-18	44,350.9-22,709.6	360	350
18-30	22,709.6-661.76	551	600

3.4.4.7. Comparison of O-Cell and Static Loading Data. Both curves are given in the following figure.

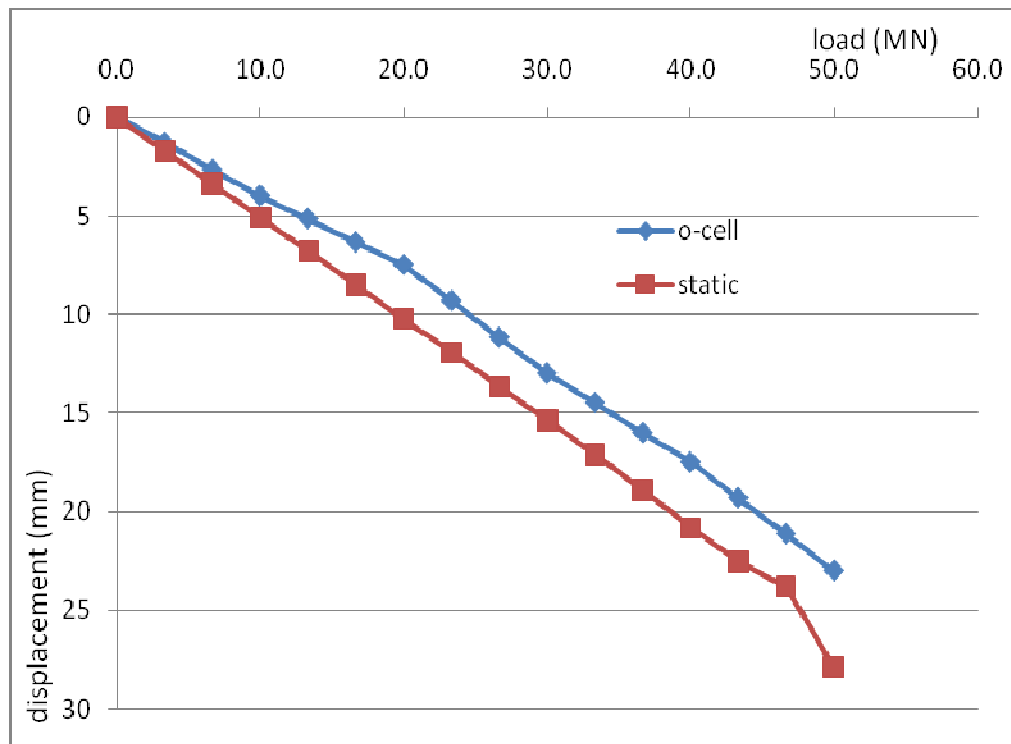


Figure 3.51. Simulated load vs. displacement curve comparison of case 3.

From the preceding FEM study, static loading test` displacement values are higher than O-Cell test` results. In addition, one can see that by O-Cell load test increase of displacement values gets higher after 20 MN. On the other hand, by static loading increase of displacement values are in the same rate. O-cell test results can provide not only the same soil-pile interaction information as conventional head-down static loading test, but also allow for separate determination of the shaft resistance and end bearing components.

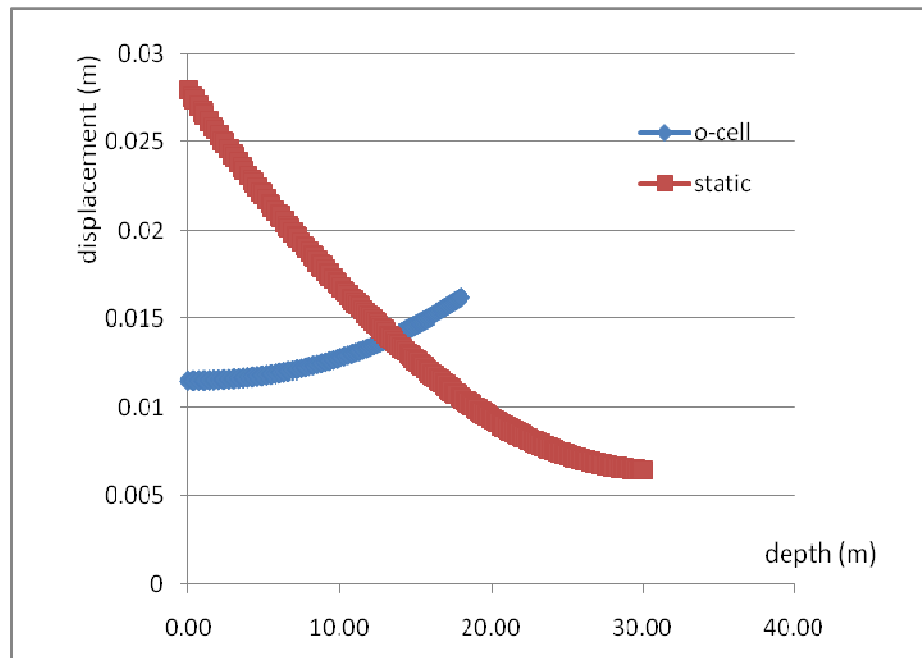


Figure 3.52. Depth (m) versus Displacement (m) Curve of case3.

The FEM computation indicates that the shaft resistance-movement curve for upward movement of the pile is fairly comparable with the downward shaft resistance-movement component of a conventional head-down test according to the Figure 3.52, in which from top to bottom cross-section for conventional test and from top to O-Cell for O-Cell test were used. The result of undrained analysis suggests that a little difference of mobilized shaft resistance may exist between O-cell test and conventional test in the short-term. The reason is that the shaft resistance is lower in the upper strata of soil. By O-Cell tests, because loading is from O-Cell to the top, which means from higher shaft resistance to lower one, small effective stresses occur; unlike conventional tests.

At the smaller deflection, the end bearing of O-cell test increases faster than the conventional static test. One possible reason is that for the head-down test, the load transfer from the top to the toe is unable to effectively activate the end bearing and relies heavily on the side shear resistance; the O-cell however is able to immediately activate the end bearing and therefore gives a faster response. As the test progresses, the conventional test is able to activate the end bearing when the shaft resistance becomes fully mobilized, Therefore the end bearing becomes closer to that of O-cell test thereafter.

Conventional tests show less stiff elastic response and lower ultimate capacity than O-Cell tests. The end-bearing is always mobilized more lately at the same displacement for the conventional test than the O-cell tests. The reason for these is that the source in O-Cell test is near to the toe, so frictional resistance can be overcome to get toe resistance by small loading conditions when compared to conventional tests. By conventional tests, load is applied from the top and there is a 30 meters long frictional resistance to overcome, so one can see the less stiff response, given in Figure 3.53, in which from top to bottom cross-section for conventional test and from O-Cell to bottom cross-section for O-cell test were used.

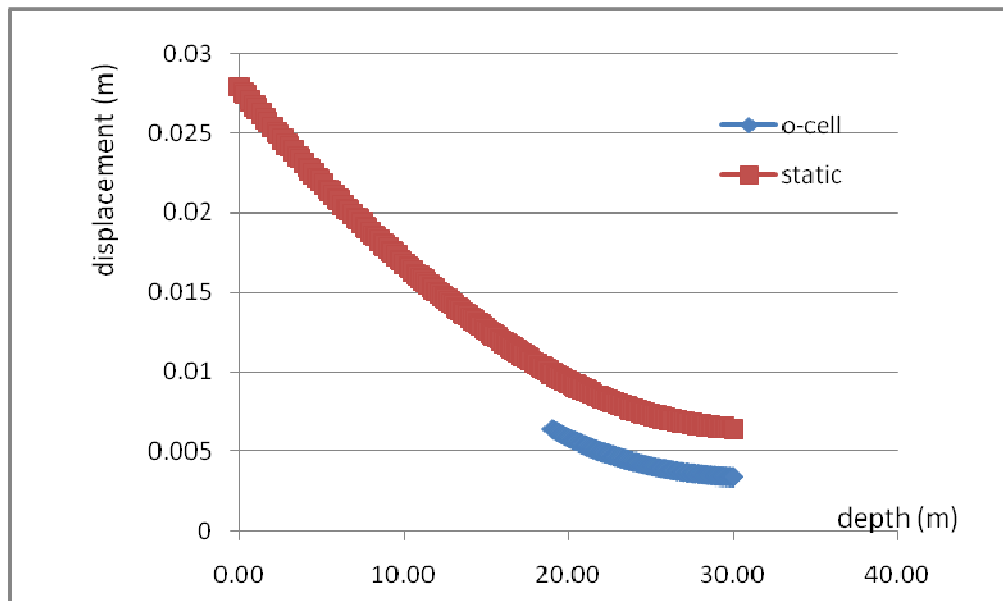


Figure 3.53. Depth (m) – Displacement (m) Curve of case3.

3.5. Case Study 4

3.5.1. FEM Study on Case Study 4

The idealized finite element model of static pile load test is simulated with finite element analysis and illustrated in Figure 3.54, which is the model of conventional top load test. The test is simulated in Mohr Coulomb soil model with 15 node elements; the mesh is

PROJE/PROJECT : SK-6							SONDAJ METODU/BORING METHOD : Karotlu Sondaj								
SONDAJ YERİ/BOREHOLE LOCATION : BARTIN							BAŞLAMA TARİHİ/DATE COMMENCED : 05.04.2009								
SONDAJ KOTU/GROUND LEVEL : 10 m							BİTİŞ TARİHİ/DATE COMPLETED : 05.04.2009								
SONDAJ DERİNLİĞİ/BOREHOLE DEPTH : 50 m							YERALTI SU SEVİYESİ/ G.W.LEVEL : 5.50 m								
KOORDİNATLAR/COORDINATES 1/2000				442373 4612006											
Derinlik (m) Depth (m)	Numune	Manevra Boyu (m) Run	STANDART PENETRASYON DENEYİ Standart Penetration Test				LİTOLOJİ Lithology	JEOTEKNİK TANIMLAMA Geotechnical Description	Dayanım Gücü Strength	Ayrışma Weathering	Kırılma İndisi, m Fracture Index, m	Karot Verimi, % Core Recovery, %	RQD, %		
			Darbe Sayısı Number of Blows			GRAFIK/Graph									
			0-15cm	15-30cm	30-40cm									N	
1	SPT	1,5	1	3	4	7									
2															
3	UD	1,5	2	3	5	8									
4															
5		1,5	1	1	1	2									
6	SPT	1,5	1	2	1	3									
7															
8	UD	1,5	3	1	2	3									
9															
10		1,5	2	3	3	6									
11															
12	SPT	1,5	3	2	4	6									
13		1,5	2	4	5	9									
14		1,5	3	2	2	4									
15															
Dayanım Gücü-Strength			Ayrışma-Weathering				İnce taneli-fine grained			İri Taneli-Coarse					
I Çok Zayıf (very weak) II Zayıf (weak) III Orta Dayanımlı (mod strong) IV Dayanımlı (strong) V Çok Dayanımlı (very strong) VI Aşırı Dayanımlı (extrem strong)			I Taze (fresh) II Az ayrılmış (slightly weath) III Orta derecede ayrılmış (mod weath) IV Çok ayrılmış (highly weath) V Tamamen ayrılmış (comp. Weath)				N = 0 - 2 Çok Yumuşak (very soft) N = 3 - 5 Yumuşak (soft) N = 6 - 9 Orta katı (medium) N = 10 - 16 Katı (soft) N = 17 - 30 Sert (hard)			N = 0 - 4 Çok Gevşek (very loose) N = 4 - 10 Gevşek (loose) N = 10 - 30 Orta Sıkı (medium dense) N = 30 - 50 Sıkı (dense) N = > 50 Çok Sıkı (hard)					
Kaya Kalitesi-RQD			Kırıklar-Fractures				SPT : Standart Penetrasyon Deneyi (standart penetration test) D : Örselenmiş Numune (disturbed sample) UD : Örselenmemiş Numune (undisturbed sample) K : Karot Numunesi (core sample) P : Presiyometri Deneyi (pressuremeter test) VST : Vane Deneyi (vane shear test)								
% 0 - 25 Çok kötü (very poor) % 25 - 50 Kötü (poor) % 50 - 75 Orta (fair) % 75 - 90 İyi (good) % 90 -100 Çok iyi (excellent)			< 1 Seyrek (wide) 1 - 2 Orta (moderate) 2 - 10 Sık (close) 10 - 20 Çok sıkı (intense) > 20 Parçalı (crushed)				W M CL I CR								
Sondaj Mühendisi (drilling engineer) :														İmza:	

Figure 3.55. Boring Log Page 1.

PROJE/PROJECT : SK-6						SONDAJ METODU/BORING METHOD : Karotlu Sondaj											
SONDAJ YERİ/BOREHOLE LOCATION : BARTIN						BAŞLAMA TARİHİ/DATE COMMENCED : 05.04.2009											
SONDAJ KOTU/GROUND LEVEL : 10 m						BİTİŞ TARİHİ/DATE COMPLETED : 05.04.2009											
SONDAJ DERİNLİĞİ/BOREHOLE DEPTH : 50 m						YERALTI SU SEVİYESİ/ G.W.LEVEL : 5.50 m											
KOORDİNATLAR/COORDINATES 1/25 000				442373 4612006													
Derinlik (m) Depth (m)	Numune	Manevra Boyut (m) Run	STANDART PENETRASYON DENEYİ Standart Penetration Test					LİTOLOJİ Lithology	JEOTEKNİK TANIMLAMA Geotechnical Description	Dayanım Strength	Ayrışma Weathering	Kırılma İndisi, m Fracture Index, m	Karot Verimi, % Core Recovery, %	RQD, %			
			Darbe Sayısı Number of Blows			GRAFIK/Graph											
			0-15cm	15-30cm	30-40cm	N	10								20	30	40
16	SPT	1,5	3	2	2	4						CL					
17		1,5	2	2	4	6											
18		1,5	1	1	3	4											
19							2	4	4	8							
20		1,5	3	3	3	6											
21		1,5	3	4	7	11											
22							2	3	5	8							
23		1,5	5	5	7	12											
24		1,5	4	5	7	12											
25							6	5	8	13							
26																	
27																	
28																	
29																	
30																	
Dayanım Gücü-Strength			Ayrışma-Weathering			İnce taneli-fine grained			İri Taneli-Coarse								
I Çok Zayıf (very weak) II Zayıf (weak) III Orta Dayanımlı (mod strong) IV Dayanımlı (strong) V Çok Dayanımlı (very strong) VI Aşırı Dayanımlı (extrem strong)			I Taze (fresh) II Az ayrılmış (slightly weath) III Orta derecede ayrılmış (mod weath) IV Çok ayrılmış (highly weath) V Tamamen ayrılmış (comp. Weath)			N = 0 - 2 Çok Yumuşak (very soft) N = 3 - 5 Yumuşak (soft) N = 6 - 9 Orta katı (medium) N = 10 - 16 Katı (soft) N = 17 - 30 Sert (hard)			N = 0 - 4 Çok Gevşek (very loose) N = 4 - 10 Gevşek (loose) N = 10 - 30 Orta Sıkı (medium dense) N = 30 - 50 Sıkı (dense) N = > 50 Çok Sıkı (hard)								
Kaya Kalitesi-RQD			Kırıklar-Fractures			SPT : Standart Penetrasyon Deneyi (standart penetration test) D : Örselenmiş Numune (disturbed sample) UD : Örselenmemiş Numune (undisturbed sample) K : Karot Numunesi (core sample) P : Presiyometri Deneyi (pressuremeter test) VST : Vane Deneyi (vane shear test)											
% 0 - 25 Çok kötü (very poor) % 25 - 50 Kötü (poor) % 50 - 75 Orta (fair) % 75 - 90 İyi (good) % 90 -100 Çok iyi (excellent)			< 1 Seyrek (wide) W 1 - 2 Orta (moderate) M 2 - 10 Sık (close) CL 10 - 20 Çok sıkı (intense) I > 20 Parçalı (crushed) CR														
Sondaj Mühendisi (drilling engineer) :													İmza:				

Figure 3.56. Boring Log Page 2.

PROJE/PROJECT		: SK-6		SONDAJ METODU/BORING METHOD		: Karotlu Sondaj															
SONDAJ YERİ/BOREHOLE LOCATION		: BARTIN		BAŞLAMA TARİHİ/DATE COMMENCED		: 05.04.2009															
SONDAJ KOTU/GROUND LEVEL		: 10 m		BİTİŞ TARİHİ/DATE COMPLETED		: 05.04.2009															
SONDAJ DERİNLİĞİ/BOREHOLE DEPTH		: 50 m		YERALTI SU SEVİYESİ/ G.W.LEVEL		: 5.50 m															
KOORDİNATLAR/COORDINATES 1/25 000		442373 4612006																			
Derinlik (m) Depth (m)	Numune	Manevra Boyut (m) Run	STANDART PENETRASYON DENEYİ Standard Penetration Test				LİTOLOJİ Lithology	JEOTEKNİK TANIMLAMA Geotechnical Description	Dayanım Strength	Ayrışma Weathering	Kırılma İndisi Fracture Index, m	Korut Verim % Core Recovery %	RQD, %								
		Darbe Sayısı Number of Blows		GRAFIK/Graph																	
		0-15cm	15-30cm	30-40cm	N	10								20	30	40	50				
32	SPT	5	6	6	7	13						CL									
34																					
36		5	5	7	8	15															
38																					
40	SPT	10	6	7	8	15						CL									
42																					
44																					
46																					
48	SPT											CL									
50																					
Dayanım Gücü-Strength			Ayrışma-Weathering			İnce taneli-fine grained			İri Taneli-Coarse												
I Çok Zayıf (very weak) II Zayıf (weak) III Orta Dayanım (mod strong) IV Dayanım (strong) V Çok Dayanım (very strong) VI Aşırı Dayanım (extrem strong)			I Taze (fresh) II Az ayrılmış (slightly weath) III Orta derecede ayrılmış (mod weath) IV Çok ayrılmış (highly weath) V Tamamen ayrılmış (comp. Weath)			N = 0 - 2 Çok Yumuşak (very soft) N = 3 - 5 Yumuşak (soft) N = 6 - 9 Orta katı (medium) N = 10 - 16 Kırılgan (soft) N = 17 - 30 Sert (hard)			N = 0 - 4 Çok Gevşek (very loose) N = 4 - 10 Gevşek (loose) N = 10 - 30 Orta Sıkı (medium dense) N = 30 - 50 Sıkı (dense) N = > 50 Çok Sıkı (hard)												
Kaya Kalitesi-RQD			Kırılgan-Fractures			SPT : Standart Penetrasyon Deneyi (standart penetration test) D : Örselemiş Numune (disturbed sample) UD : Örselememiş Numune (undisturbed sample) K : Karot Numunesi (core sample) P : Presiyometri Deneyi (pressuremeter test) VST : Vane Deneyi (vane shear test)															
% 0 - 25 Çok kötü (very poor) % 25 - 50 Kötü (poor) % 50 - 75 Orta (fair) % 75 - 90 İyi (good) % 90 -100 Çok iyi (excellent)			< 1 Seyrek (wide) 1 - 2 Orta (moderate) 2 - 10 Sık (close) 10 - 20 Çok sıkı (Intense) > 20 Parçalı (crushed)			W M CL I CR															
Sondaj Mühendisi (drilling engineer) :															İmza:						

Figure 3.57. Boring Log Page 3.

AKADEMİ ZEMİN MEKANİĞİ LABORATUVARI
Yeni Başkent Sanayi Sitesi 248/7 Sokak No: 65 Batıkent - ANKARA
Tel:0(312)278 48 11, Faks:0(312)278 48 10

SONUÇLAR DENEY RAPORU

FİRMA : TESFAM TEMEL SU JEOTEKNİK YERALTISULARI MÜHENDİSLİK HİZMETLERİ LTD.ŞTİ.
NUMUNE YERİ : Bartın Çimento Sanayi ve Ticaret A.Ş. Sahası
PROJE : Jeolojik - Jeoteknik Etüt

Sayfa No: 08,04,2009
Rapor Tarihi: 17,04,2009
Lab. Kayıt No: C / 1158

Çukur / Sondaj No.	Numune No.	Derinlik (m)	Su İçeriği (%)	Doğal Birim Hacim Ağırlık (kN/m ³)	Hidrometre	Elek Analizi		Atterberg Limitleri			Zemin Sınıfı	Tek Eksenli Basınç Deneyi		Direk Kesme Deneyi		Üç Eksenli Basınç Deneyi (UU)		Konsolidasyon Deneyi	
						No. 4 Kalan (%)	No. 200 Geçen (%)	LL (%)	PL (%)	PI (%)		q _u (kgf/cm ²)	c (kgf/cm ²)	φ (°)	c (kgf/cm ²)	φ (°)	Şişme Yüzdesi	Şişme Basıncı (kgf/cm ²)	
SK-6	SPT	3,00	28,9			0,0	89,6	38,3	19,7	18,6	CL								
"	UD	4,00	22,4	18,16	✓	1,4	72,9	35,5	18,9	16,6	CL					0,48	4		
"	SPT	4,50	24,2			0,0	89,6	35,1	17,7	17,4	CL								
"	SPT	6,00	26,9			1,9	76,3	36,4	16,4	20,0	CL								
"	SPT	7,50	25,1			0,0	83,2	33,4	17,0	16,4	CL								
"	UD	8,50	21,6	18,34	✓	0,0	84,2	31,1	18,7	12,4	CL					0,54	5	0,62	
"	SPT	9,00	24,2			0,0	91,9	36,7	18,9	17,8	CL								
"	SPT	10,50	28,1			0,8	74,1	29,3	15,2	14,1	CL								
"	SPT	12,00	23,7			0,0	77,9	37,1	18,9	18,2	CL								
"	SPT	13,50	28,0			0,0	84,2	32,4	17,2	15,2	CL								
"	SPT	15,00	27,8			2,1	72,7	24,7	14,6	10,1	CL								
"	SPT	16,50	27,9			1,2	73,2	29,6	15,2	14,4	CL								
"	SPT	18,00	28,2			1,4	72,3	31,1	20,0	11,1	CL								
"	SPT	19,50	28,2			0,0	83,2	30,4	19,7	10,7	CL								
"	SPT	21,00	27,8			0,0	85,3	29,7	16,8	12,9	CL								
"	SPT	22,50	28,3			0,0	84,6	29,3	16,9	12,4	CL								
"	SPT	24,00	27,3			0,0	85,1	32,4	16,8	15,6	CL								
"	SPT	25,50	27,5			0,5	81,4	33,3	18,7	14,6	CL								
"	SPT	27,00	28,1			1,2	77,5	23,1	13,2	9,9	CL								
"	SPT	28,50	28,8			0,7	76,9	26,8	14,0	12,8	CL								
"	SPT	30,00	28,6			0,0	79,1	30,1	17,8	12,3	CL								
"	SPT	35,00	28,7			0,0	87,5	24,6	14,0	10,6	CL								
"	SPT	40,00	27,6			1,2	72,9	25,8	14,6	11,2	CL								
"	SPT	50,00	28,7			0,7	78,6	31,2	16,7	12,5	CL								
"	SPT	52,00	21,6			5,9	62,2	24,3	12,4	11,9	CL								
"	UD	54,00	23,2	18,63		6,4	57,4	25,2	13,1	12,1	CL					0,71	9		

NP=Non-plastik Deney Sorumlusu
Jeoloji Müh. Sertan DEMİR

*Adres bilgileri müşteri beyanıdır.
*Söz konusu deney sonuçları, sadece test edilen deney numunelerine aittir.
*Deney sonuçları laboratuvarımız izni olmadan kimsen kopyalanamaz ve çoğaltılamaz.
*Laboratuvarımız 4708 Sayılı Kanun Gereği Bayındırlık ve İskan Bakanlığı Yapı İşleri Genel Müdürlüğü tarafından verilen 17/10/2003 tarih ve 67 No' lu Laboratuvar İzin Belgesine Sahiptir.

Lab. Denetçi Müh.
Jeoloji Müh. Sertan DEMİR
AKADEMİ ZEMİN MEKANİĞİ LABORATUVARI
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FORM:29

Figure 3.58. Lab Report.

According to the boring log the Layer 1 is taken from the ground level +0,00 to -4,50 m , the Layer 2 is taken from – 4,50 m to – 9,00 m, Layer 3 which has the same properties with Layer 1 is taken from -9,00 m to -14,00 m, Layer 4 which has the same properties with Layer 2 is taken from -14,00 m to 19,50 m and finally Layer 5 which is also has the same properties with Layer 1 is taken from -19,50 m to 60,00 m. Ground water level starts from -3.5 meters.

For soil layer1, 3 and 5; N is assumed as 11 and undrained cohesion was found as 70 kPa. according to the formula (3.2). Young` s Modulus` assumption was made according to the formula (3.1) and found 20,000 kN/m².

For soil layer 2 and 4, N is assumed as 6.5 and undrained cohesion was found as 40 kPa. according to the formula (3.2). Young's Modulus assumption was made according to the formula (3.1) and found 12,000 kN/m².

Poisson ratio is generally taken between 0.3 and 0.35 for soils as drained parameters in undrained analysis and 0.2 for concrete in Plaxis Teaching Manuals. Saturated soil's poisson ratio 0.5-or values around 0.5- was used by simulation; however, finite element code gave error during the calculation phase consistent with the teaching manual, so for all layers it was assumed 0.33. In addition, Young's Modulus of concrete was assumed as 25,000,000 kN/m² according to the formula (3.3) because C25 type of concrete was used in the pile.

Table 3.11 lists the parameters of material parameters according to the boring log, seen by Figure 3.55, Figure 3.56, Figure 3.57 and Figure 3.58.

Table 3.11. Material Properties of the FEM Model.

Name	Depth	Type	Model	γ_{unsat}	γ_{sat}	ν	E_{ref}	c_{ref}
	m			kN/m ³	kN/m ³	-	kN/m ²	kN/m ²
Soil layer 1	0-4,5	Undrained	M-C	17	18	0.33	20000	70
Soil layer 2	4,5-9	Undrained	M-C	17	18	0.33	12000	40
Soil layer 3	9-14	Undrained	M-C	17	18	0.33	20000	70
Soil layer 4	14-19,5	Undrained	M-C	17	18	0.33	12000	40
Soil layer 5	19,5-60	Undrained	M-C	17	18	0.33	20000	70
Concrete	n/a	Elastic	L-E	24	24	0.2	2.50E+07	-

The interface element is applied between piles and soil, the strength reduction factor R_{inter} is assumed to be 0.7. The definition of parameters can be referred to Section 2.3.2.

3.5.1.2. Results. In the following Figure 3.62 and Figure 3.63 displacement behaviors of numerically simulated and site measured case 4 under loading are given respectively. These two data are combined on the same template to show the similarities and differences between them which is given in Figure 3.64. The shape of the mobilized displacement curve appear in agreement with the measured one. The low load stages predicted

displacement curve tends to be overestimated for measurement curve. In these small loading stages the values are about two times bigger than the measured values. However, this difference starts to decrease at around 300 tons load. And after 350 tons, measured displacements gets higher than the predicted displacements.

In Appendix D, additional numerical simulation results are given.

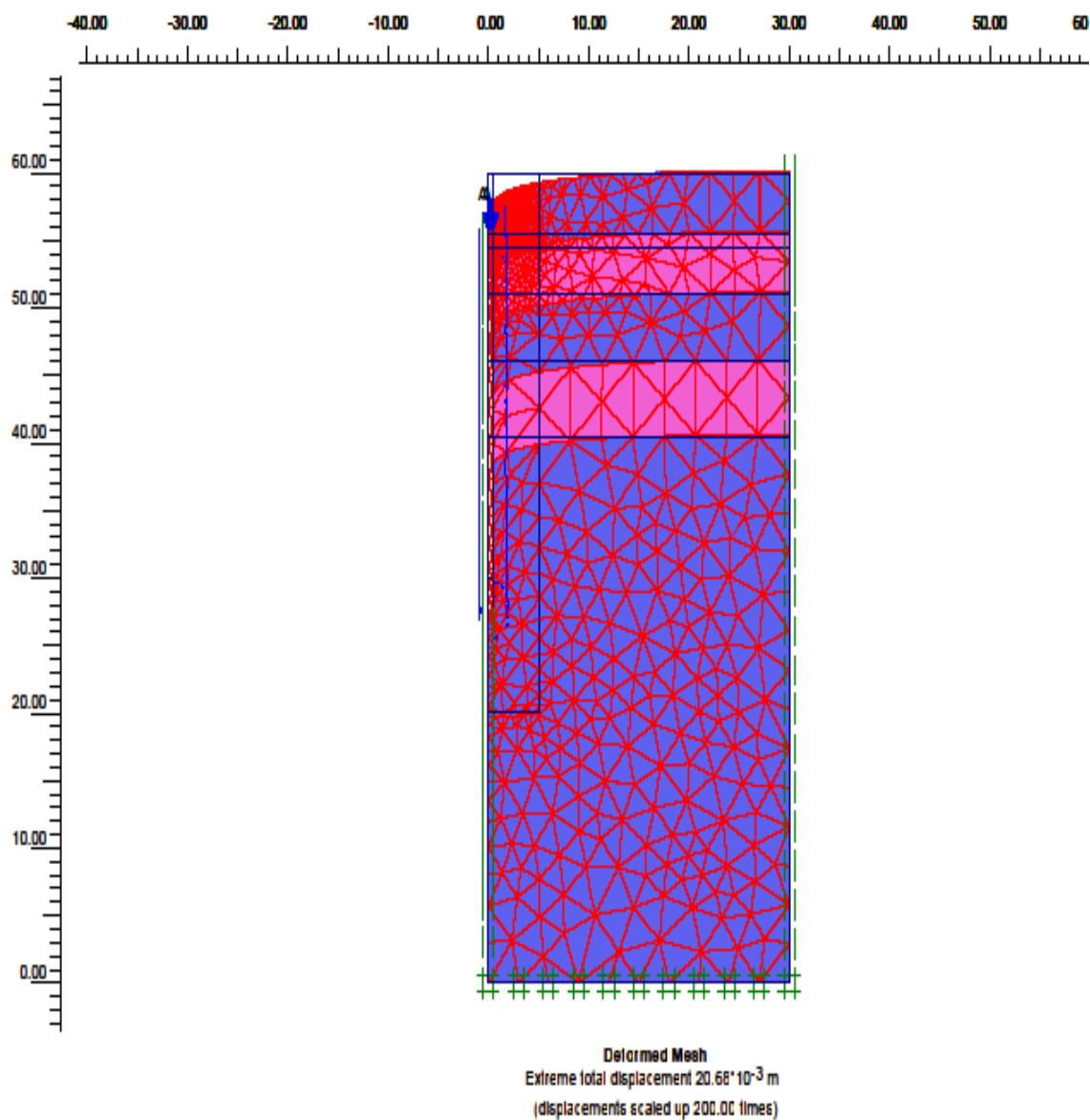


Figure 3.59. Deformed mesh of case4.

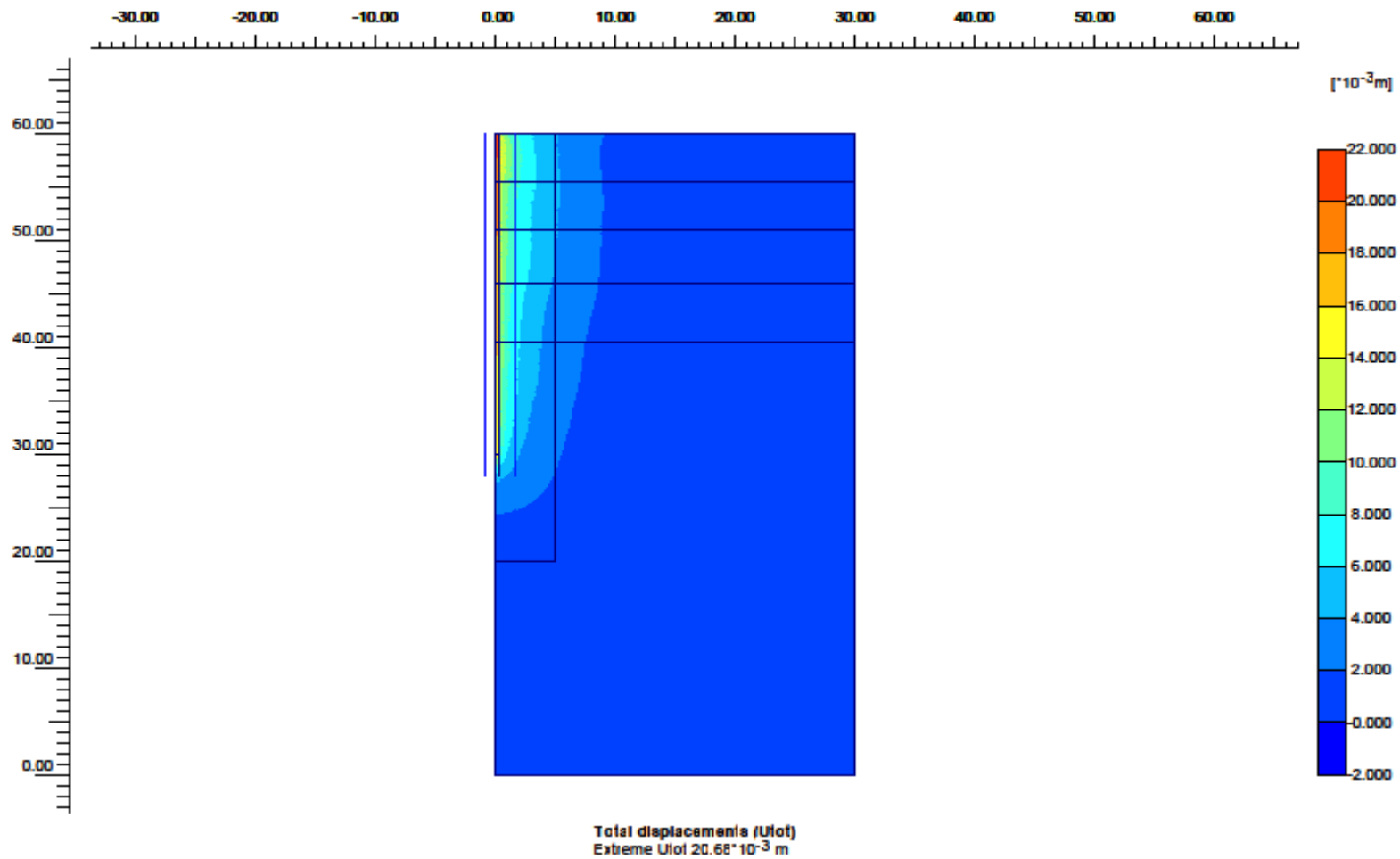


Figure 3.60. Total displacements of case4.

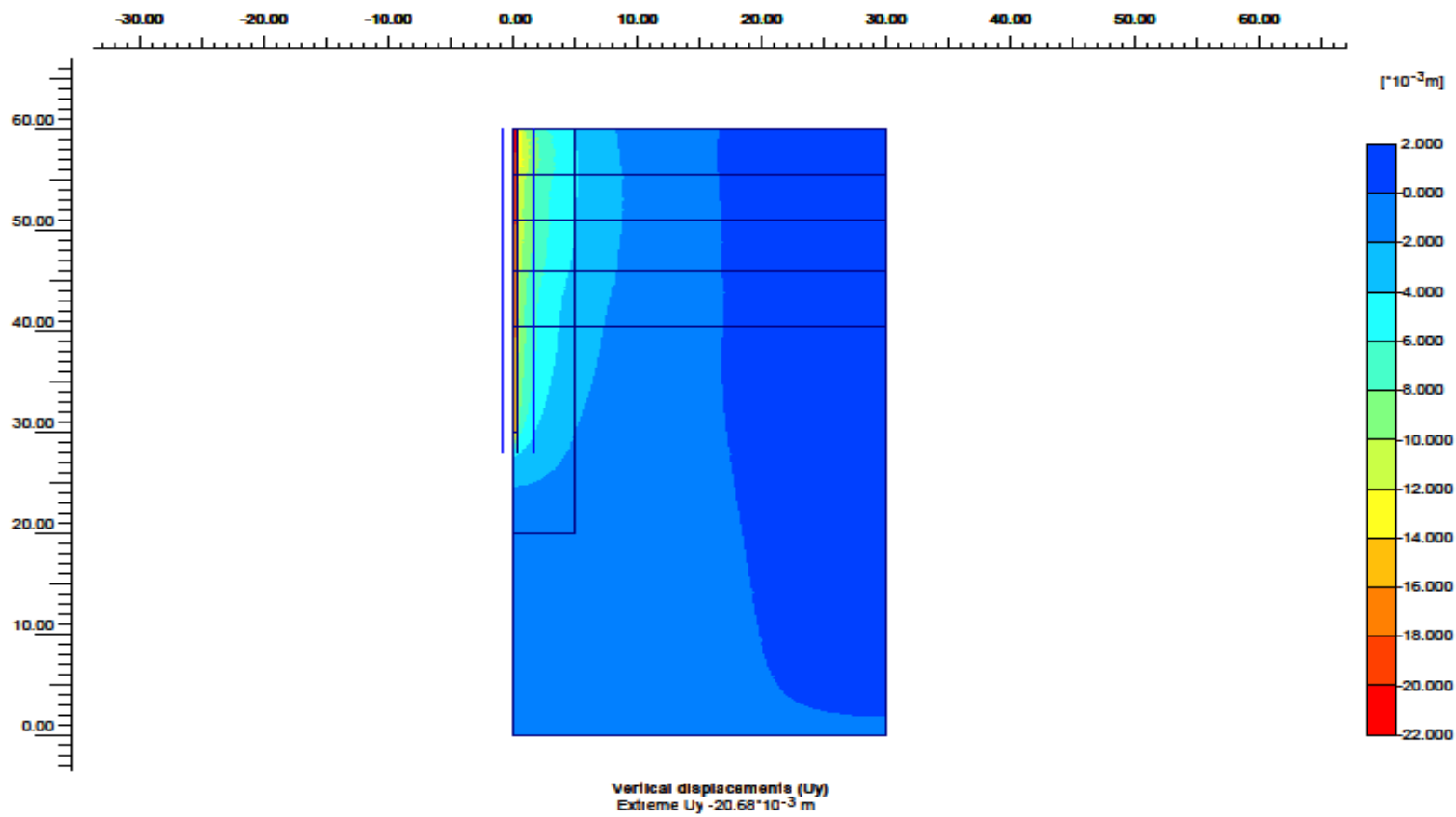


Figure 3.61. Vertical displacements of case4.

3.5.1.4. Site Measured and Numerically Simulated Load versus Displacement Curve of Case4. Both curves are given in the following figure.



Figure 3.64. Site measured(blue) and numerically simulated(red) load versus displacement curve of case4.

3.5.1.5. Stress Distribution Along the Pile Shaft. Stress distribution is given in the following figure.

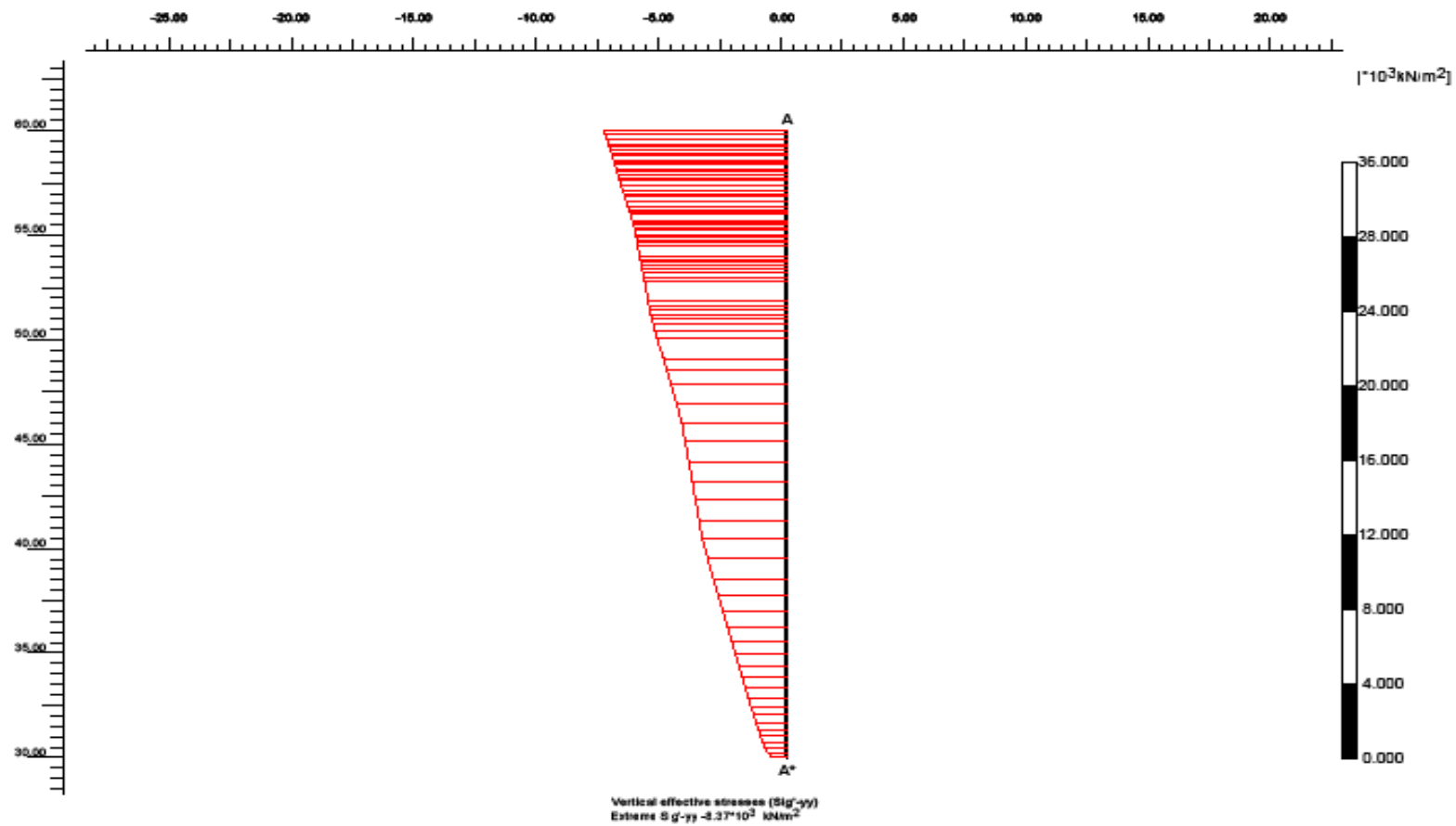


Figure 3.65. Vertical stress distribution along the pile shaft cross-section from the top to the bottom.

Obtained from Figure 3.65 in – 4.5 meters stress is $-7,011.56 \text{ kN/m}^2$ and in – 9 meters it is $-6,182.66 \text{ kN/m}^2$, in – 14 meters $-4,795.44 \text{ kN/m}^2$, in - 19.5 meters $-3,878.81 \text{ kN/m}^2$, in – 30 meters -681.964 kN/m^2 and $-8,365.75 \text{ kN/m}^2$ at the top.

Combining the data from Figure 3.27 gives the curve given in Figure 3.28 in which decreasing stress distribution with depth can be seen.

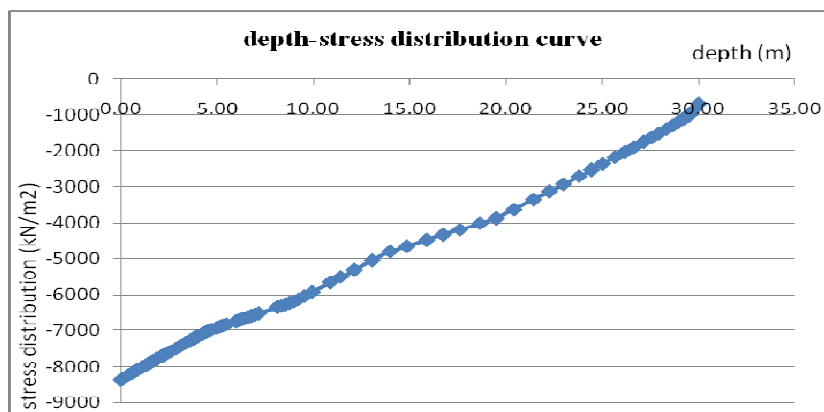


Figure 3.66. Depth(m) versus Stress(kN/m²) distribution curve of pile shaft cross-section from the top to the bottom.

Stress change within a specific depth can be obtained from the Figure 3.66 and used to calculate cohesion within this specific depth by dividing the stress change to frictional resistance area. The result of the calculations is given in Table 3.12, in which the difference between calculated and assumed cohesion values can be seen. This difference stems from that I assumed an average cohesion value for a specific depth with the help of literature data while effective stress was changing within depth.

Table 3.12. Calculated and assumed cohesion values within depth.

depth m	stress change kN/m ²	C _{calculated} kN/m ²	C _{assumed} kN/m ²
0-4.5	8,365.75-7,011.56	59	70
4.5-9	7,011.56-6,182.66	37	40
9-14	6,182.66-4,795.44	56	70
14-19.5	4,795.44-3,878.81	33	40
19.5-30	3,878.81-681.96	61	70

3.6. Case Study 5

3.6.1. FEM Study on Case Study 5

The idealized finite element model of static pile load test is simulated and illustrated in Figure 3.67, which is the model of conventional top load test. The test is simulated in Mohr Coulomb soil model with 15 node elements; the mesh is generated axisymmetrically, with the boundary of 26 meters wide and 52 meters deep. The test pile is in the same size of diameter of 1,0 meter and length of 26 meters. To ensure the precision of the calculation, the meshes of an area of 5 meters wide, 32 meters deep around the test pile are refined.

3.6.1.1. Soil Conditions and Design Parameters. Design parameters are obtained by incorporation of soil boring log which is seen in Figure 3.68, Figure 3.69.

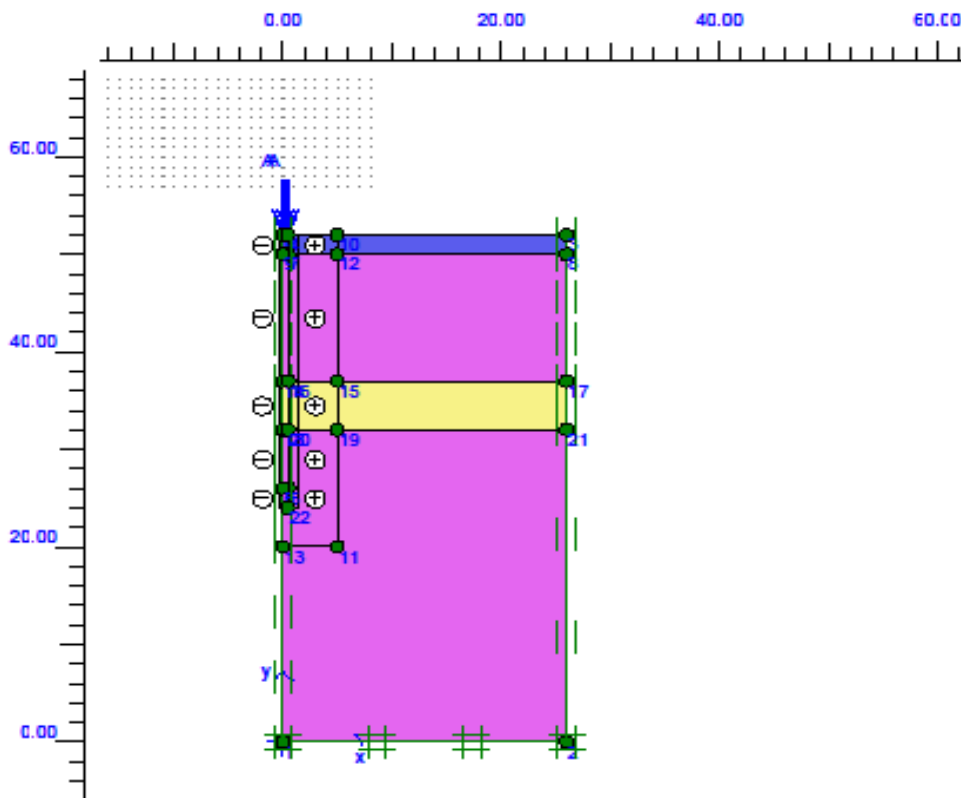


Figure 3.67. Finite element model of case5.

KARDEŞ SONDAJCILIK LTD.ŞTİ.										Sondaj No / Boring No : SK-20		Sayfa No / Sheet No : 1											
SONDAJ LOGU / BORING LOG																							
PROJE ADI / PROJECT NAME : İstanbul İl, Ataşehir İlçesi, 3385 Ada				SONDAJ MAKİNASI / DRILLING RIG : CREALUIS D-750																			
SONDAJ YERİ / BORING LOCATION : 4 Parsel, C3 Blok Otopark Zemin Etüdü				SONDÖR / FOREMAN (1. Vardiya) : Uğur KIRAZ																			
KOORDİNATLAR / COORDINATES : 426138.998 , 4539911.214				SONDÖR / FOREMAN (2. Vardiya) : Cenk KIRAN																			
SONDAJ KOTU / ELEVATION (m) : 96.74 m.				MÜHENDİS / ENGINEER : 23.12.2009																			
SONDAJ DERİNLİĞİ / BORING DEPTH (m) : 25.00 m.				BAŞLAMA TARİHİ / DATE STARTED : 23.12.2009																			
YERALTISUYU / GROUNDWATER (m) : Yok				BİTİŞ TARİHİ / DATE COMPLETED : 23.12.2009																			
MUHAFAZA BORU / CASING SIZE : Dış ÇAP / O.D.(mm):				İÇ ÇAP / I.D. (mm):				UZUNLUĞU / LENGTH (m):															
DERİNLİK DEPTH (m)	KAROT ve %'si (CORE % RECOVERY)	R.O.D. %	NUMUNE NO. (SAMPLE NO.)	NUMUNE DERİNLİĞİ (SAMPLE DEPTH.)	ZEMİN TANIMLAMASI (SOIL DESCRIPTION)	ZEMİN PROFİLİ (SOIL PROFILE)	DAYANIKLILIK (STRENGTH)	AYRIŞMA (WEATHER.)	KIRIK / 30 cm (FRAC./30 cm.)	KIRIK AÇISI (FRA. ANG.)	SU KAYBI % (WAT. LOSS)	STANDART PENETRASYON DENEYİ STANDARD PENETRATION TEST											
												Darbe Sayısı No. of Blows			N ₆₀	GRAFİK GRAPH							
												0-15	15-30	30-45									
1					<u>Dolgu</u> : Yerel ve inşaat atığı malzemeden oluşmuş killi, kumlu, çakıllı ve bloklu seviyedir.							4	8	10	18								
2					2.10 m.																		
3			SPT-2	3.00-3.45	<u>Sev1</u> : Sarımsı kahve, yer yer morumsu - boz renkte, ileri derecede tamamen ayrılarak çok katı kıvamda kil görünümü kazanmış, üst seviyelerde (1.00 m.) sık, alt seviyelerde seyrek köşeli, feldspat, mika ve kuvars çakıllı seviyedir.							9	11	17	28								
4																							
5			SPT-3	4.50-4.95								10	14	19	33								
6																							
7			SPT-4	6.00-6.45								9	15	21	36								
8																							
9			SPT-5	7.50-7.95								13	18	23	41								
10																							
11			SPT-6	9.00-9.45								11	18	26	44								
12																							
13			SPT-7	10.50-10.95								14	19	27	46								
14																							
15			SPT-8	12.00-12.45	<u>Sev2</u> : Sarımsı kahve, yer yer morumsu boz renkte, ileri derecede tamamen ayrılarak sert kıvamda kil görünümü kazanmış, seyrek, köşeli, feldspat, mika ve kuvars çakıllı seviyedir.							21	27	31	>50								
16																							
17			SPT-9	13.50-13.95								18	29	36	>50								
18																							
19																							
20			SPT-10	15.00-15.45								23	38	44	>50								
KIVAM DURUMU / STIFFNESS												SİKLİLİK / DENSITY				ORANLAR / PROPORTIONS				KIRIKLAR / 30 cm - FRACTURES / 30 cm.			
N= 0-2 Çok yumuşak V.Soft				N= 0-4 Çok gevşek V.Loose				0-10 % Pök az Trace				> 1 Seyrek Wide (W)											
N= 3-4 Yumuşak Soft				N= 5-10 Gevşek Loose				10-20 % Az Little				1-2 Orta Moderate (M)											
N= 5-8 Orta Kati M.Stiff				N= 11-30 Orta sıkı M.dense				20-35 % Sıfat Adjective(Or some)				2-10 Sık Close (C)											
N= 9-15 Kati Stiff				N= 31-50 Sıkı Dense				35-50 % Ve And				10-20 Çok sık Intense (I)											
N= 16-30 Çok kati V.stiff				N> 50 Çok sıkı V.dense								> 20 Parçali Crushed (Cr)											
N> 30 Sert Hard																							
DAYANIMLILIK / STRENGTH												AYRIŞMA / WEATHERING				KAYA KALİTESİ TANIMI / RQD.				KISALTIMALAR			
I Çok zayıf Very weak				I Tamamen ayrılmış Comp.weathered				0-25 % Çok zayıf Very poor				UD : Örselenmemiş Numune / Undisturbed Sample											
II Zayıf Weak				II Çok ayrılmış Highly weathered				25-50 % Zayıf Poor				D : Örselenmiş Numune / Disturbed Sample											
III Orta zayıf M.weak				III Orta ayrılmış Mod. weathered				50-75 % Orta Fair				SPT : Standart Penetrasyon Deneyi / Standart Pen.Test											
IV Orta dayanımlı M.strong				IV Az ayrılmış Slightly weathered				75-90 % İyi Good				VST : Vane Deneyi / Vane Test											
V Dayanımlı Strong				V Taze Fresh				90-100 % Çok İyi Excellent				P : Presiyometre deneyi / Pressometer Test											

Figure 3.68. Boring Log Page 1.

KARDEŞ SONDAJCILIK LTD.ŞTİ.

Sondaj No /Boring No. : SK-20

Sayfa No / Sheet No. : 2

Proje / Project : İstanbul İli, Ataşehir İlçesi, 3385 Ada, 4 Parsel

DERİNLİK DEPTH (m)	KAROT ve % si (CORE % RECOVERY)	R.Q.D. %	NUMUNENÖ. (SAMPLE NO.)	NUMUNE DERİNLİĞİ (SAMPLE DEPTH.)		ZEMİN TANIMLAMASI (SOIL DESCRIPTION)	ZEMİN PROFİLİ (SOIL PROFILE)	DAYANIKLILIK (STRENGTH)	AYRISMA (WEATHERING)	KIRILMA (FRACT. ANGLE)	SU KAYBI % (WATER LOSS)	STANDART PENETRASYON DENEYİ STANDARD PENETRATION TEST																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
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Figure 3.69. Boring Log Page 2.

According to the boring log the Layer 1 is taken from the ground level +0,00 to -2,00 m , the Layer 2 is taken from – 2,00 m to – 15,00 m, Layer 3 is taken from -15,00 m to - 20,00 m, Layer 4 which has the same properties with Layer 2 is taken from -20,00 m to 52,00 m. Ground water level starts from – 26 meters.

For soil layer1, N is assumed as 19 and undrained cohesion was found as 120 kPa. according to the formula (3.2). Young` s Modulus` assumption was made according to the formula (3.1) and found 22,000 kN/m².

For soil layer2 and 4, N is assumed as 43 and undrained cohesion was found as 270 kPa. according to the formula (3.2). Young` s Modulus` assumption was made according to the formula(3.1) and found 100,000 kN/m².

For soil layer3, N is assumed as 50 and undrained cohesion was found as 310 kPa. according to the formula (3.2). Young` s Modulus` assumption was made according to the formula (3.1) and found 140,000 kN/m².

Poisson ratio is generally taken between 0.3 and 0.35 for soils as drained parameters in undrained analysis and 0.2 for concrete in Plaxis Teaching Manuals. Saturated soil`s poisson ratio 0.5-or values around 0.5- was used by simulation; however , finite element code gave error during the calculation phase consistent with the teaching manual, so for all layers it was assumed 0.33. In addition, Young` s Modulus of concrete was assumed as 27,400,000 kN/m² according to the formula(3.3)) because C25 type of concrete was used in the pile.

Table 3.13 lists the parameters of material parameters according to the boring log , seen by Figure 3.68 and Figure 3.69.

Table 3.13. Material Properties of the FEM Model.

Name	Depth	Type	Model	γ_{unsat}	γ_{sat}	v	Eref	c_{ref}
	m			kN/m3	kN/m3	-	kN/m2	kN/m2
Soil layer 1	0-2	Undrained	M-C	21	23	0.33	22000	120
Soil layer 2	2-15	Undrained	M-C	21	23	0.33	100000	270
Soil layer 3	15-20	Undrained	M-C	21	23	0.33	140000	310
Soil layer 4	20-52	Undrained	M-C	21	23	0.33	100000	270
Concrete	n/a	Elastic	L-E	24	24	0.2	2.74E+07	-

The interface element is applied between piles and soil, the strength reduction factor R_{inter} is assumed to be 0,7. The definition of parameters can be referred to Section 2.3.2.

3.6.1.2. Results. In Figure 3.70 and Figure 3.71 total displacements of case 5 are given. As can be seen there, maximum deformation observed is 13.48 mm. The deformations observed indicates that the soil up to 9 m is effected by the movement. As can be seen in the following Figure 3.72, maximum vertical deformation observed is the same as total displacement value 13.48 mm which means there isn't any distinctive horizontal displacement.

In the following Figure 3.73 and Figure 3.74 displacement behaviors of numerically simulated and site measured case 5 under loading are given respectively. These two data are combined on the same template to show the similarities and differences between them which is given in Figure 3.75. The overall displacement values of the predicted displacement curve tends to be overestimated for measurement curve. Low load displacement assumptions are lower than the measured ones; however, with increasing loads especially after 700 tons higher displacement estimations were get than the measured values

In Appendix E, additional numerical simulation results are given.

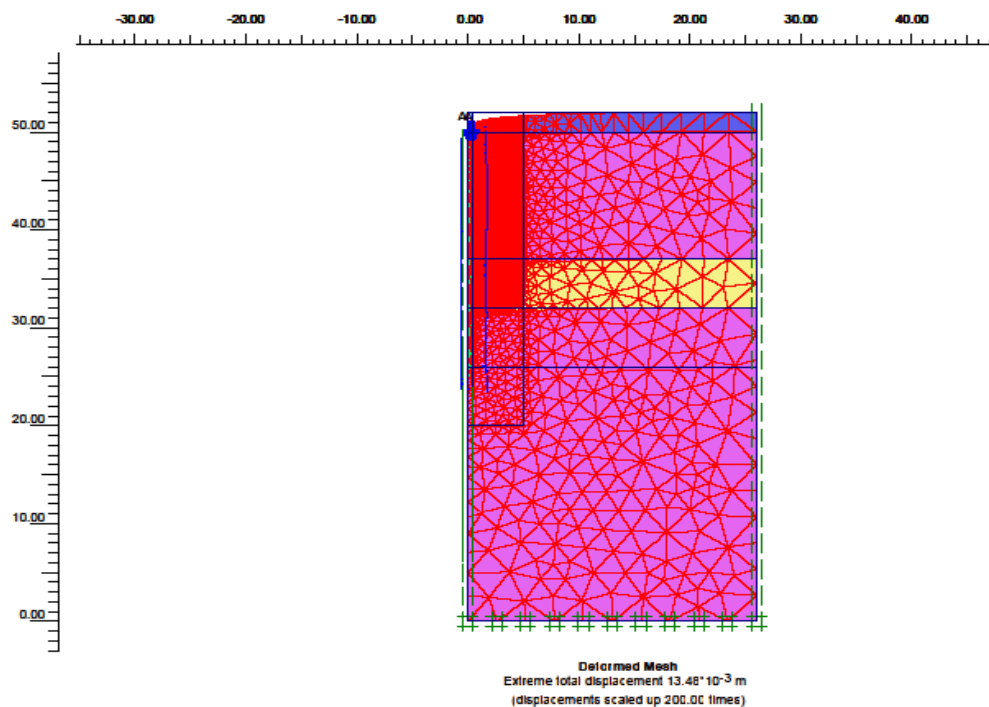


Figure 3.70. Deformed mesh of case5.

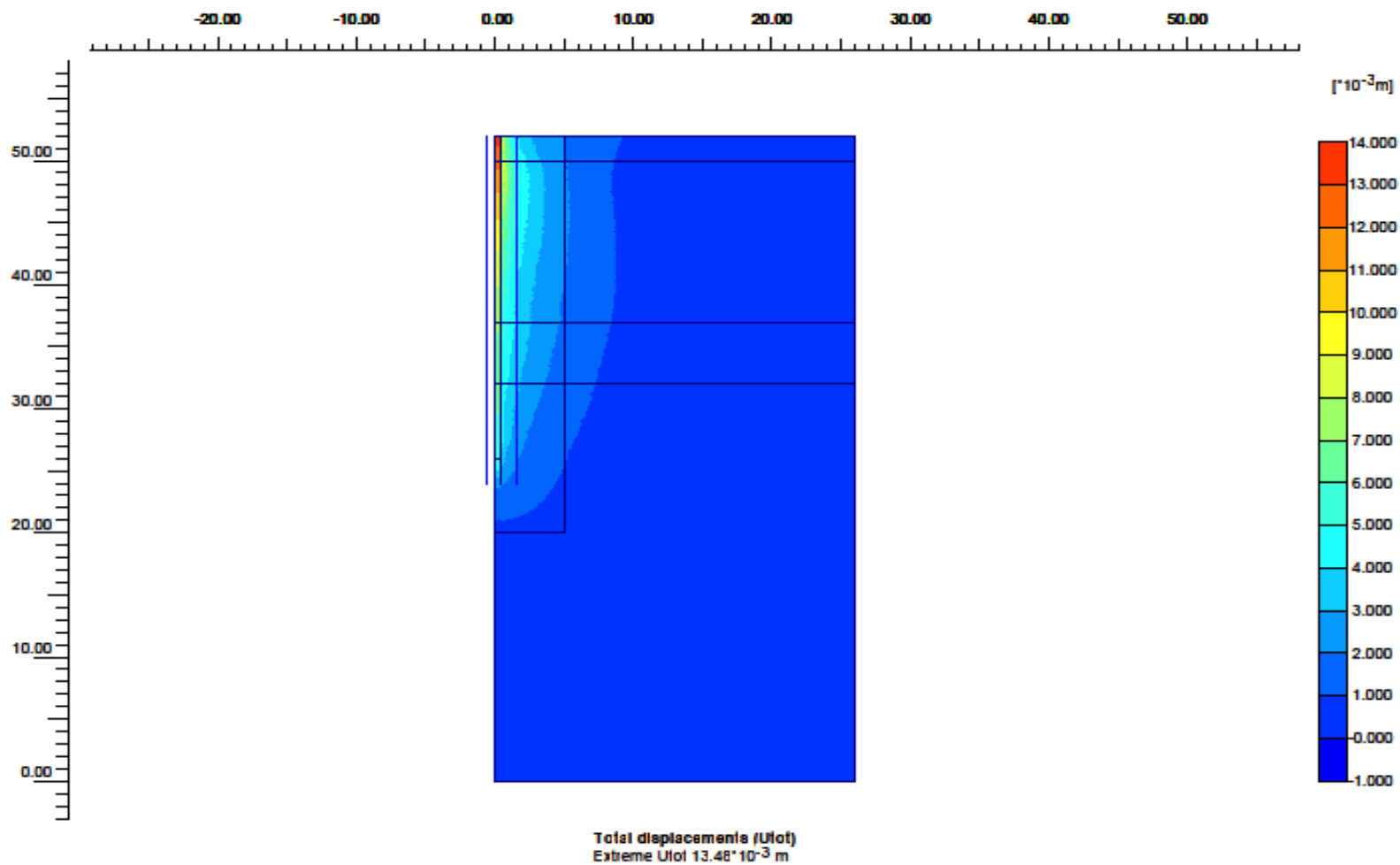


Figure 3.71. Total displacements of case5.

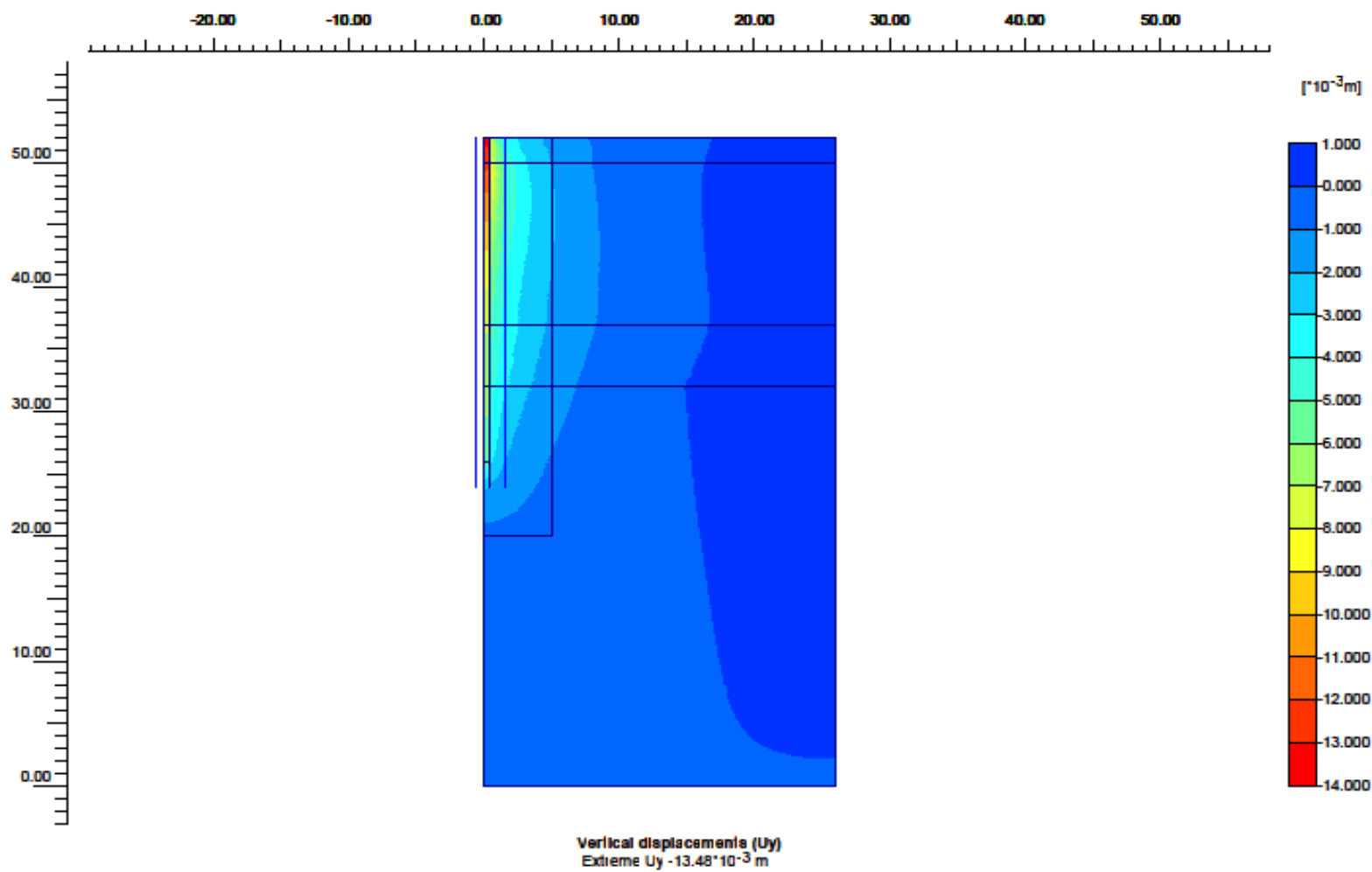


Figure 3.72. Vertical displacements of case5.

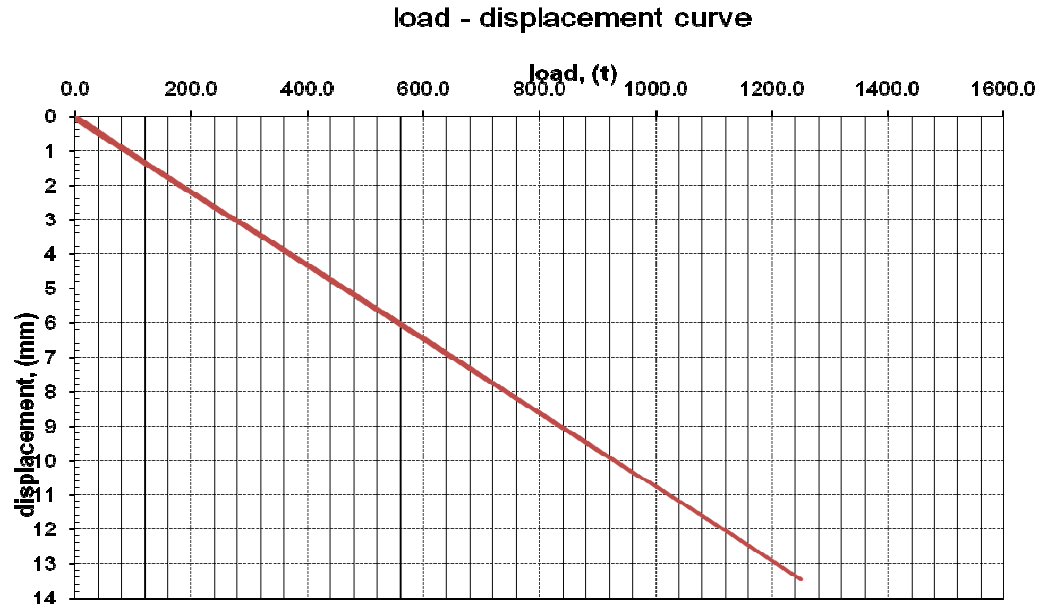


Figure 3.73. Load versus Displacement curve of case5.

3.6.1.3. Site Test Results. In Appendix E, additional site measured experimental data are given.

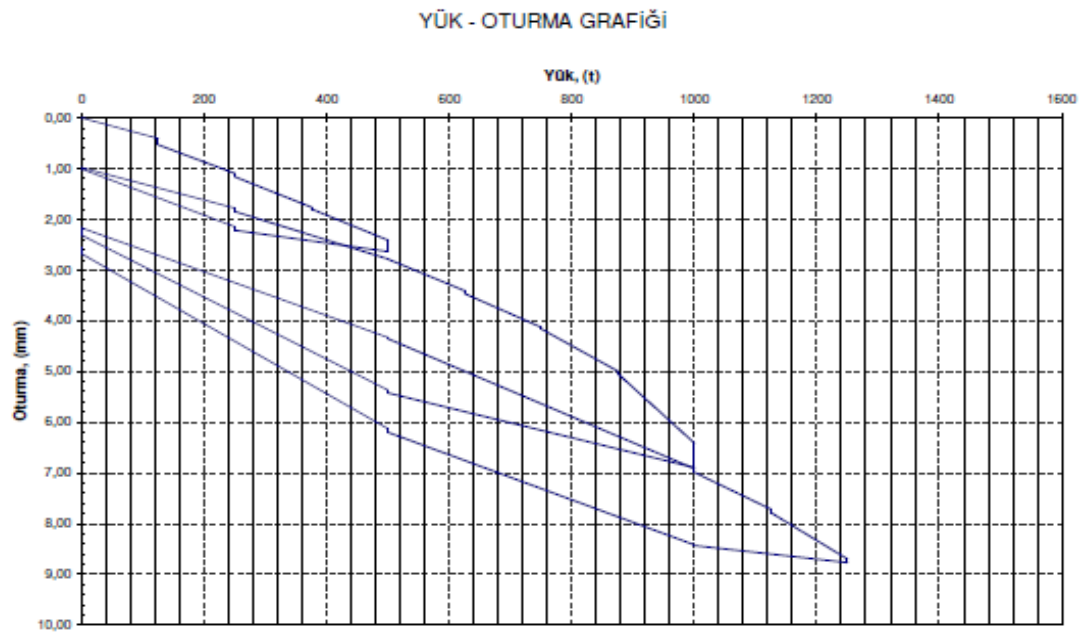


Figure 3.74. Load versus Displacement curve of case5.

3.6.1.4. Site Measured and Numerically Simulated Load versus Displacement Curve of Case5. Both curves are given in the following figure.



Figure 3.75. Site measured(blue) and numerically simulated(red) load versus displacement curve of case5.

3.6.1.5. Stress Distribution Along the Pile Shaft. Stress distribution is given in the following figure.

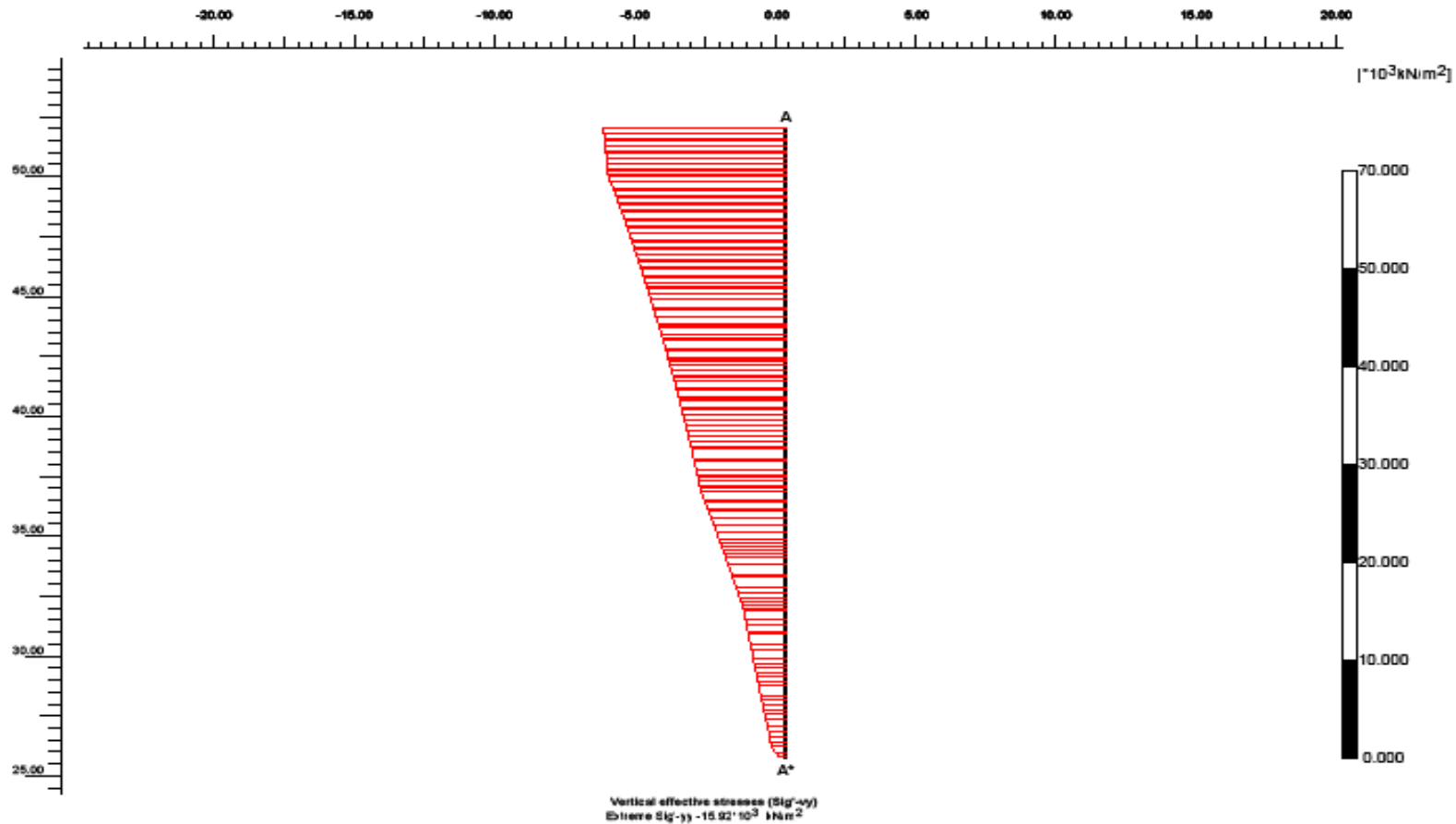


Figure 3.76. Vertical stress distribution along the pile shaft cross-section from the top to the bottom.

Obtained from Figure 3.76 in – 2 meters stress is $-15,428.2 \text{ kN/m}^2$ and in – 15 meters it is $-7,437.79 \text{ kN/m}^2$, in – 20 meters $-3,684.98 \text{ kN/m}^2$, in – 26 meters 584.66 kN/m^2 and -12967.4 kN/m^2 at the top.

Combining the data from Figure 3.76 gives the curve given in Figure 3.77 in which decreasing stress distribution with depth can be seen.

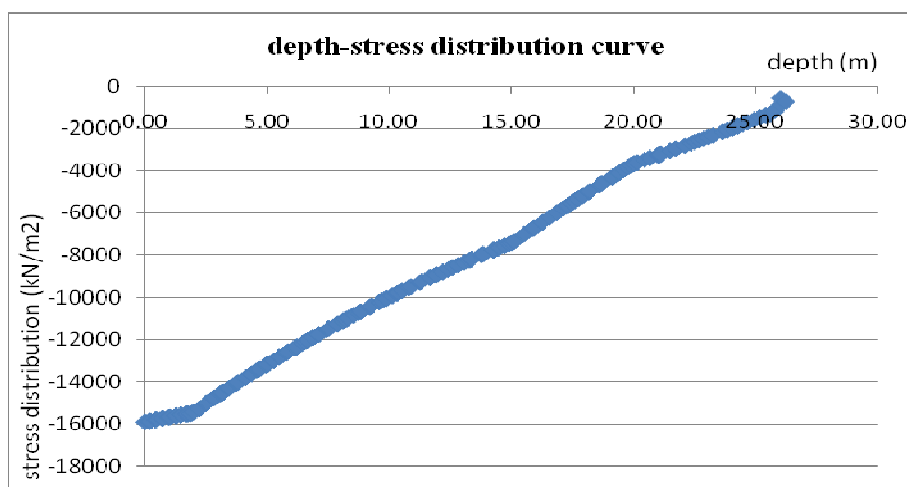


Figure 3.77. Depth(m) versus Stress(kN/m^2) distribution curve of pile shaft cross-section from the top to the bottom.

Stress change within a specific depth can be obtained from the Figure 3.77 and used to calculate cohesion within this specific depth by dividing the stress change to frictional resistance area. The result of the calculations is given in Table 3.14, in which the difference between calculated and assumed cohesion values can be seen. This difference stems from that I assumed an average cohesion value for a specific depth with the help of literature data while effective stress was changing within depth.

Table 3.14. Calculated and assumed cohesion values within depth.

depth	stress change	$c_{\text{calculated}}$	c_{assumed}
m	kN/m^2	kN/m^2	kN/m^2
0-2	15,916.7-15,428.2	61	120
2-15	15,428.2-7,437.79	154	270
15-20	7,437.79-3,684.98	188	310
20-26	3,684.98-584.66	129	270

4. CONCLUSION

Four different static pile load tests were analyzed with 2-dimensional FEM study. In addition, the difference between the field test data and the numerical models of them were performed, as well as one 2-dimensional FEM study on conventional static load test and Osterberg-cell test and comparison of these two kinds of pile load tests were performed under simplified soil profile and boundary conditions. The same research methods were applied to these five case histories to affirm findings. The findings in this thesis are:

4.1. Comparison of Osterberg-Cell Load Test with Conventional Load Test

It is concluded from the preceding FEM study, that O-cell test results can provide not only the same soil-pile interaction information as conventional head-down static loading test, but also allow for separate determination of the shaft resistance and end bearing components.

The FEM computation indicates that the shaft resistance-movement curve for upward movement of the pile is fairly comparable with the downward shaft resistance-movement component of a conventional head-down test. The result of undrained analysis suggests that a little difference of mobilized shaft resistance may exist between O-cell test and conventional test in the short-term. The reason is that the shaft resistance is lower in the upper strata of soil. By O-Cell tests, because loading is from O-Cell to the top, which means from higher shaft resistance to lower one, small effective stresses occur; unlike conventional tests.

At the smaller deflection, the end bearing of O-cell test increases faster than the conventional static test. One possible reason is that for the head-down test, the load transfer from the top to the toe is unable to effectively activate the end bearing and relies heavily on the side shear resistance; the O-cell however is able to immediately activate the end bearing and therefore gives a faster response. As the test progresses, the conventional test is able to activate the end bearing when the shaft resistance becomes fully mobilized, Therefore the end bearing becomes closer to that of O-cell test thereafter.

Conventional tests show less stiff elastic response and lower ultimate capacity than O-Cell tests. The end-bearing is always mobilized more lately at the same displacement for the conventional test than the O-cell tests. The reason for these is that the source in O-Cell test is near to the toe, so frictional resistance can be overcome to get toe resistance by small loading conditions when compared to conventional tests. By conventional tests, load is applied from the top and there is a pile depth long frictional resistance to overcome, so one can see the less stiff response.

4.2. The Difference Between the Field Test Data and The Numerical Models

The FEM computation indicates that the shape of the mobilized displacement curve appear in agreement with the site measured experimental one. The overall loading stages of the predicted displacement curve tends to be properly estimated for measurement curve.

By numerical simulation one can see in the first loading stage the elastic behavior and consecutively stages the plastic behavior, which is because of the Mohr-Coulomb model assumption. However, in reality one can realize on site measured experimental data the plastic behavior in the low loading stages, which means that the system behaves nonlinearly on site. In reality, there isn't "0" displacement in the unloading stages; as a result, there is some amount plastic displacement. By numerical simulation this displacement couldn't be seen in low load stages, while it is an idealized model.

Both the pile head load (Q_t) versus the pile head displacement are compared with the results from numerical simulation using FEM. Base reaction develops starting from the value of the end-bearing load (Q_p) imposed at the end of installation. However, there appears to be a discrepancy between the measured base resistance at failure and the predicted one corresponding to a pile head displacement. It's quite likely that the discrepancy between predicted and measured base resistance at failure is the consequence of neglecting a basic phenomenon associated to the installation the pile: the densification of the soil around the pile base. Indeed, this induces an increase in the soil stiffness and soil improvement which is not taken into account by the numerical simulations of the pile test.

Predicted displacement values are different from the site measured experimental ones. This stems from the assumptions made while the soil parameters were being predicted during the modeling. Soil boring logs have some soil data which are around the pile area so they could not reflect the exact information of the strata where the pile will be built. In addition, some assumption is made for some soil level interval which could actually have lots of soil parameters; however, only one parameter assumption is made for this soil level interval according to the common features of its strata.

APPENDIX A

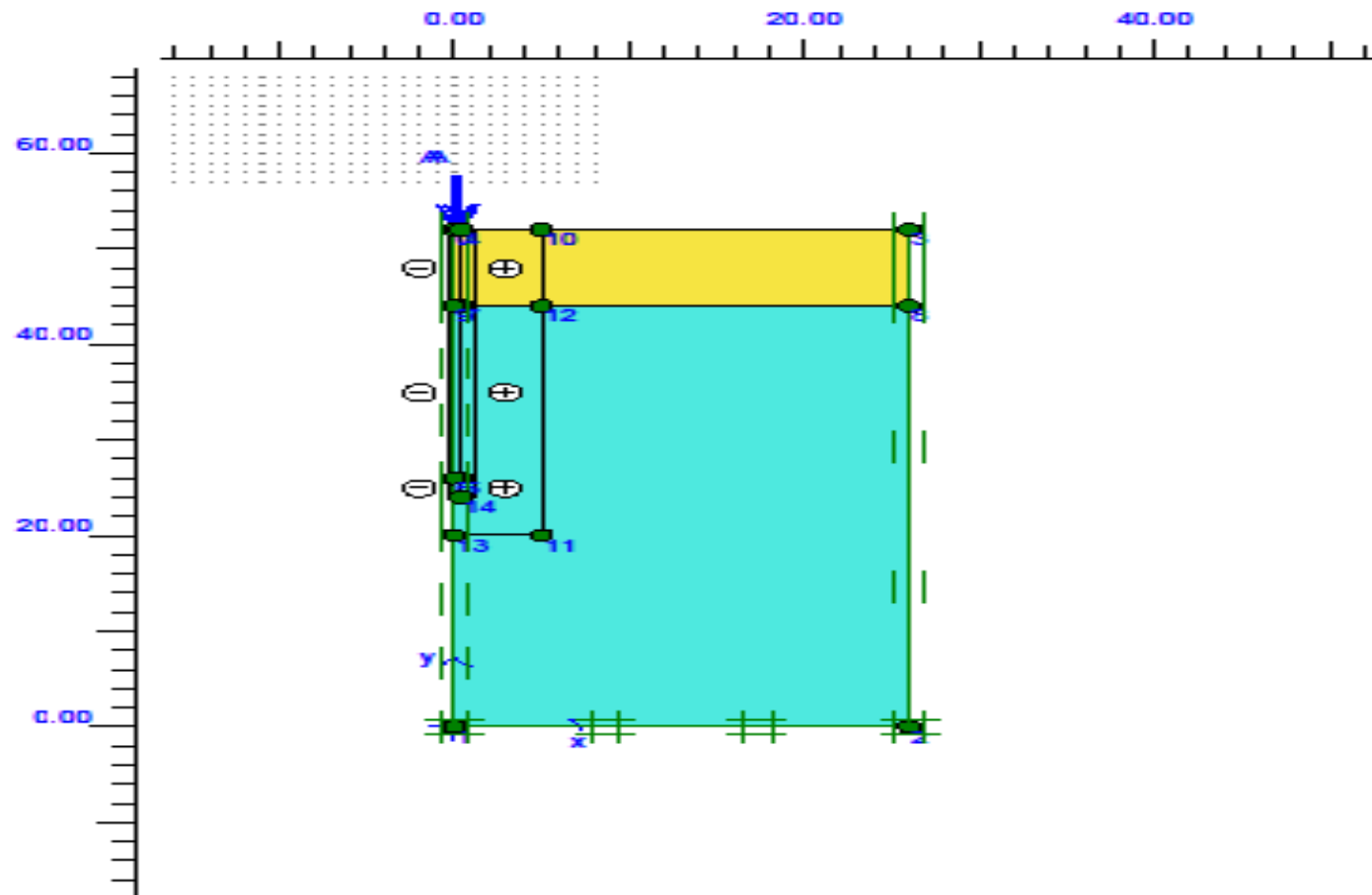


Figure A.1. Finite element model.

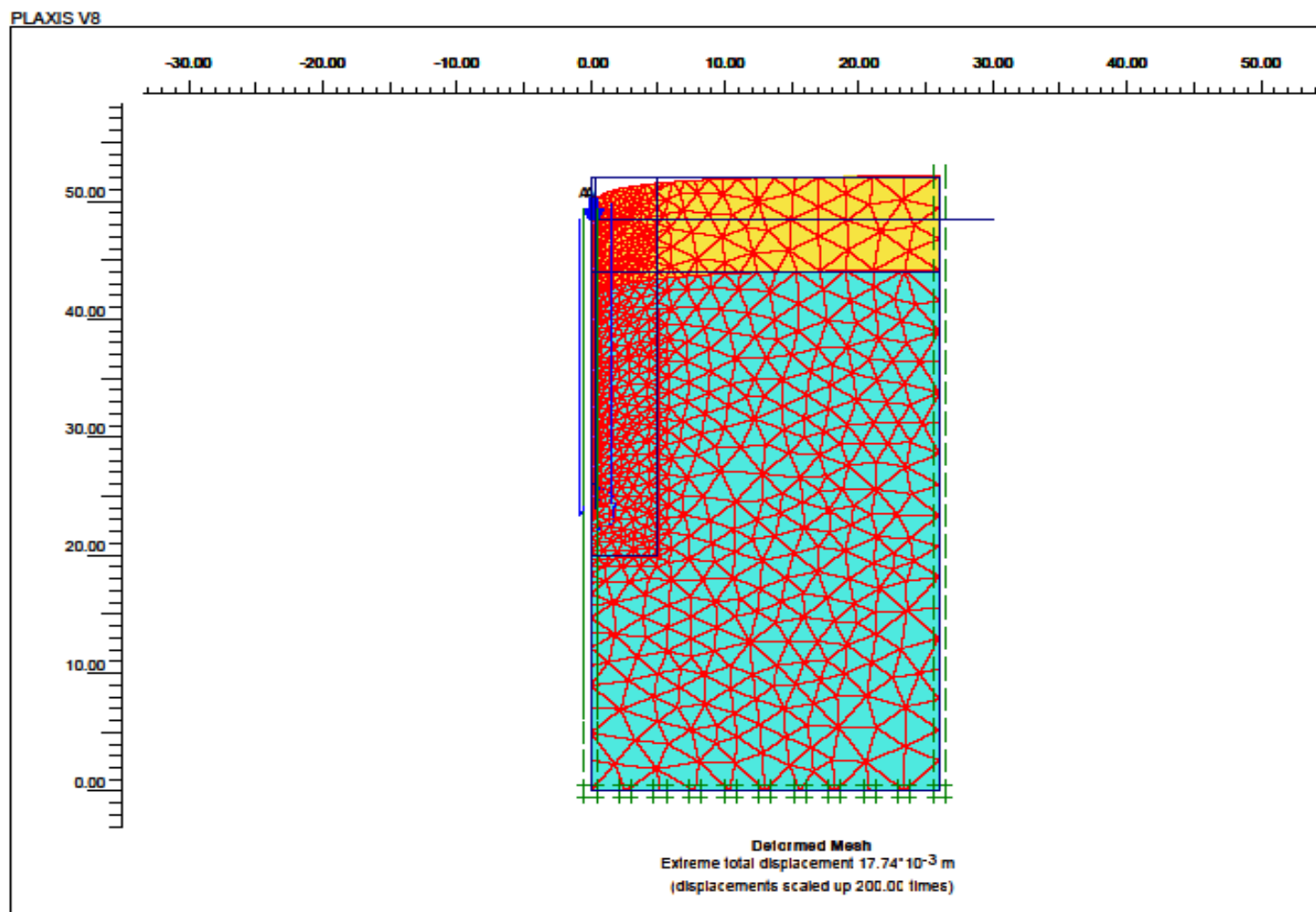


Figure A.2. Deformed mesh.

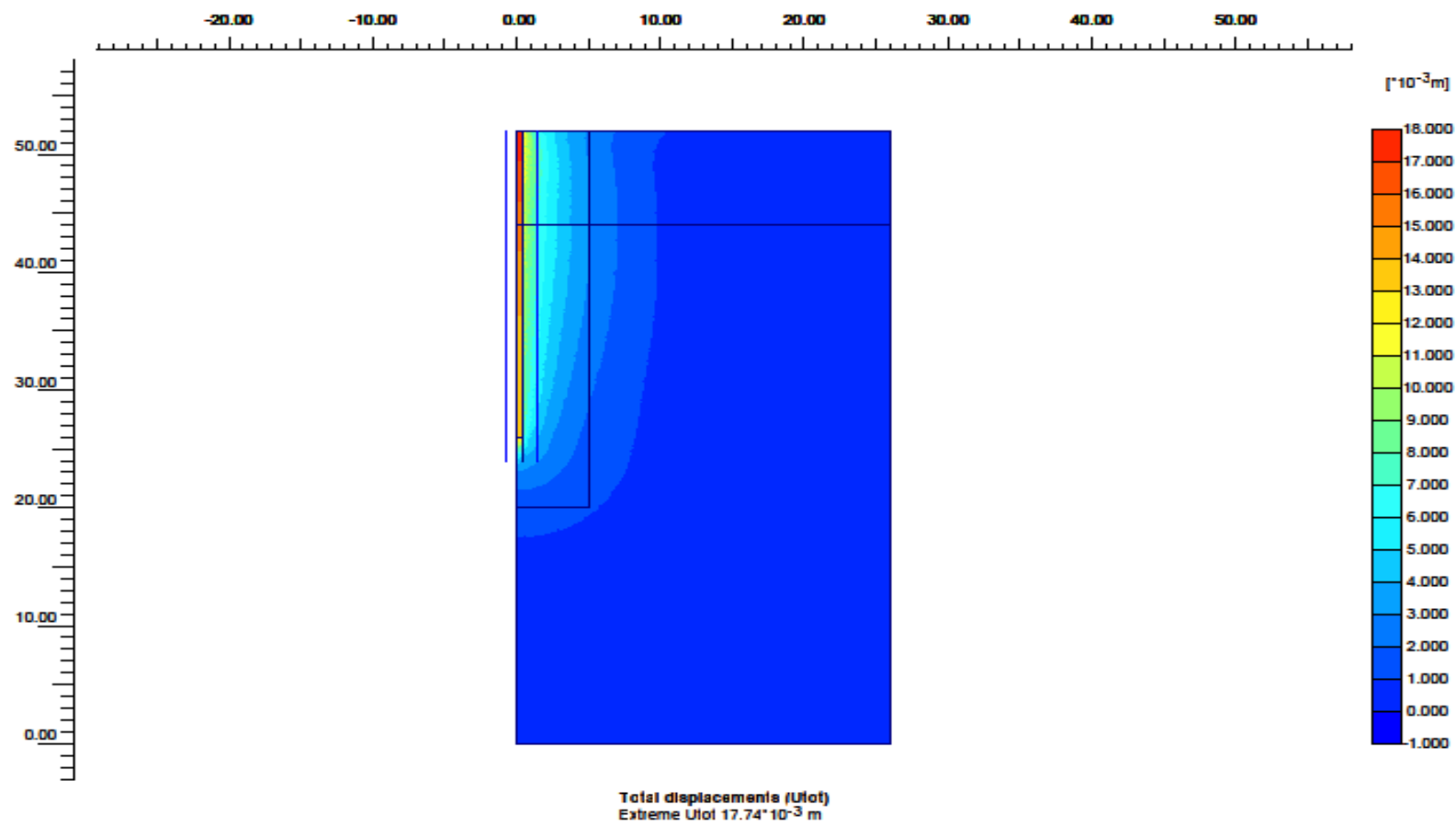


Figure A.3. Total displacements.

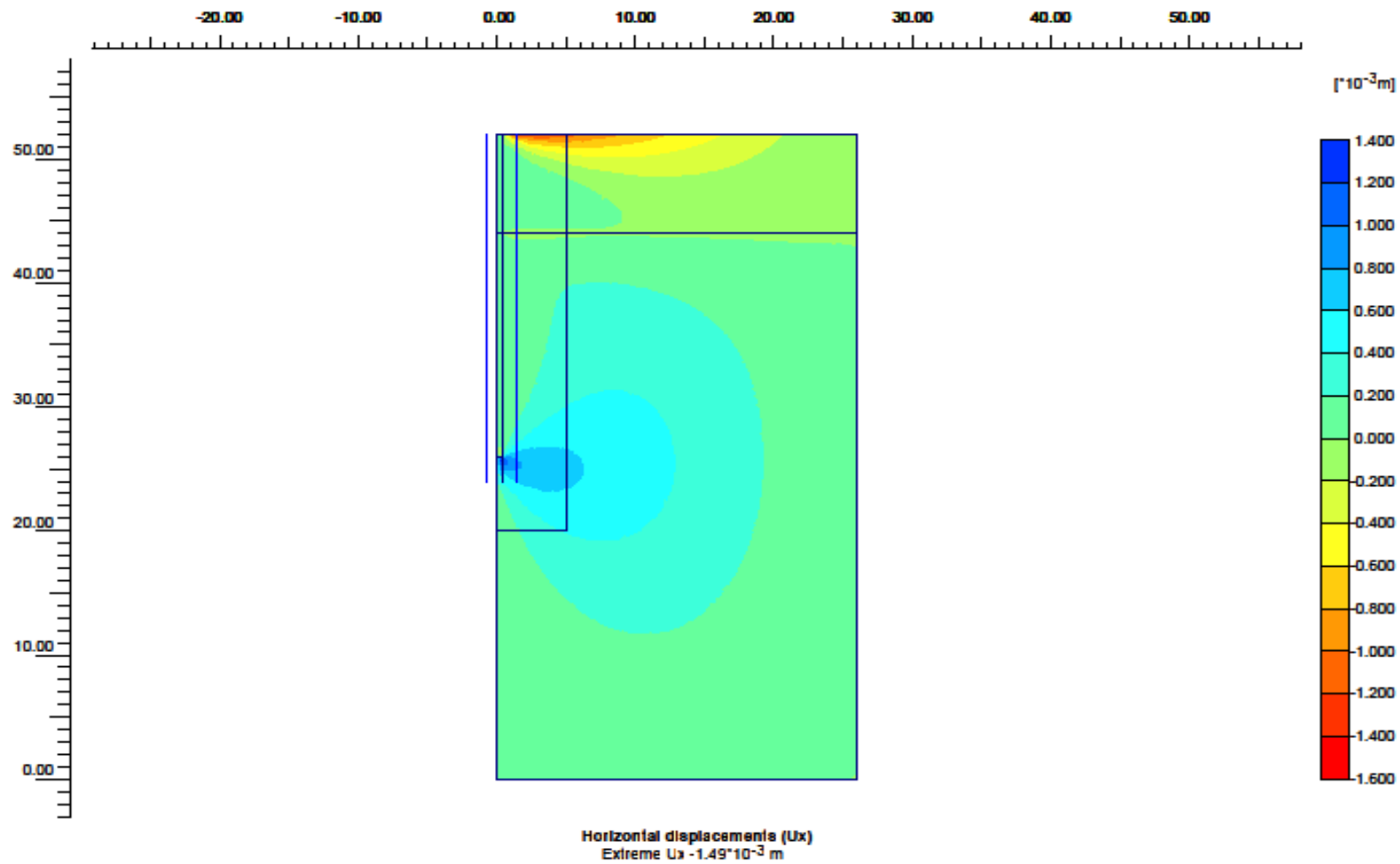


Figure A.4. Horizontal displacements.

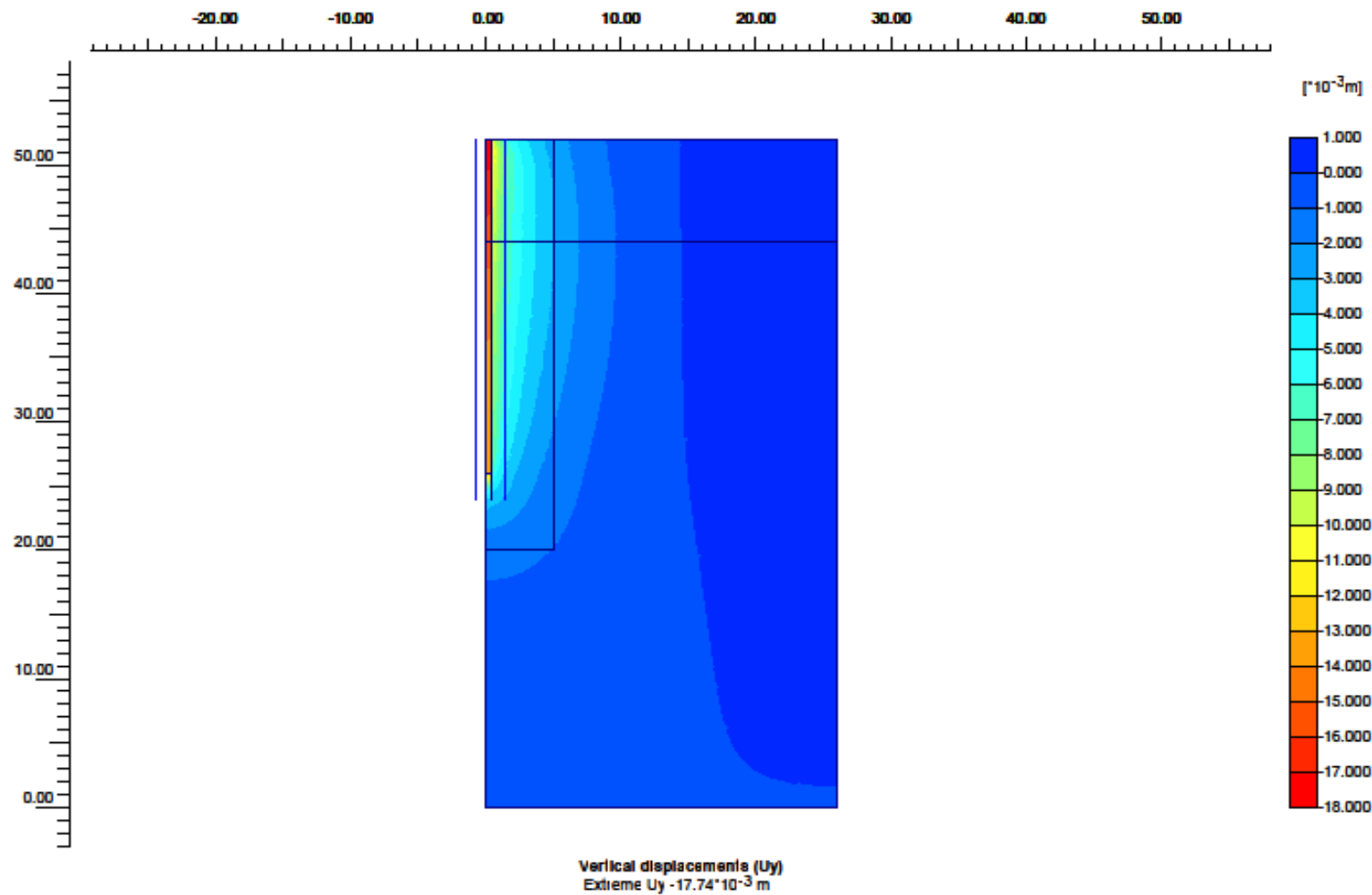


Figure A.5. Vertical Displacements.

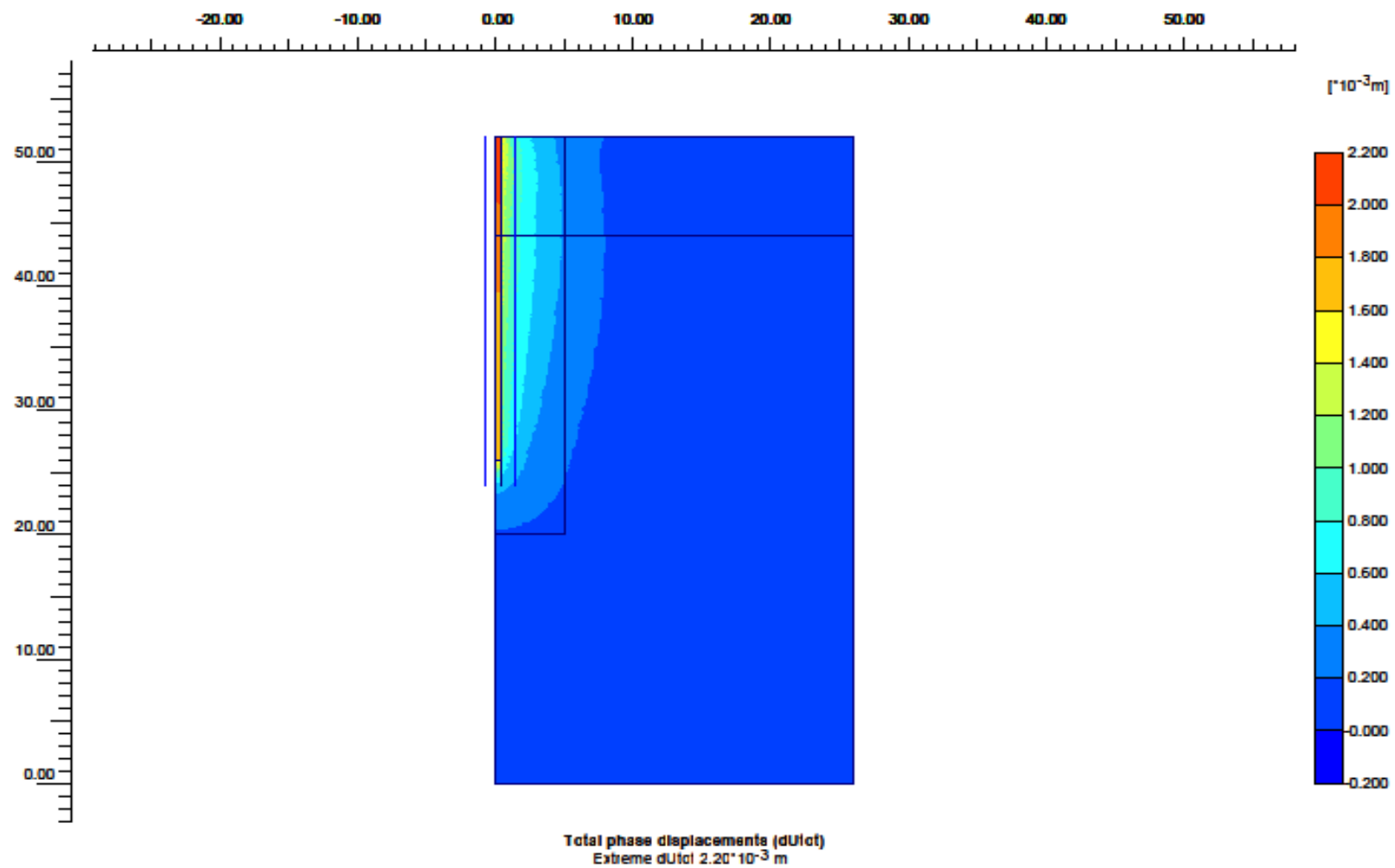


Figure A.6. Total phase displacements.

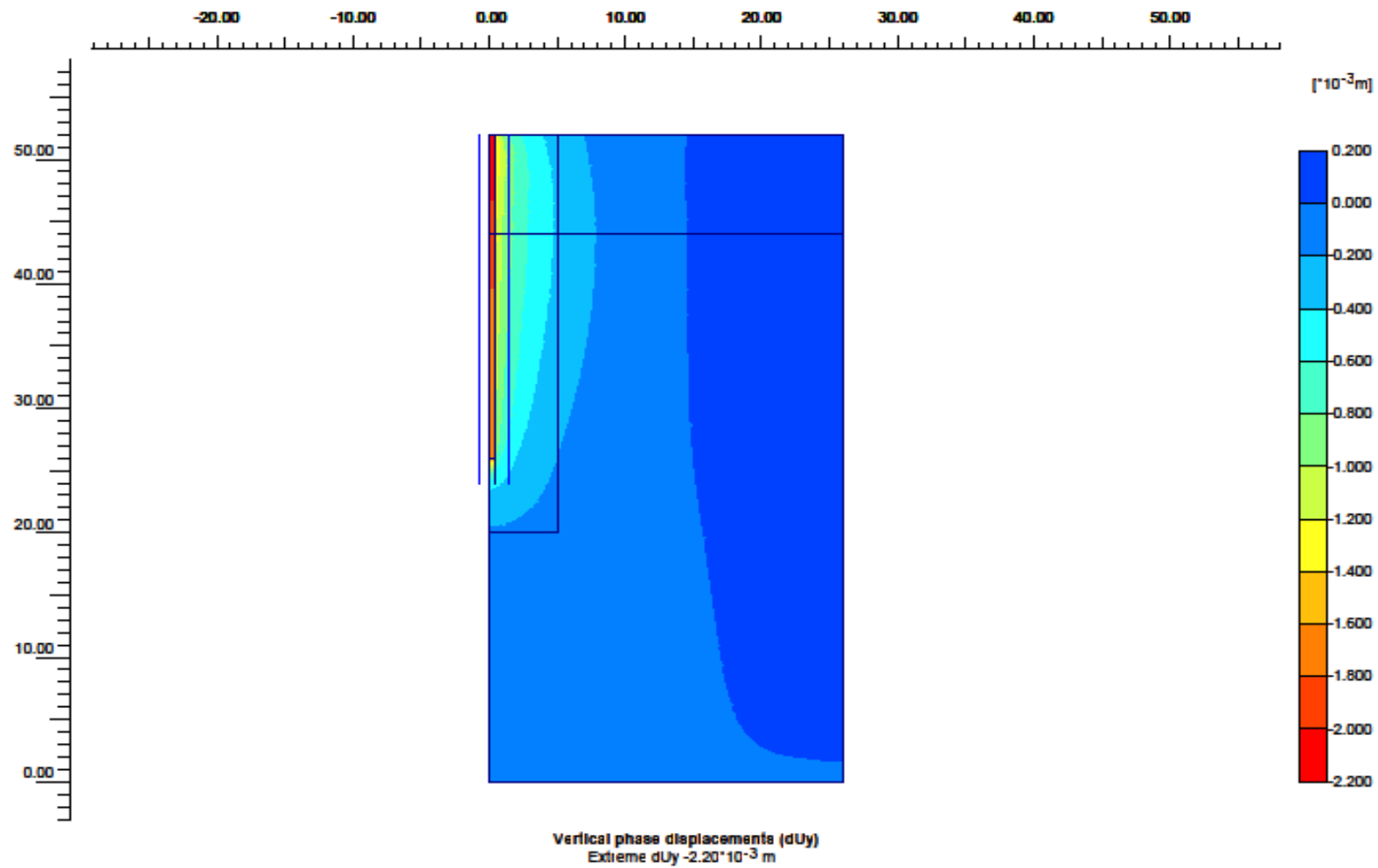


Figure A.7. Vertical phase displacements.

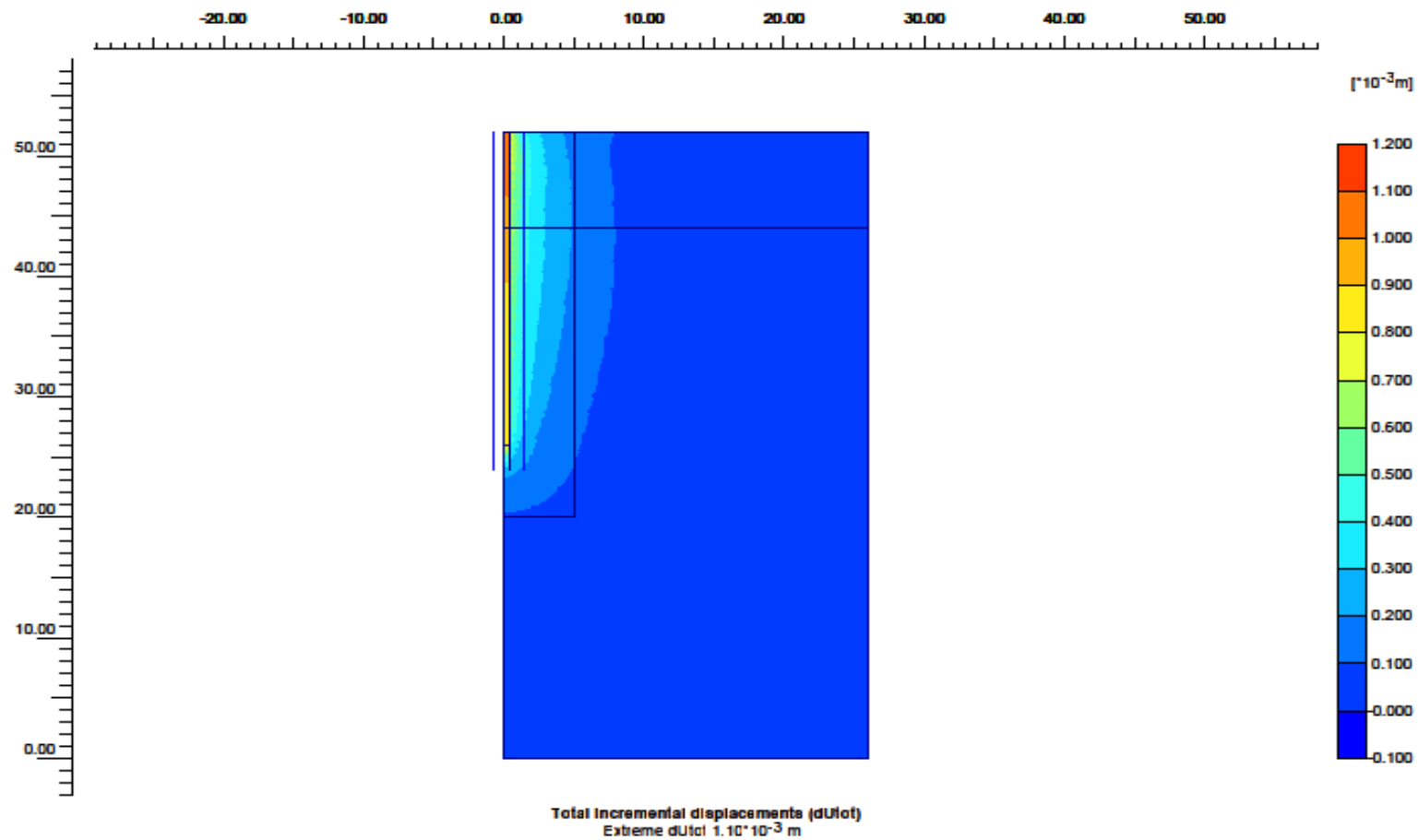


Figure A.8. Total incremental displacements.

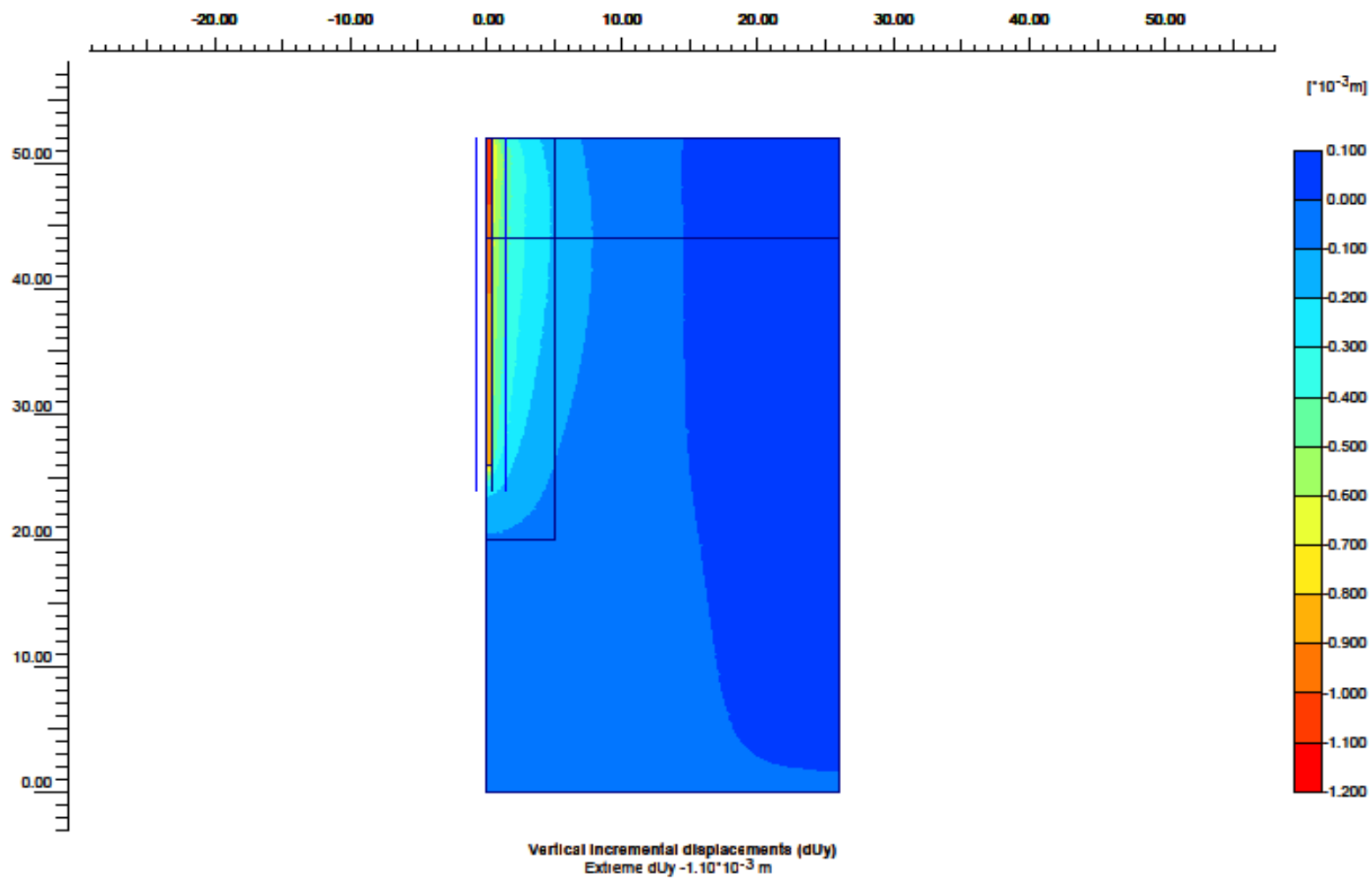


Figure A.9. Vertical incremental displacements.

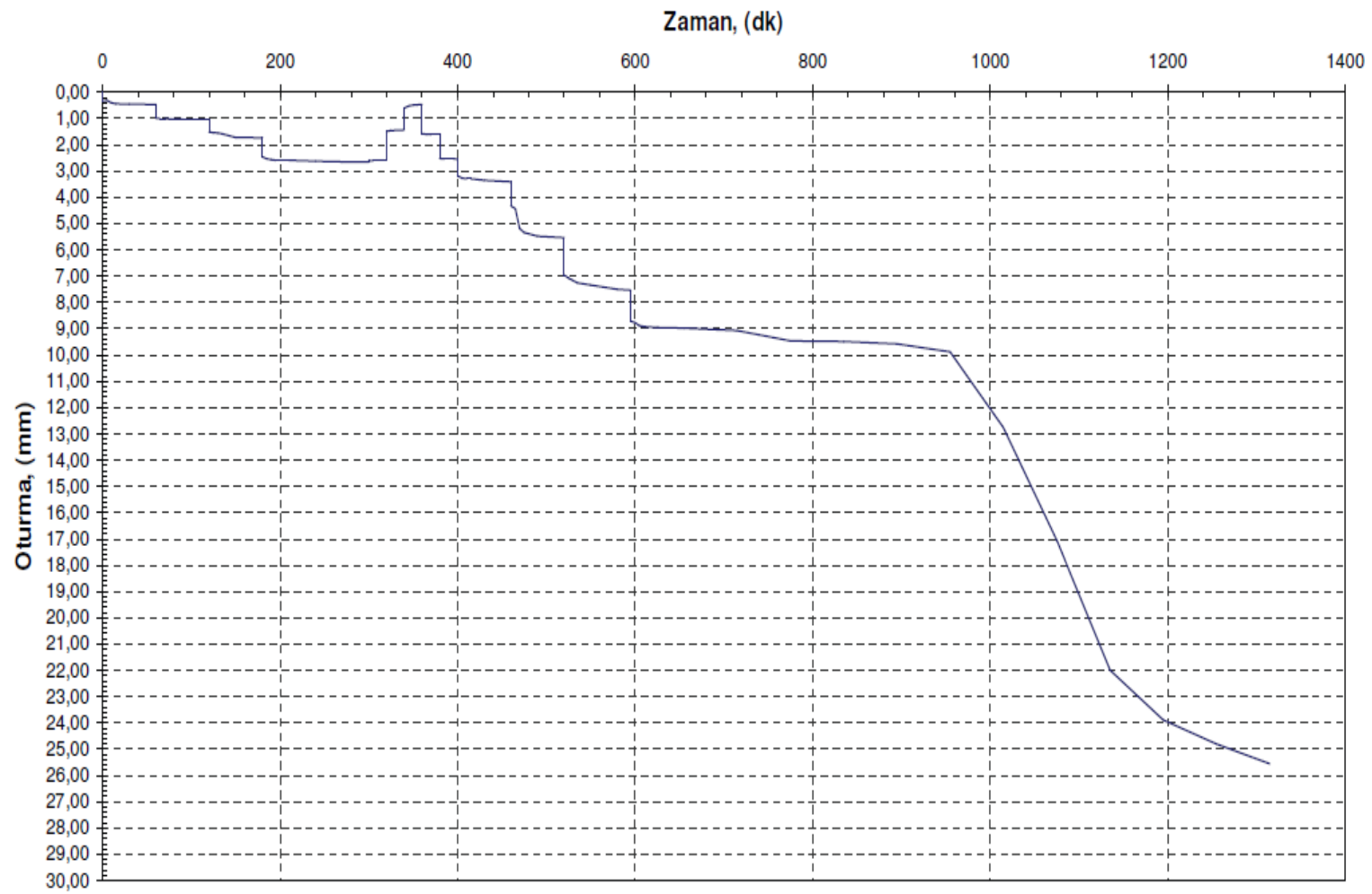


Figure A.10. Time versus Displacement.

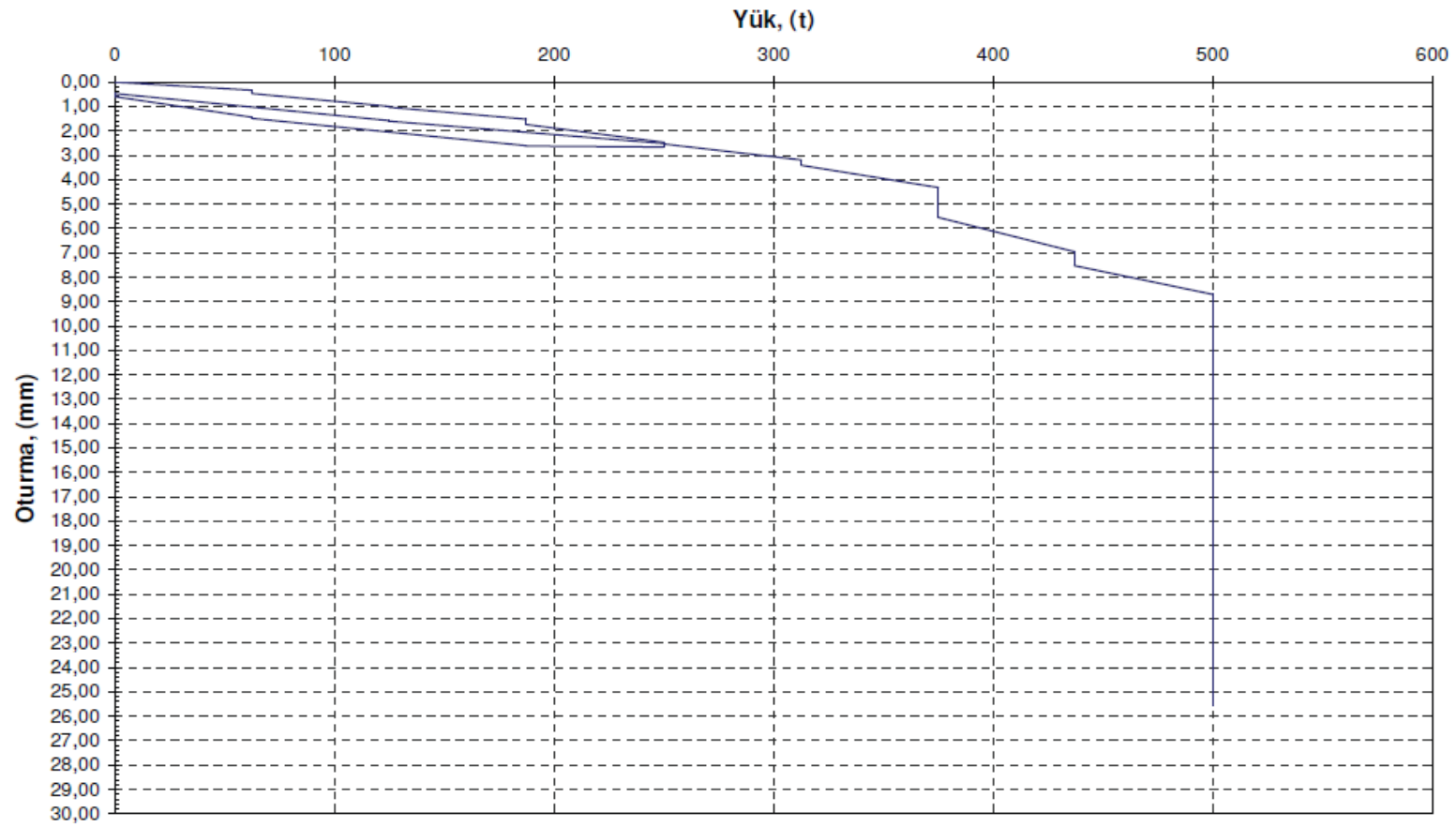


Figure A.11. Load versus Displacement.

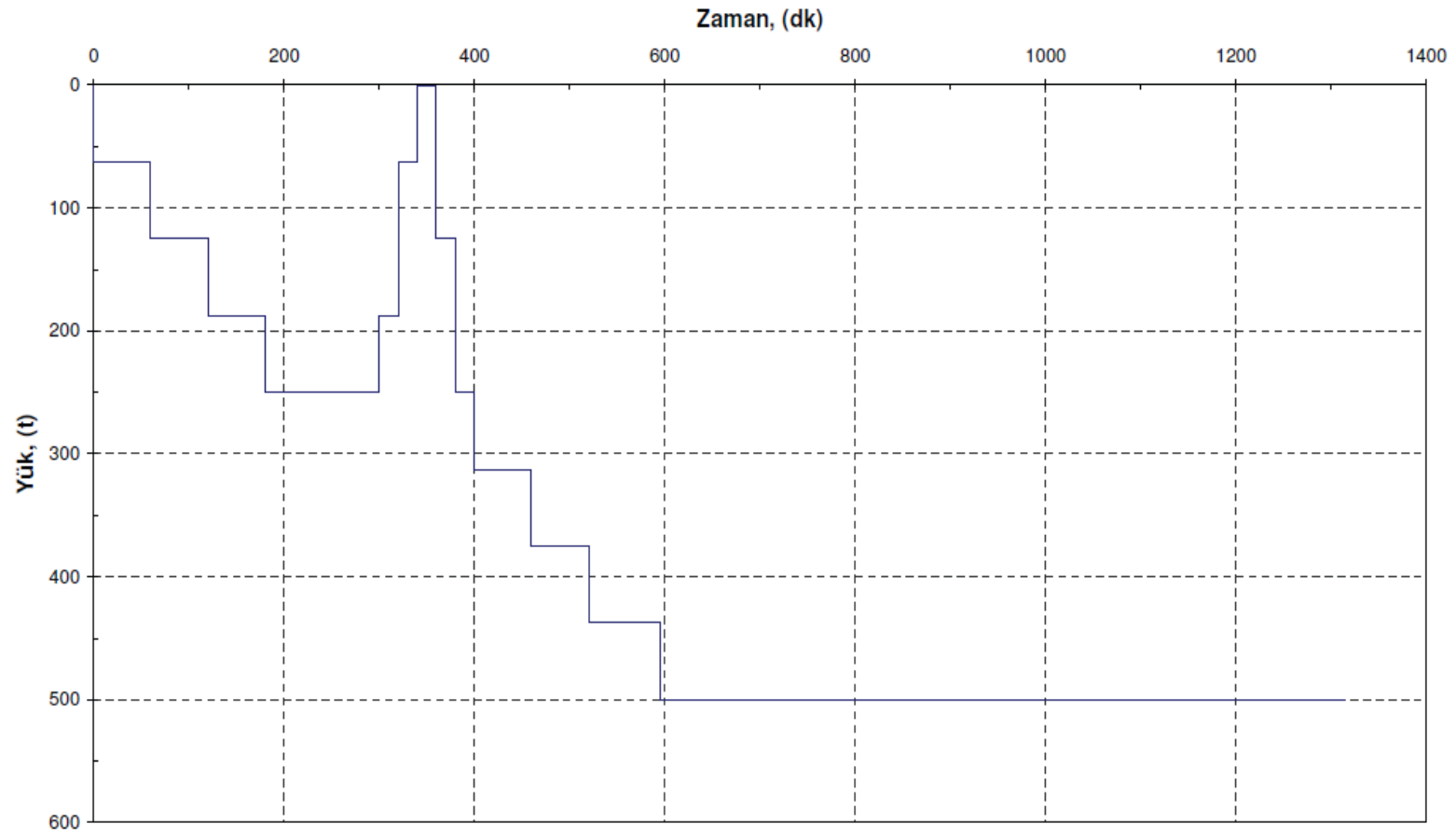


Figure A.12. Time versus Load.

APPENDIX B

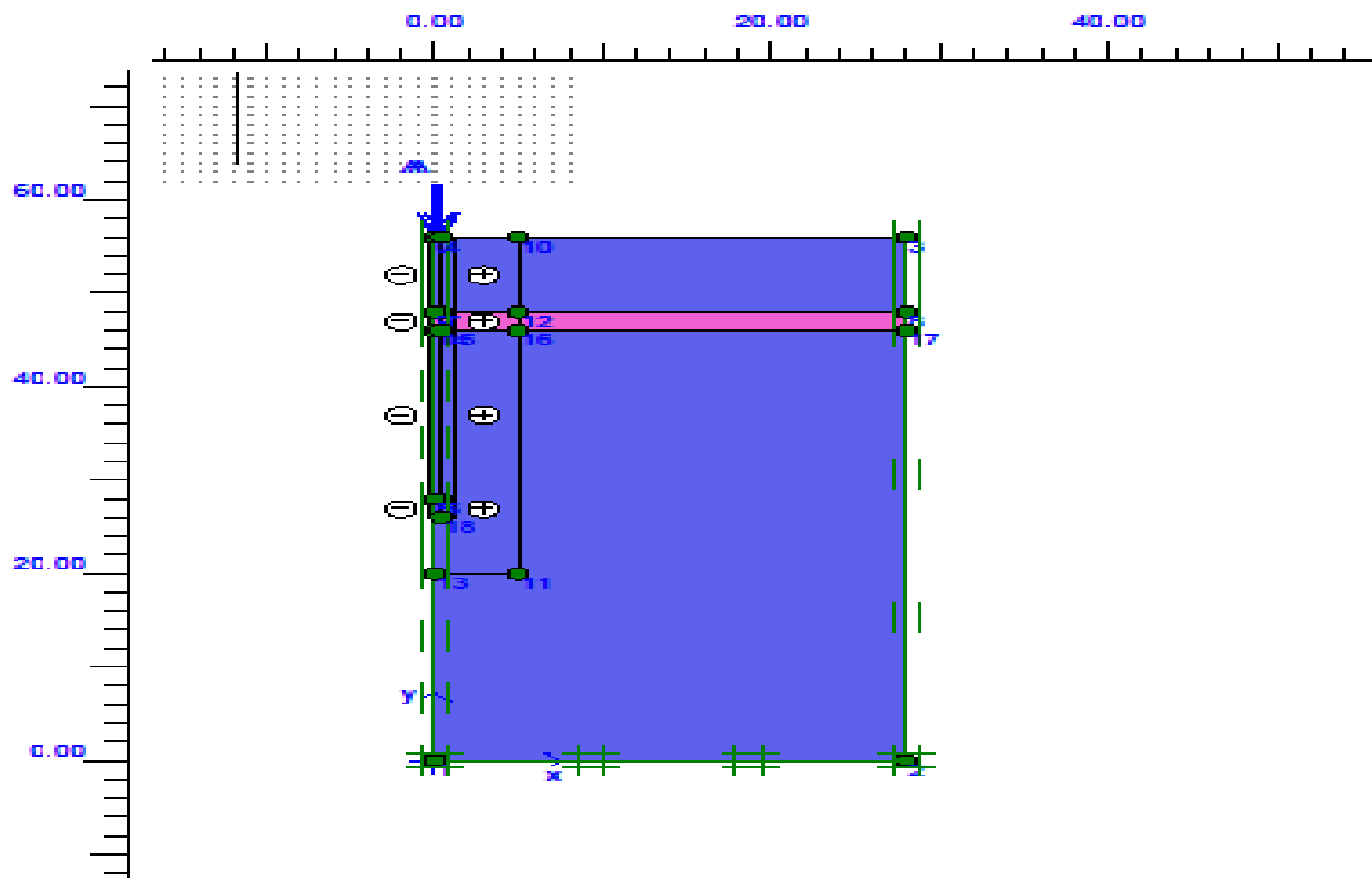


Figure B.1. Finite element model.

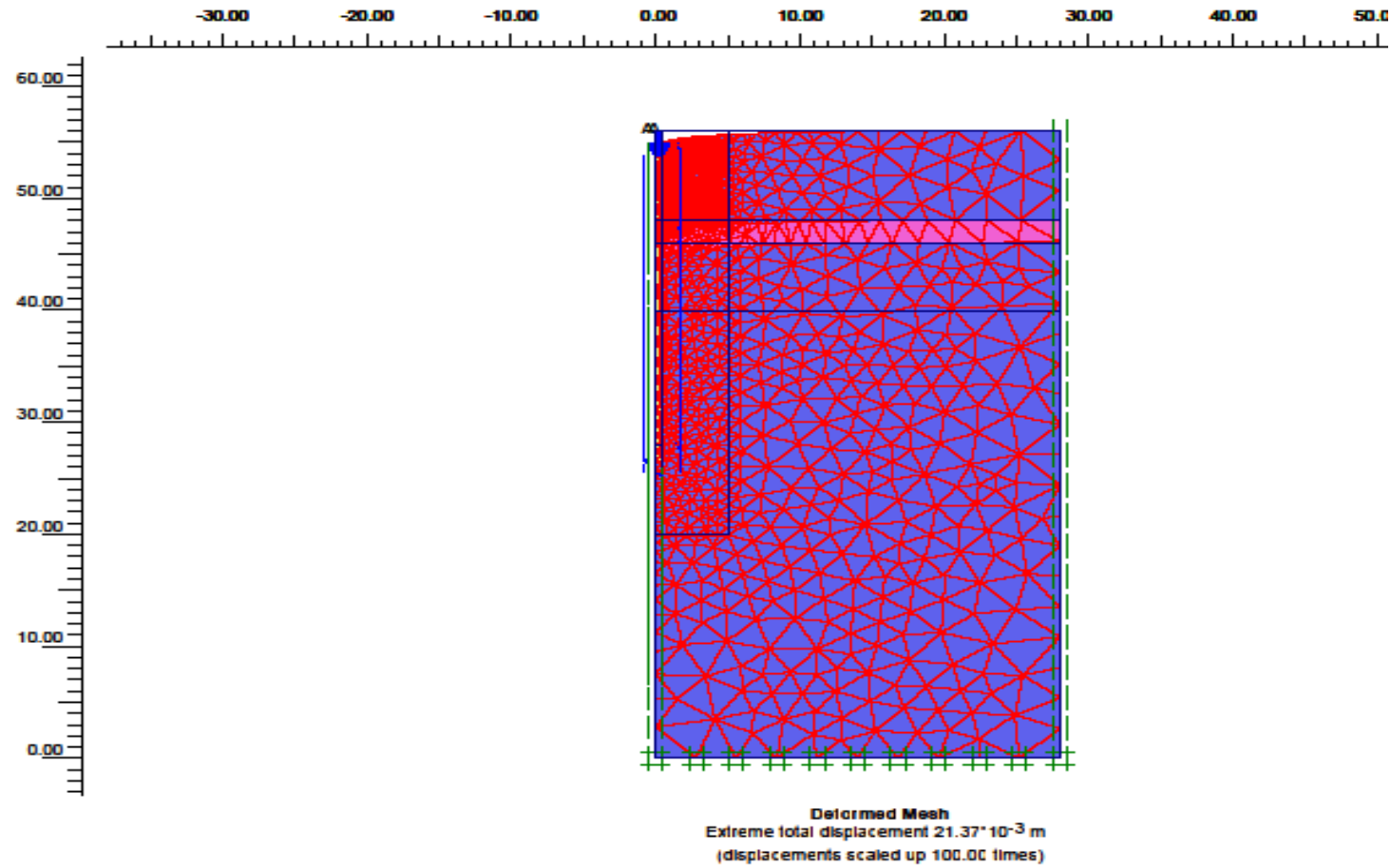


Figure B.2. Deformed mesh.

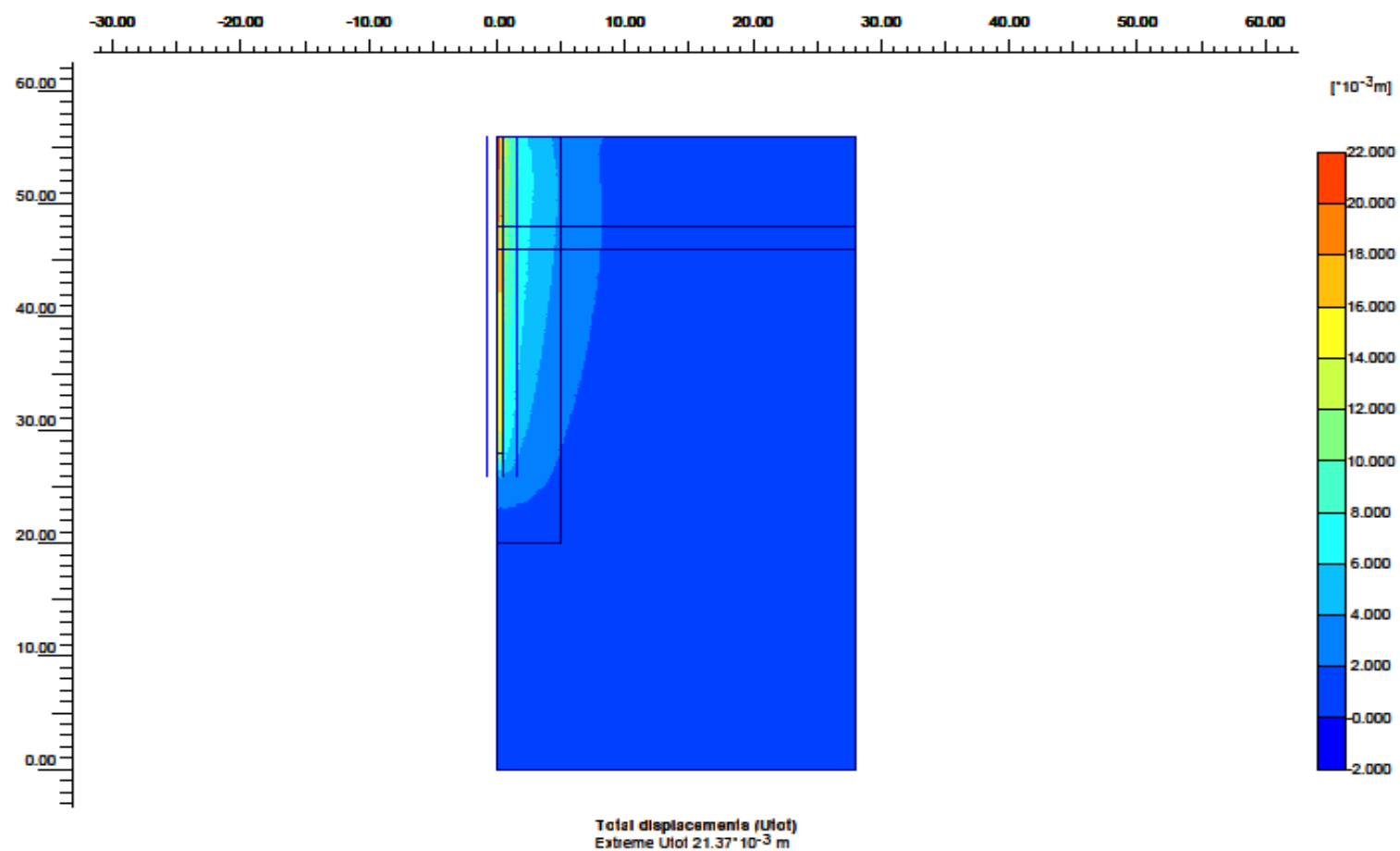


Figure B.3. Total displacements.

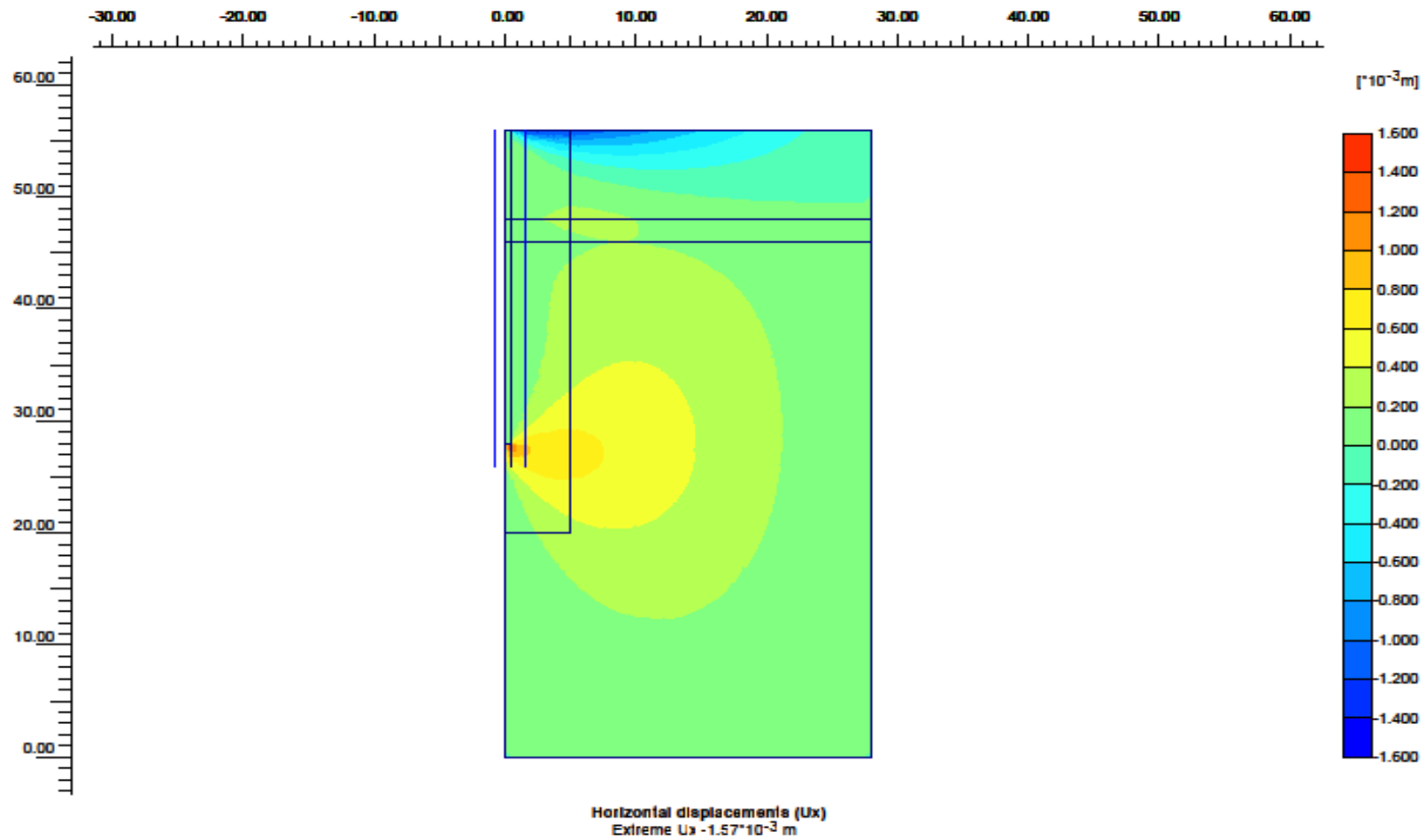


Figure B.4. Horizontal displacements.

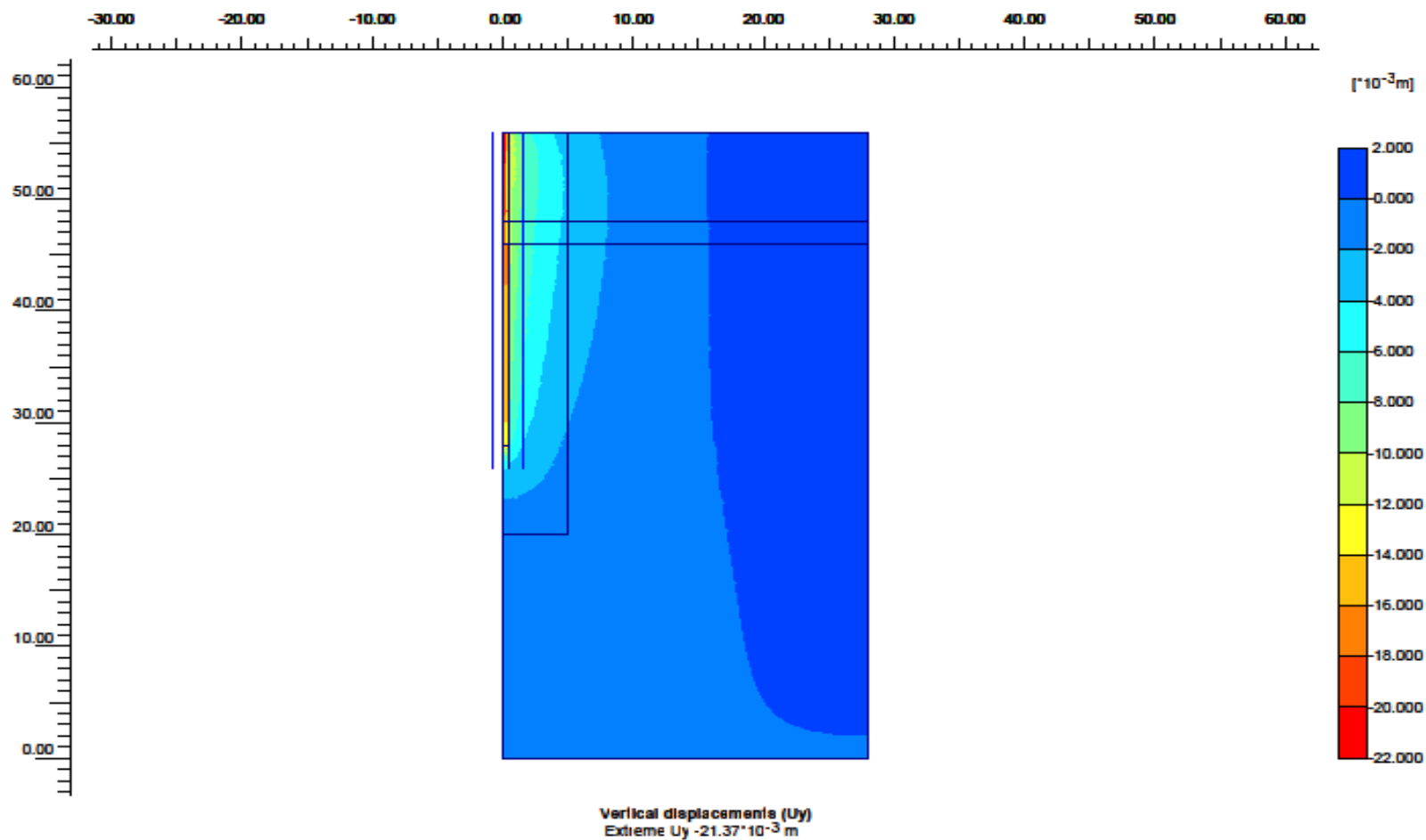


Figure B.5. Vertical displacements.

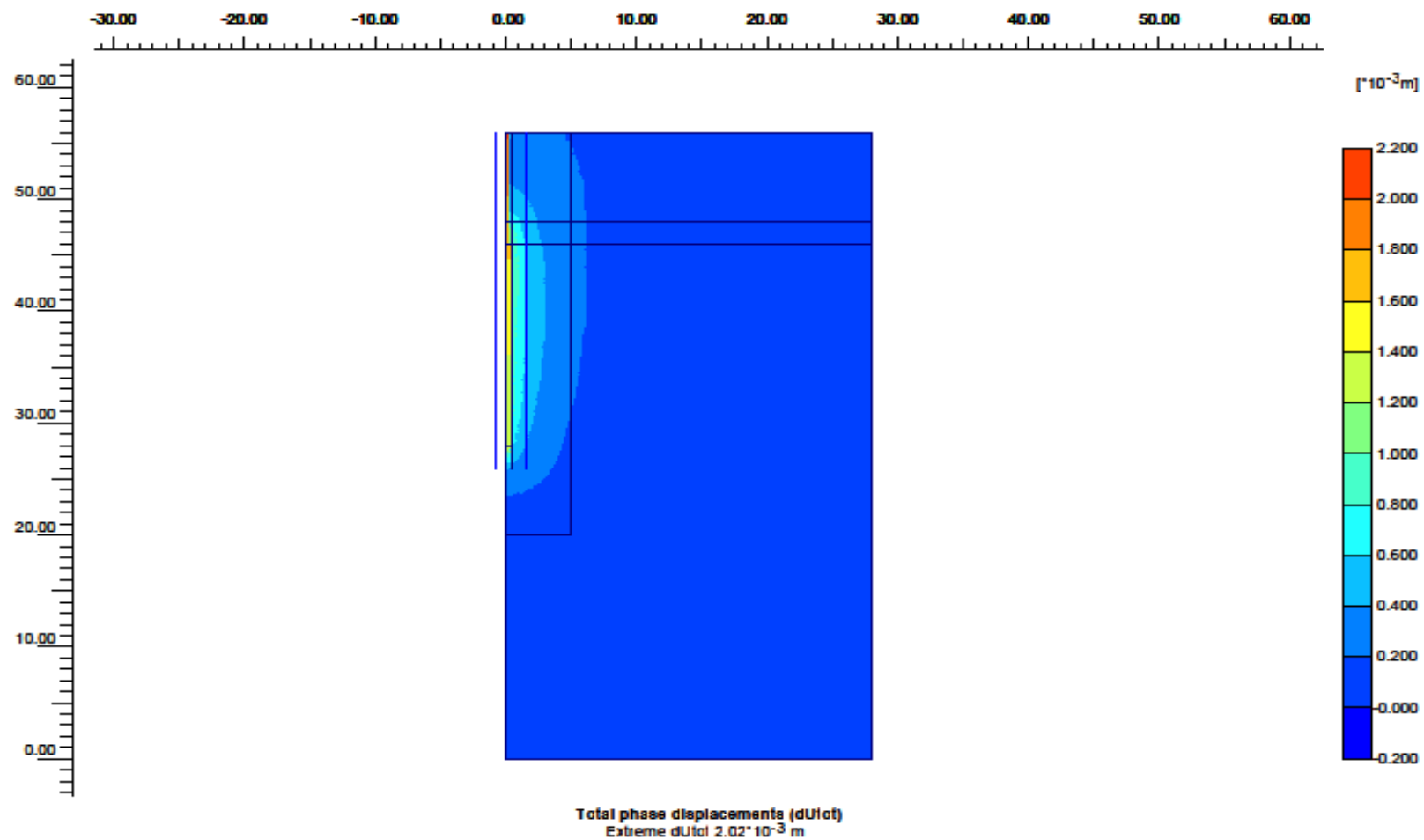


Figure B.6. Total phase displacements.

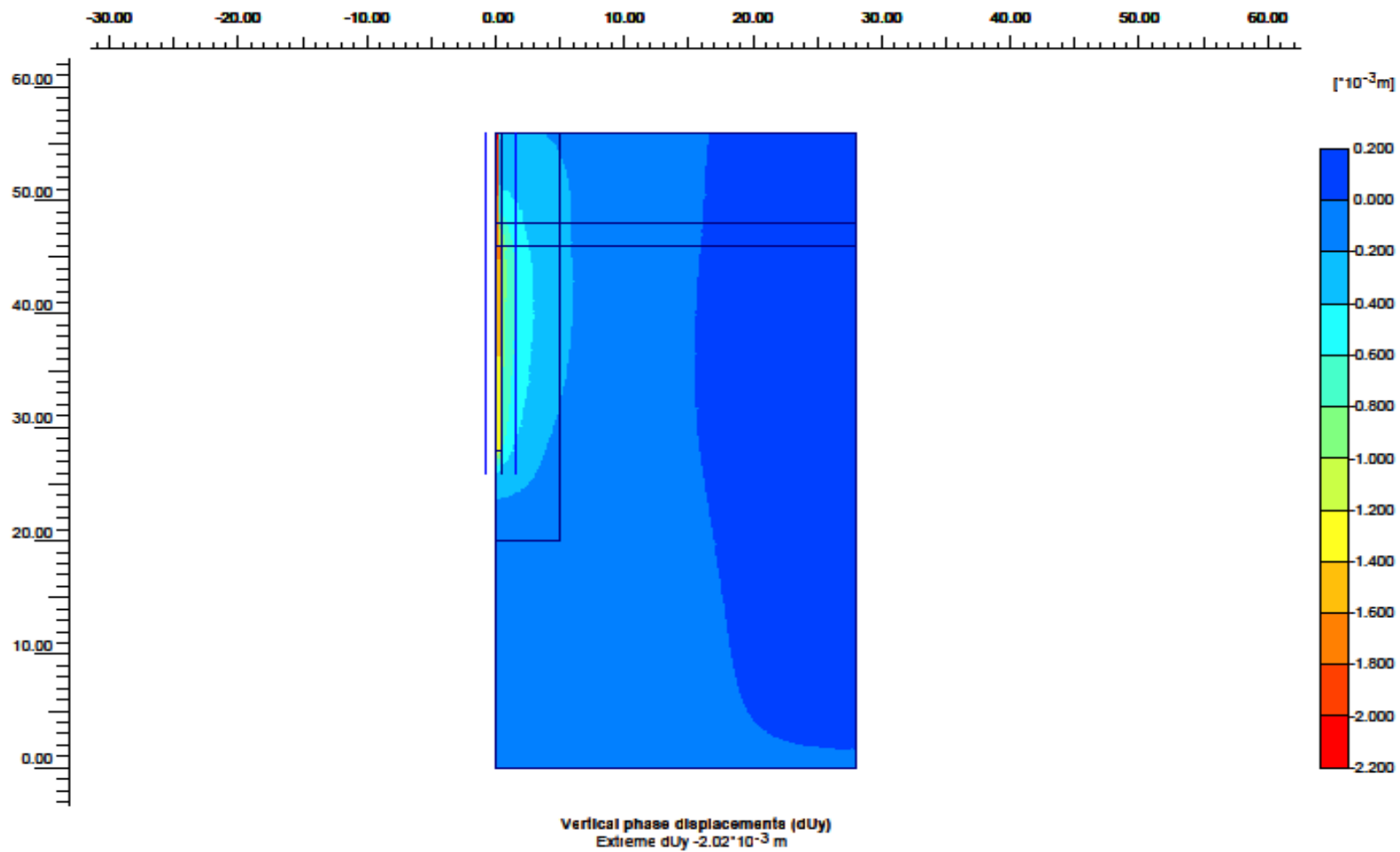


Figure B.7. Vertical phase displacements.

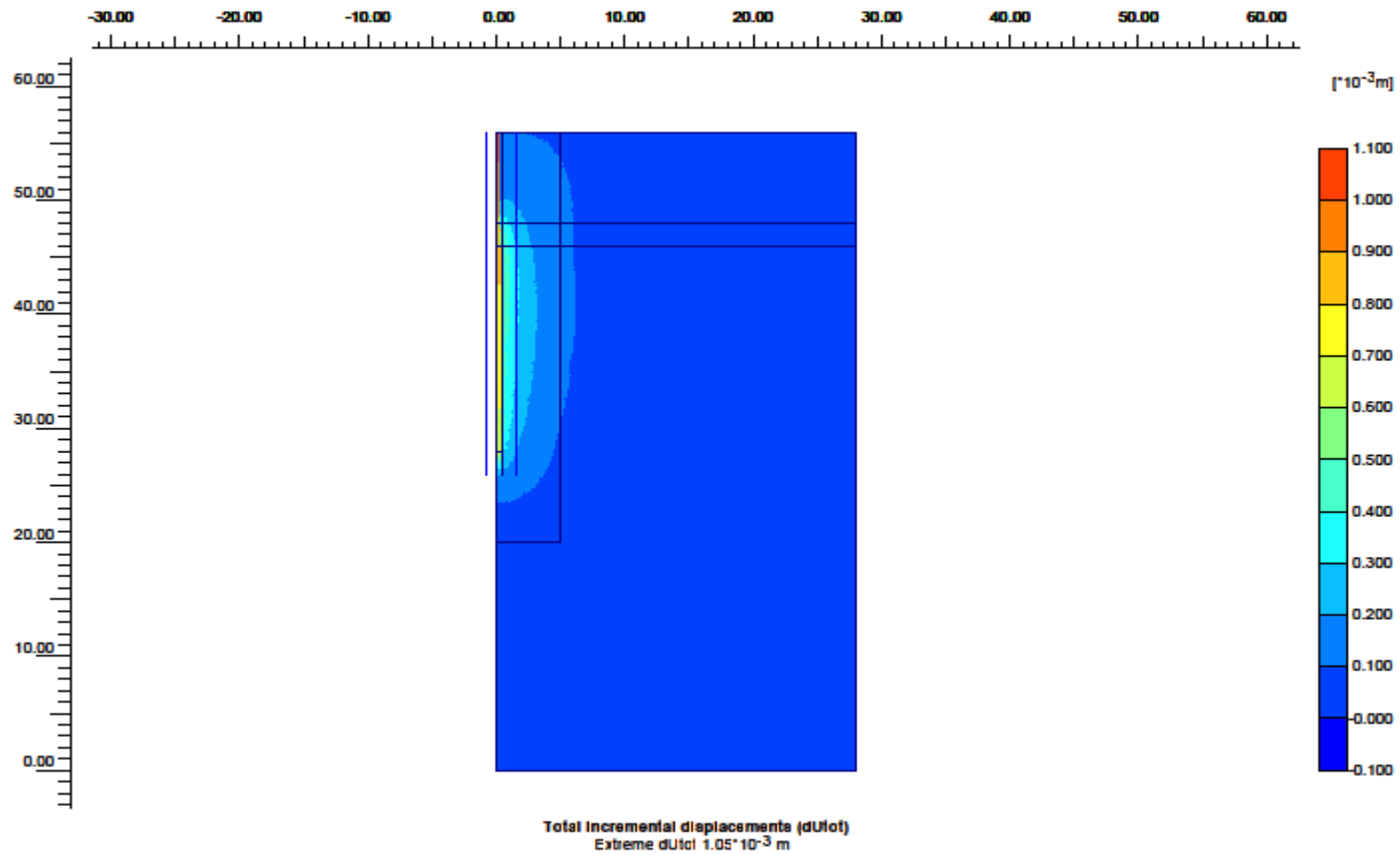


Figure B.8. Total incremental displacements.

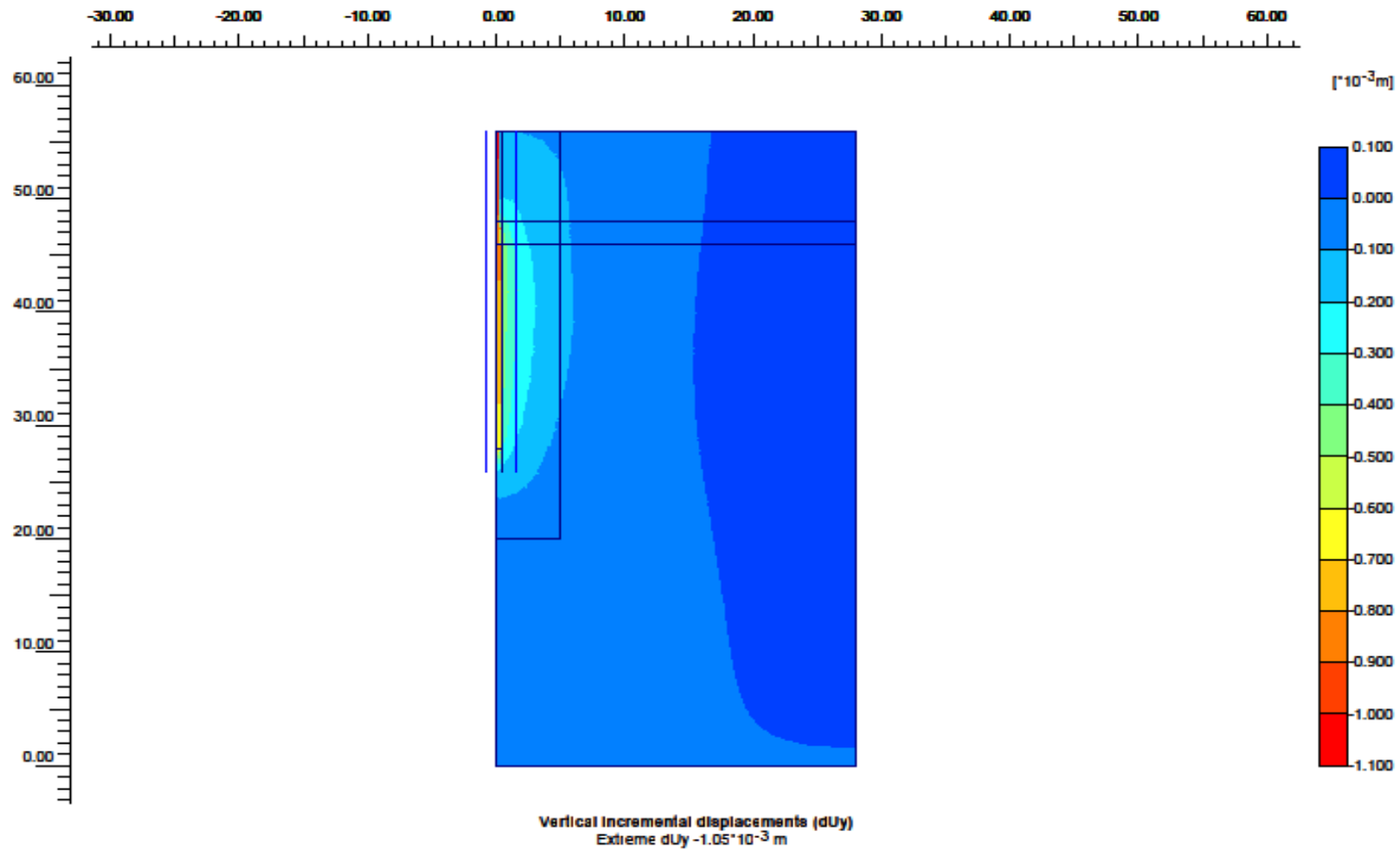


Figure B.9. Vertical incremental displacements.

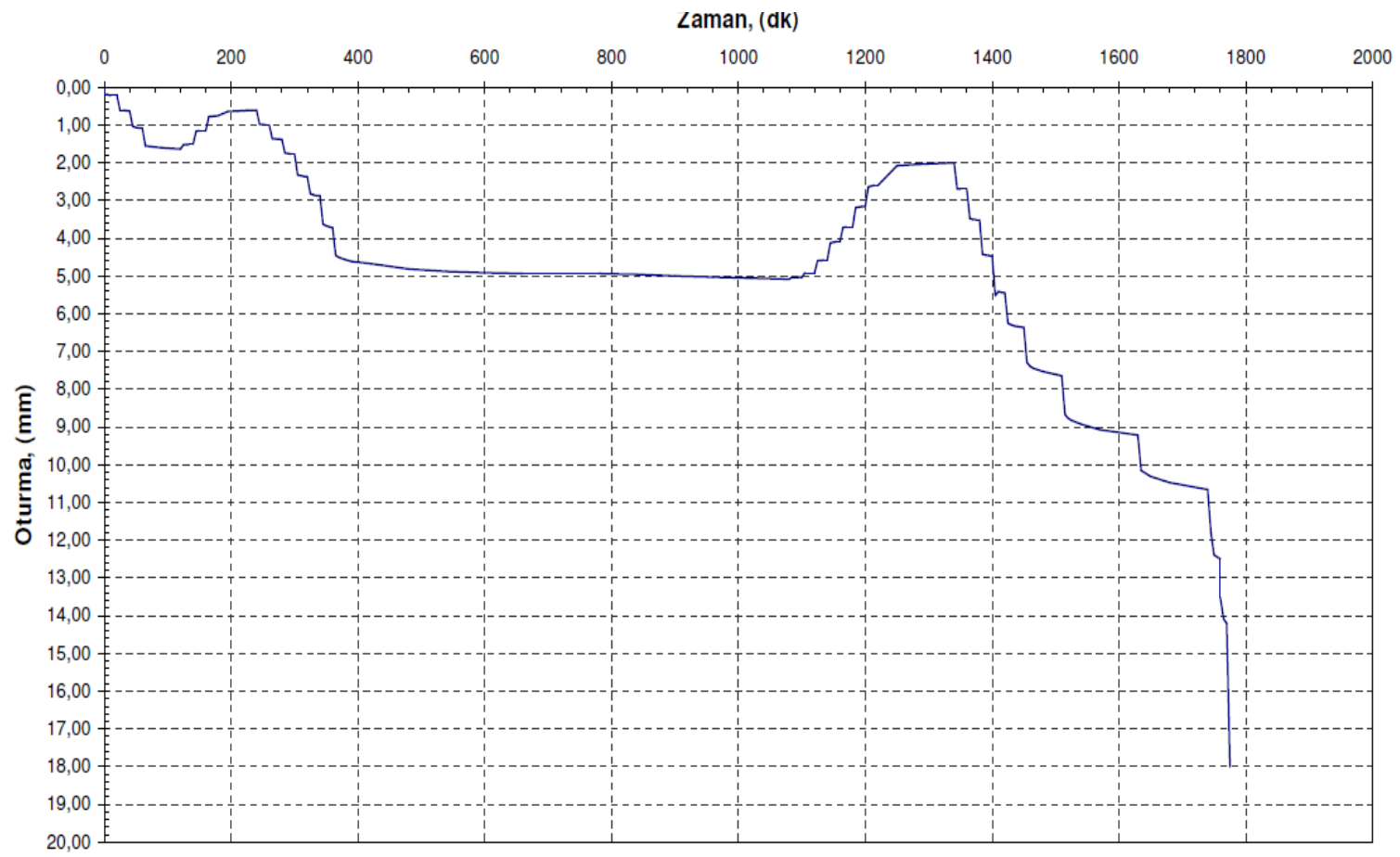


Figure B.10. Time versus Displacement.

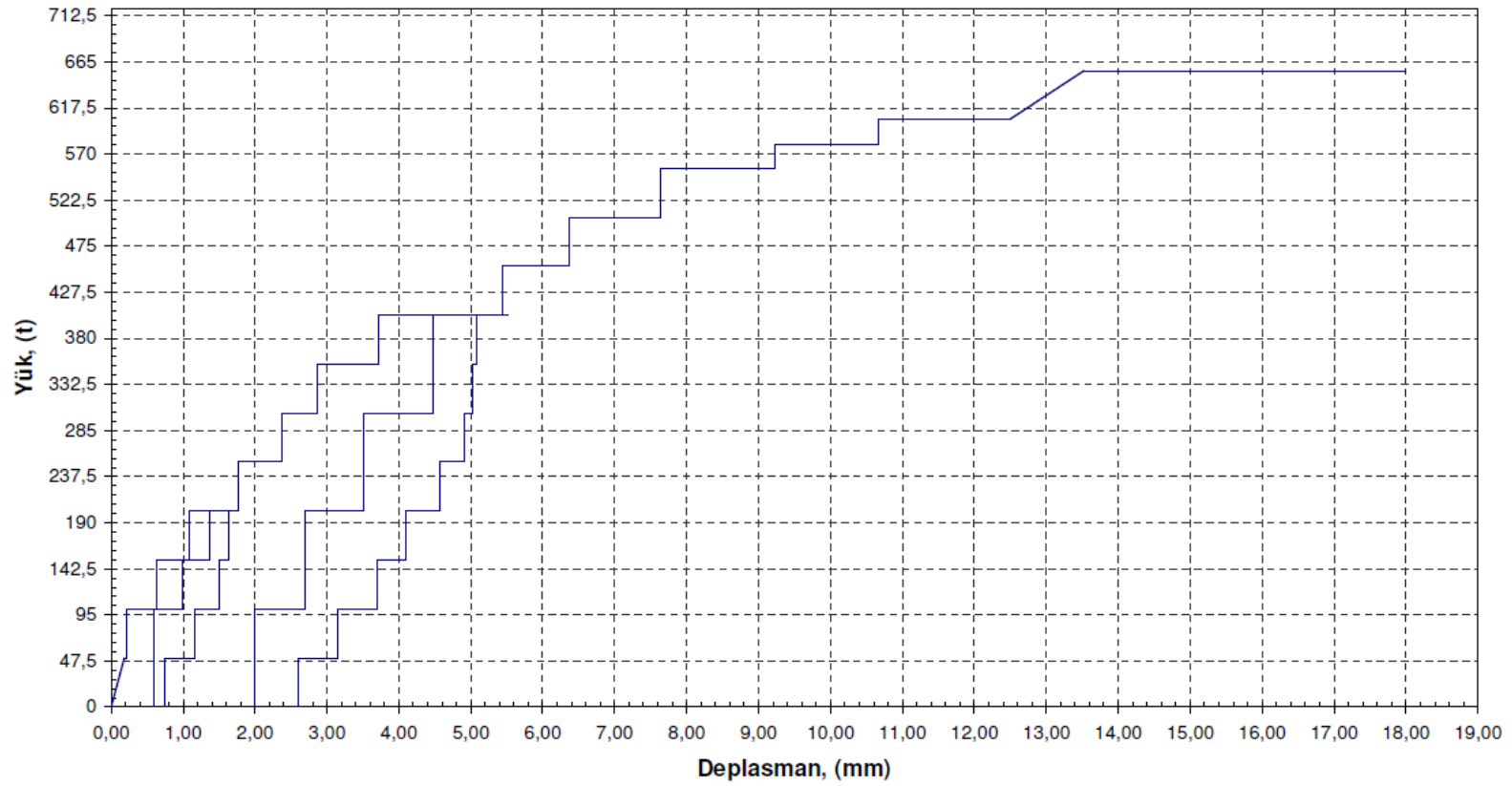


Figure B.11. Load versus Displacement.

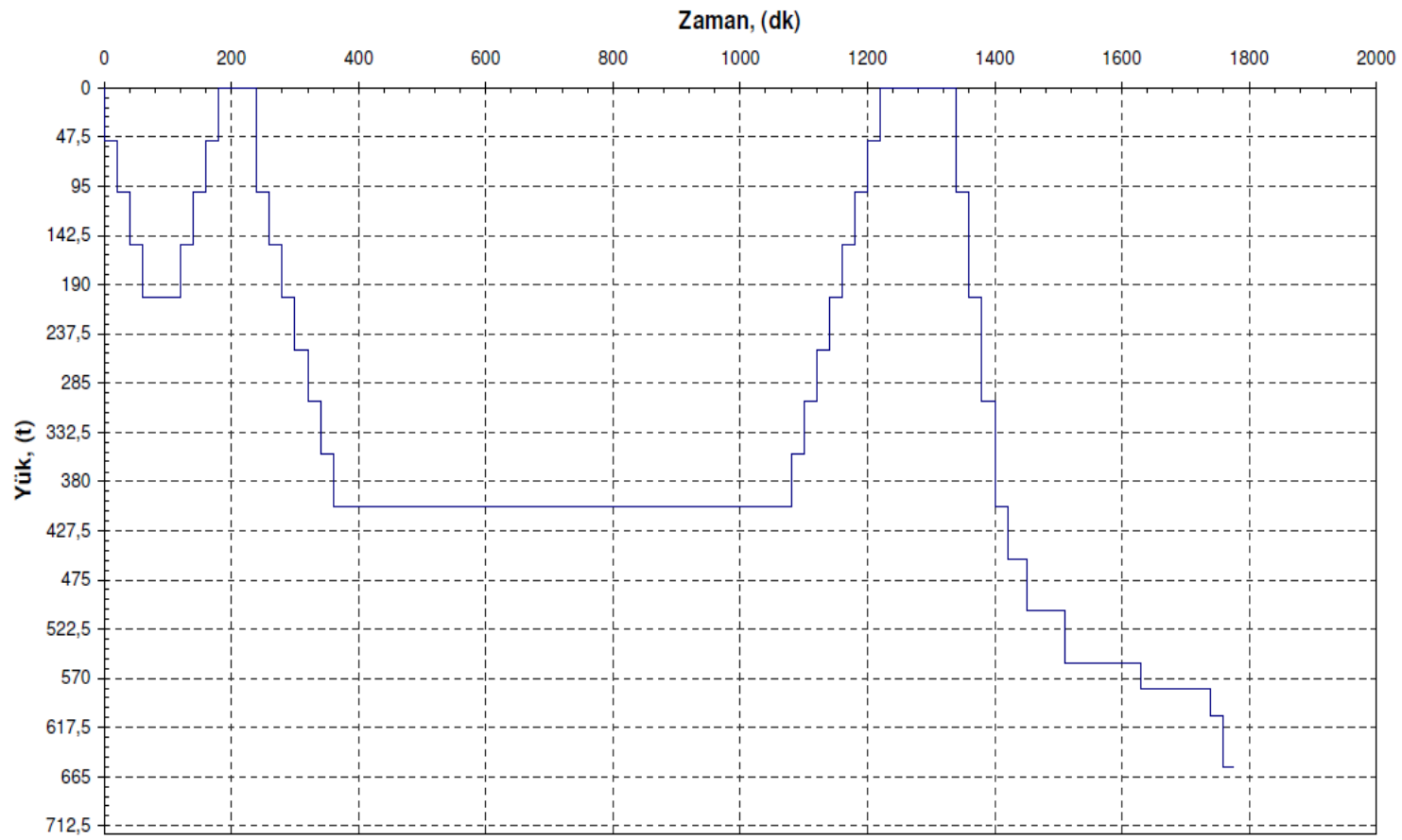


Figure B.12. Time versus Load.

APPENDIX C

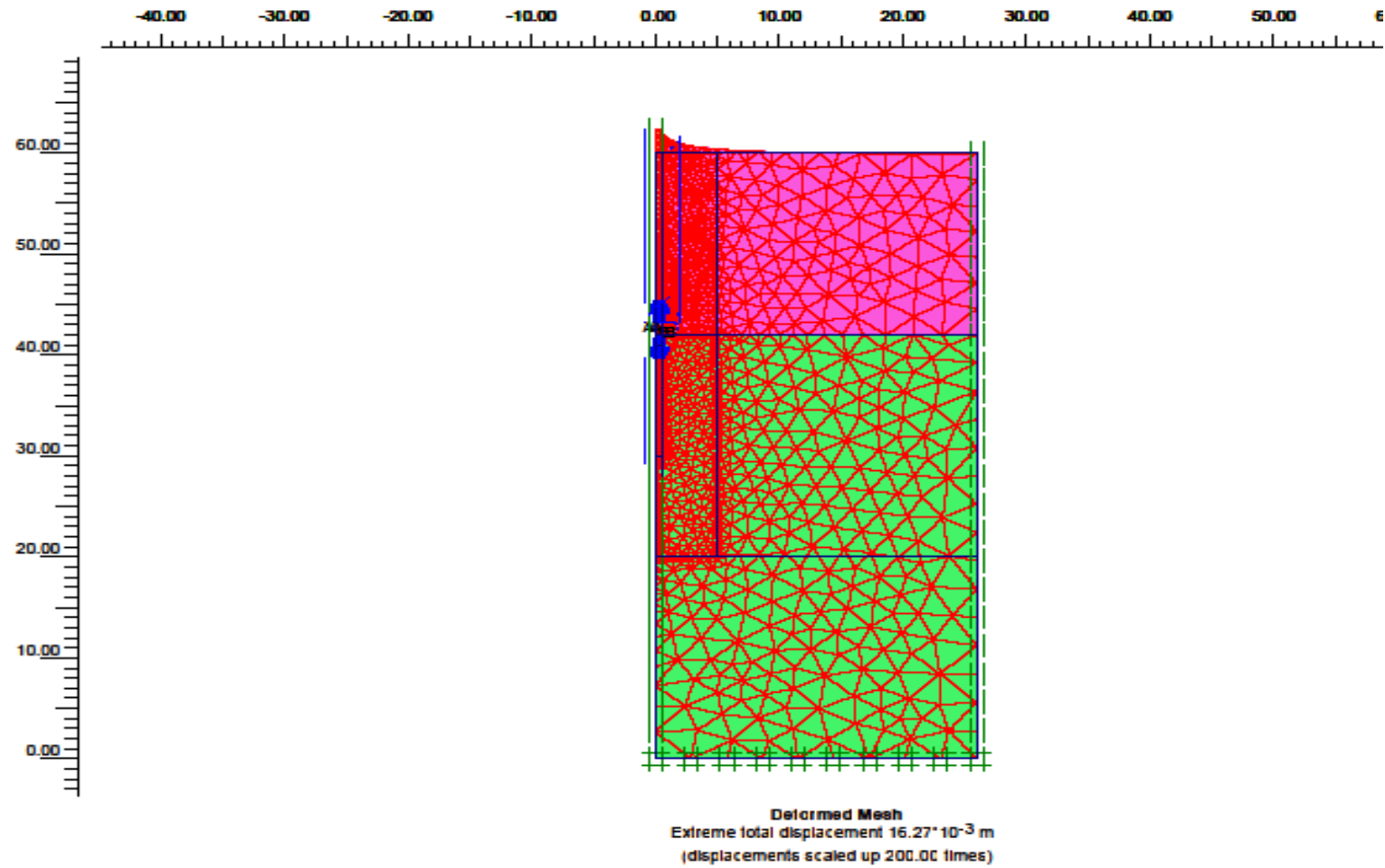


Figure C.2. Deformed mesh.

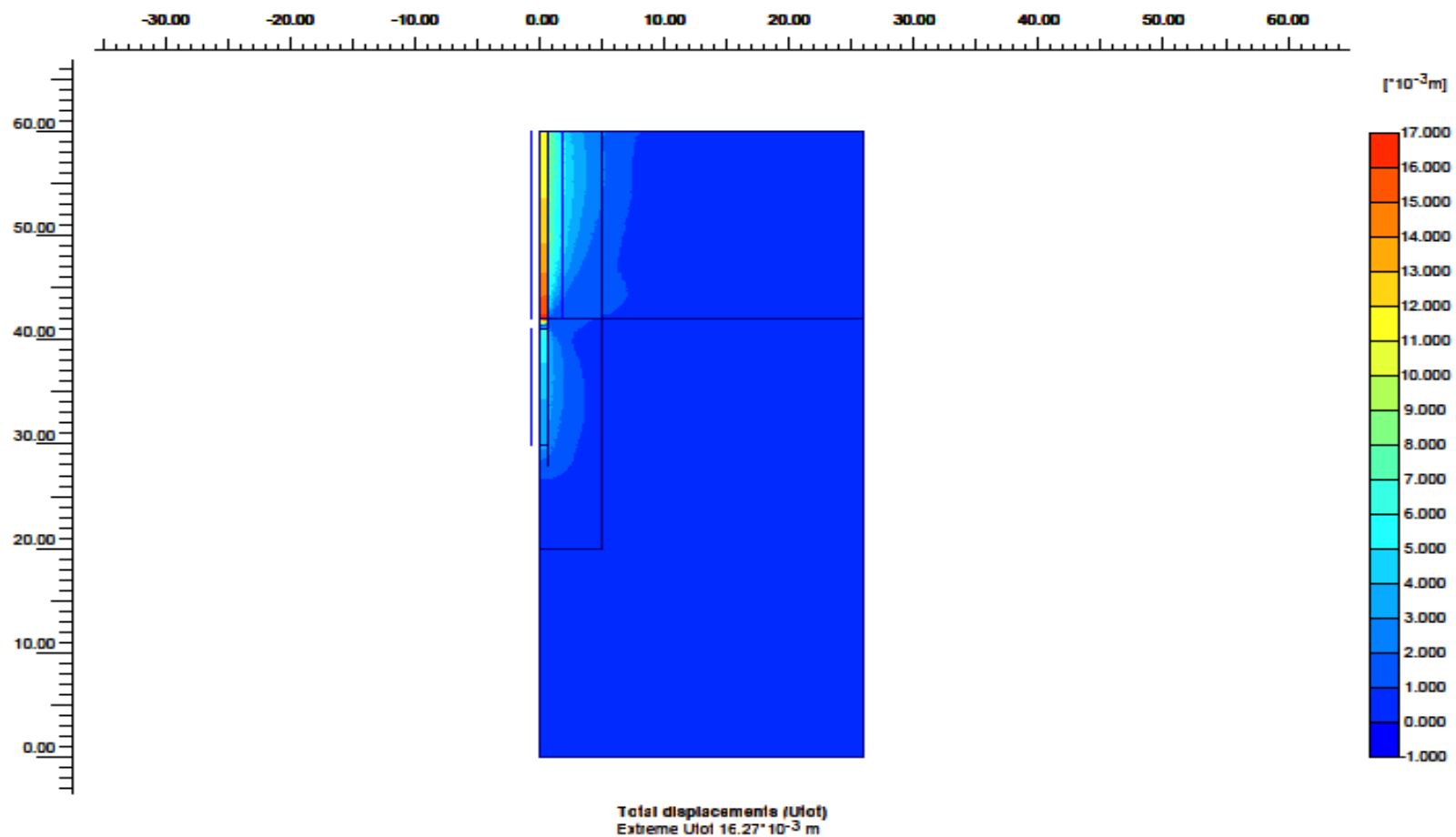


Figure C.3. Total displacements.

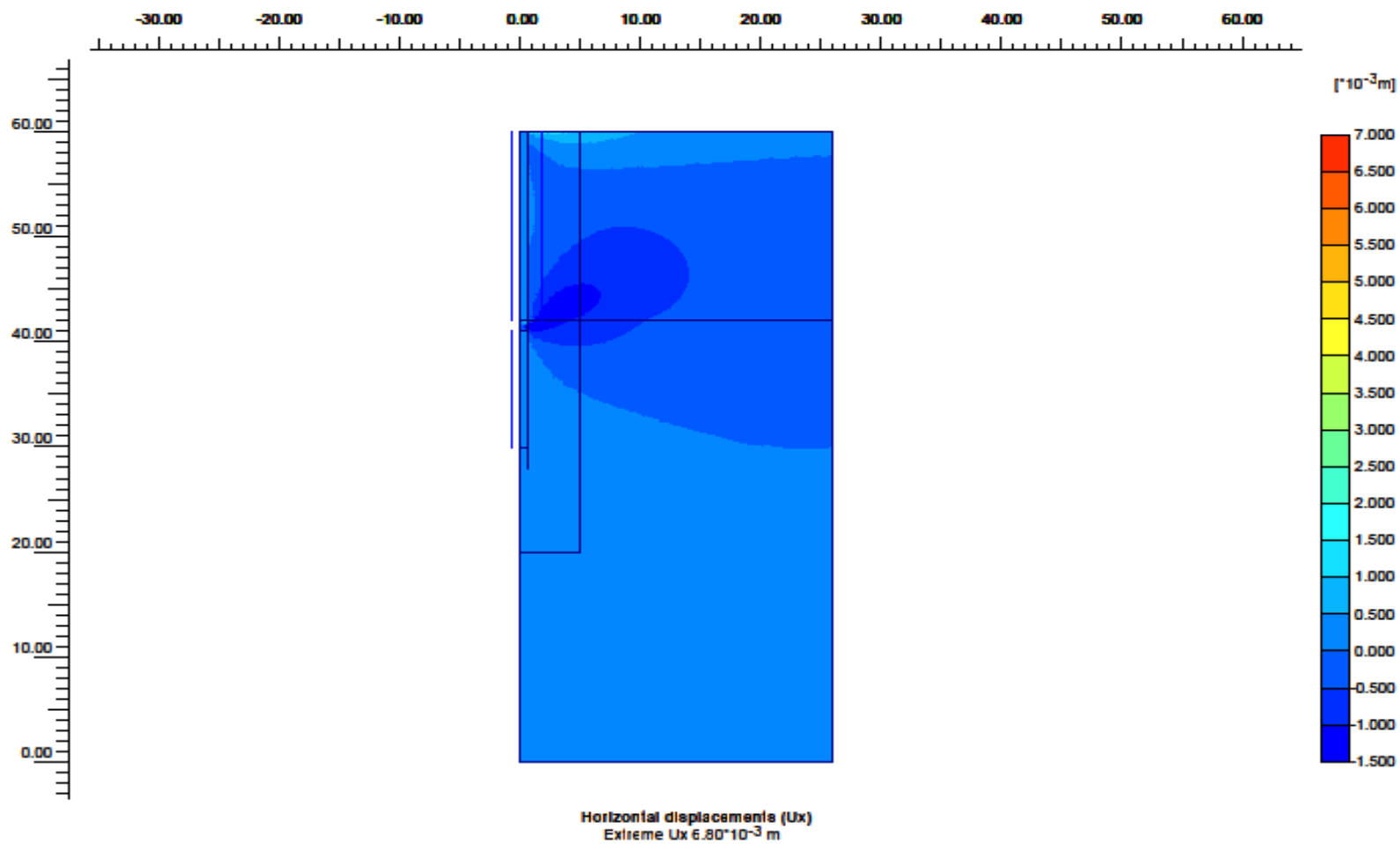


Figure C.4. Horizontal displacements.

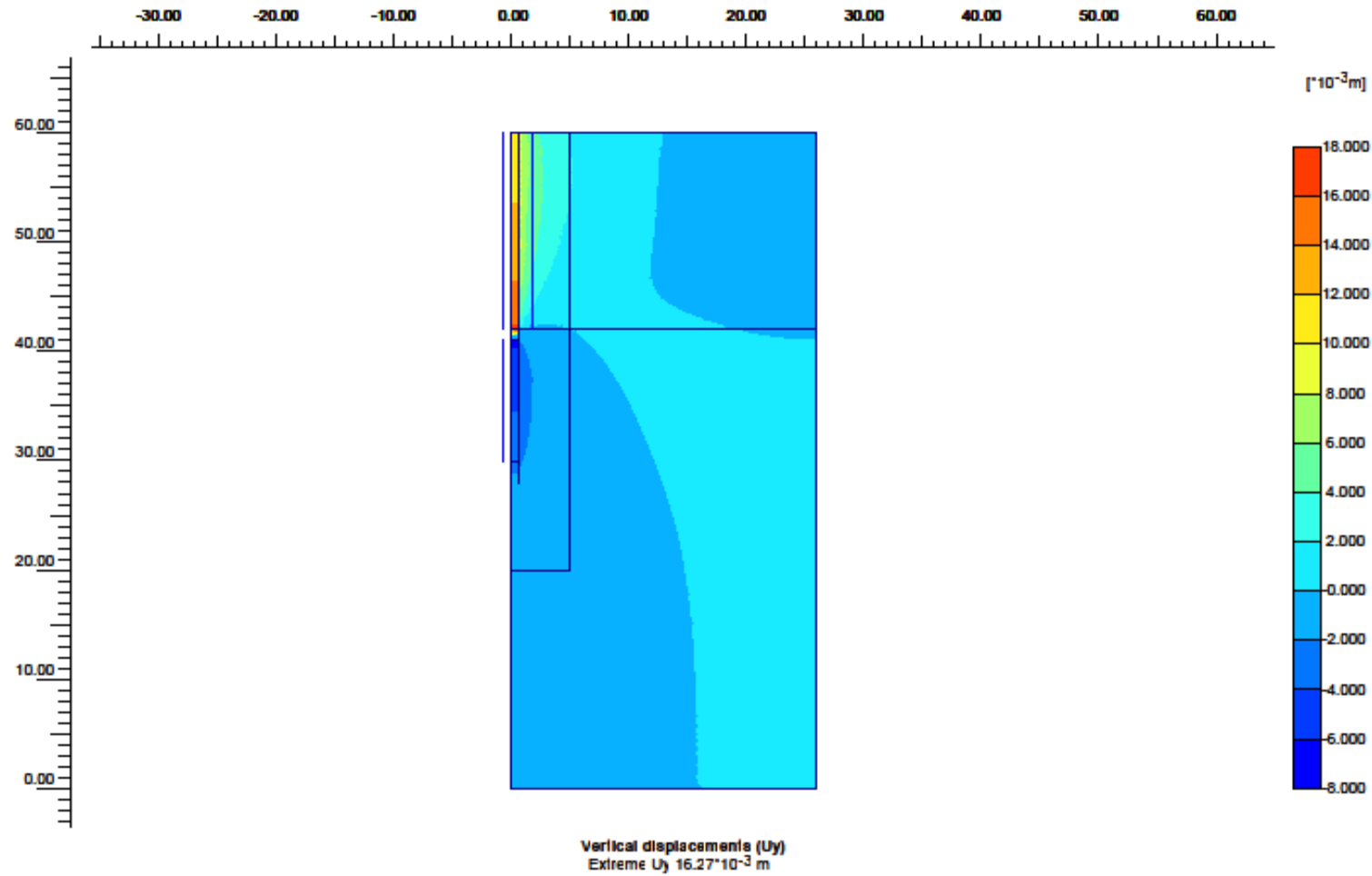


Figure C.5. Vertical displacements.

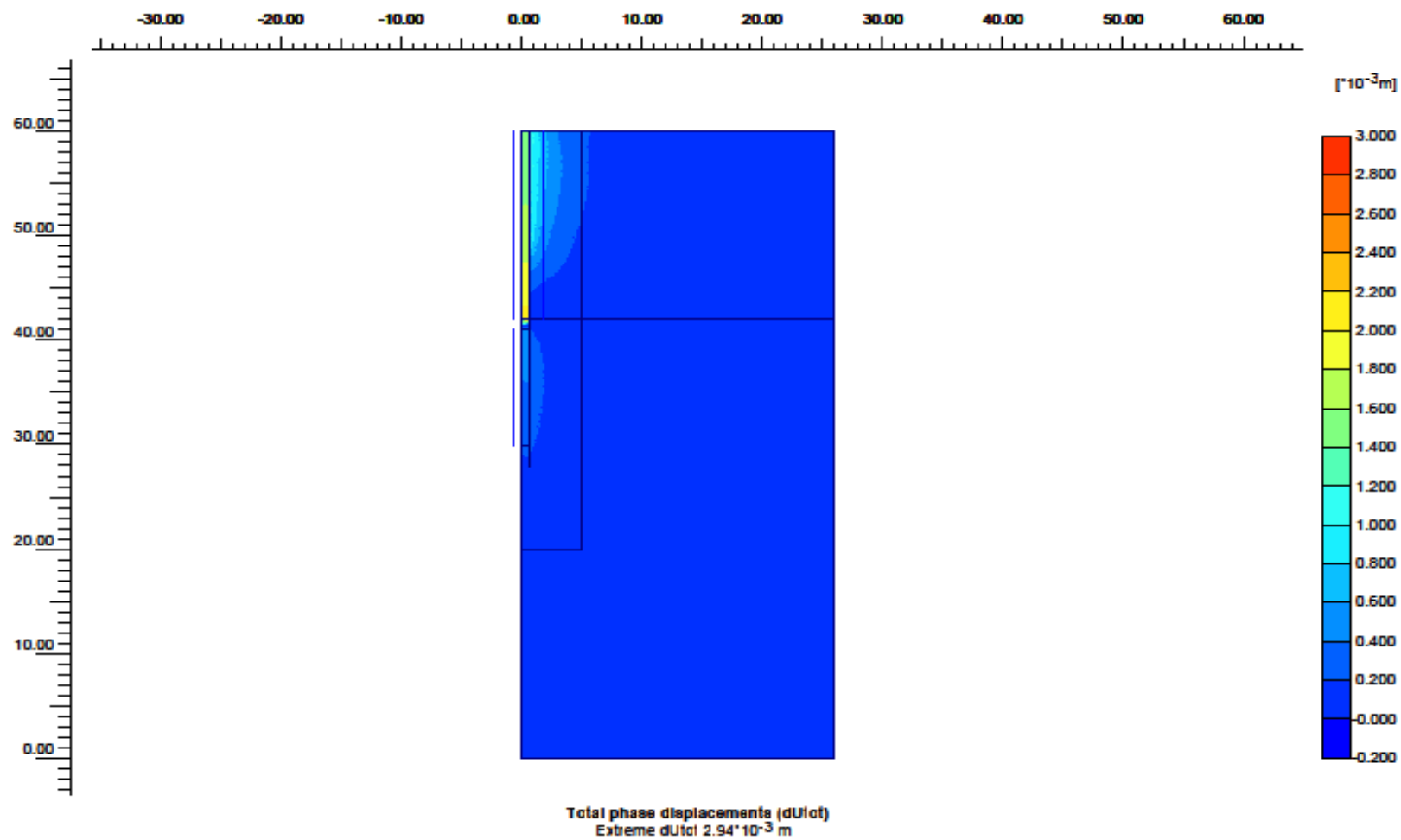


Figure C.6. Total phase displacements.

Figure C.7. Vertical phase displacements.

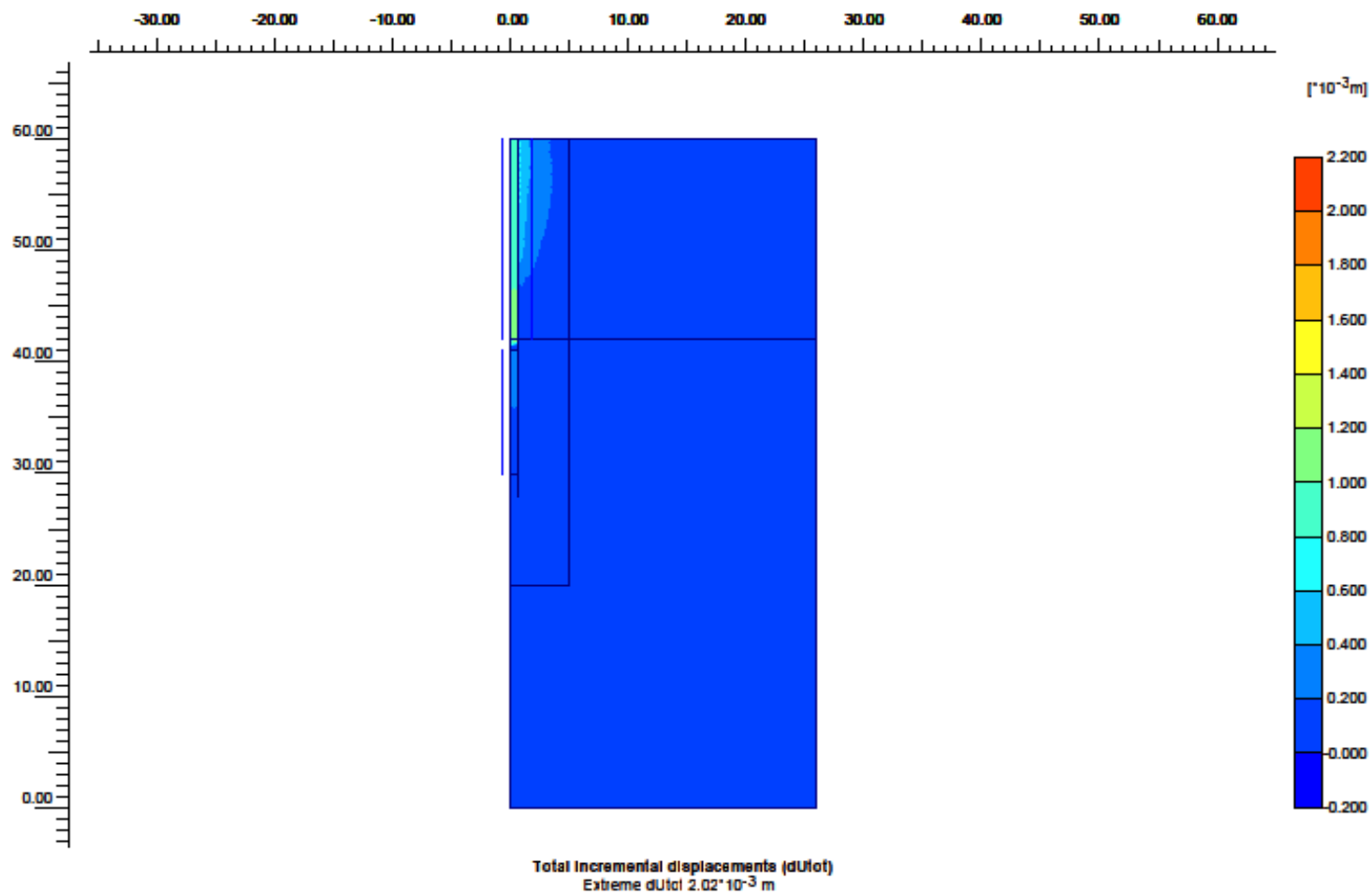


Figure C.8. Total incremental displacements.

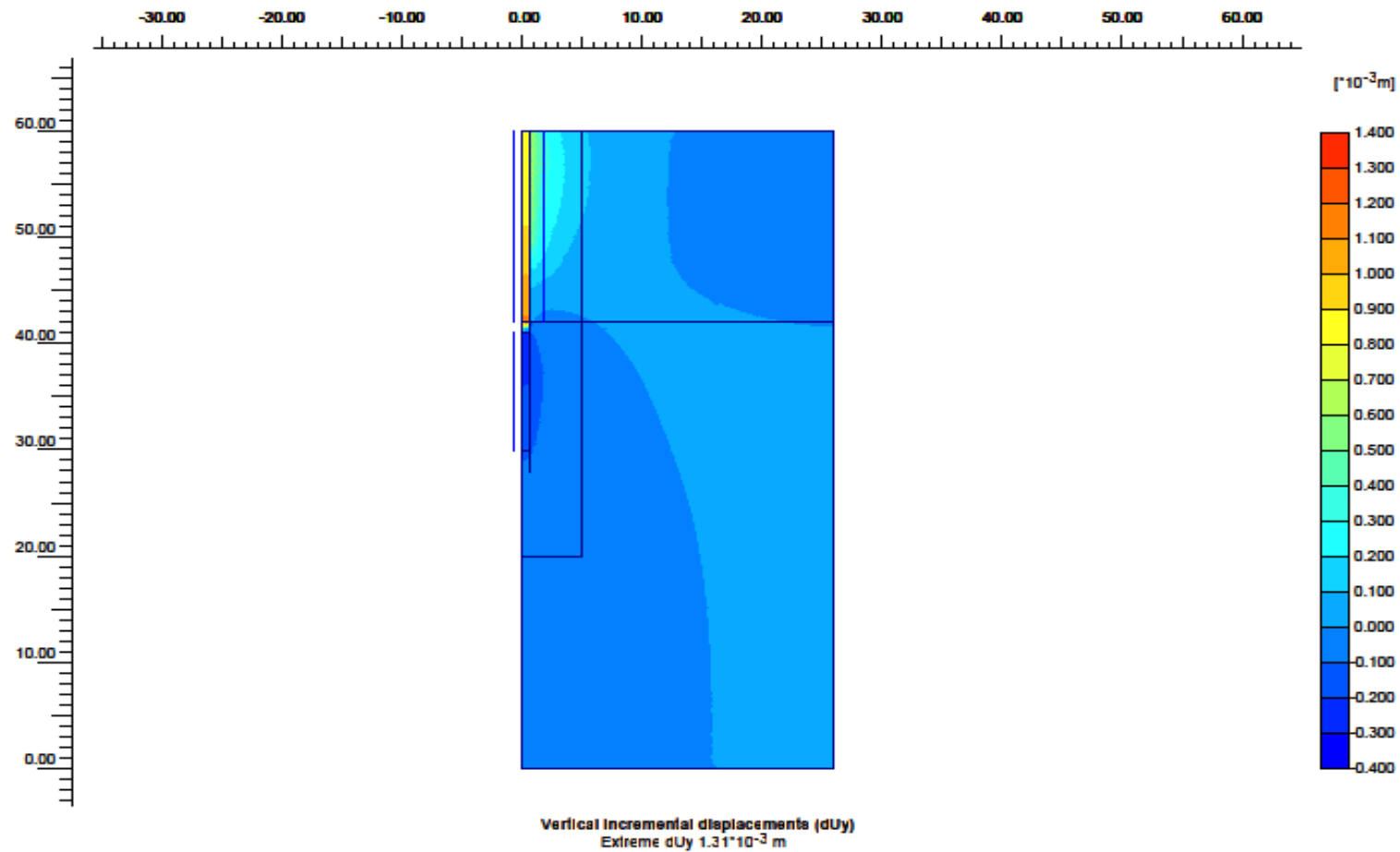


Figure C.9. Vertical incremental displacements.

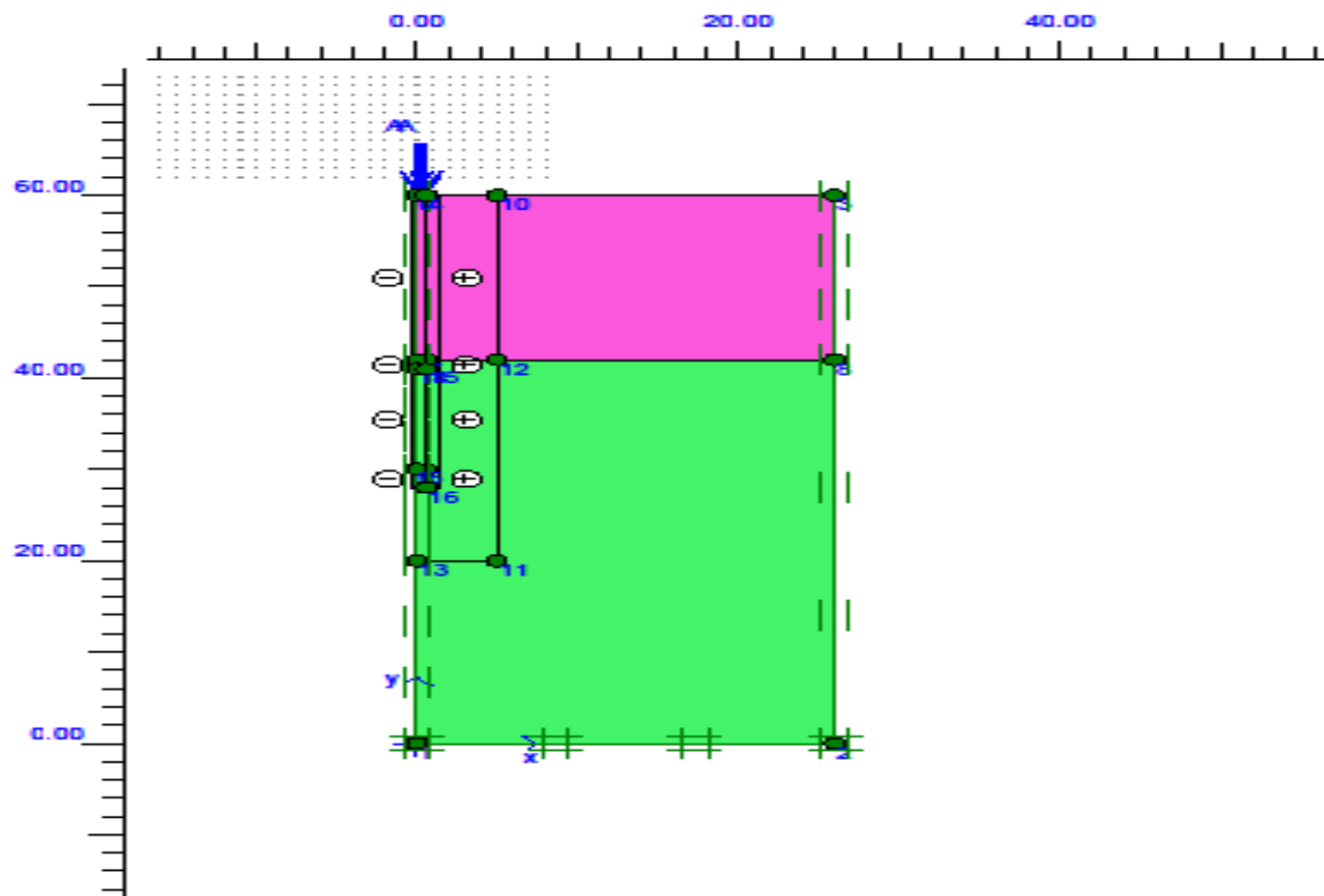


Figure C.10. Finite element model static case.

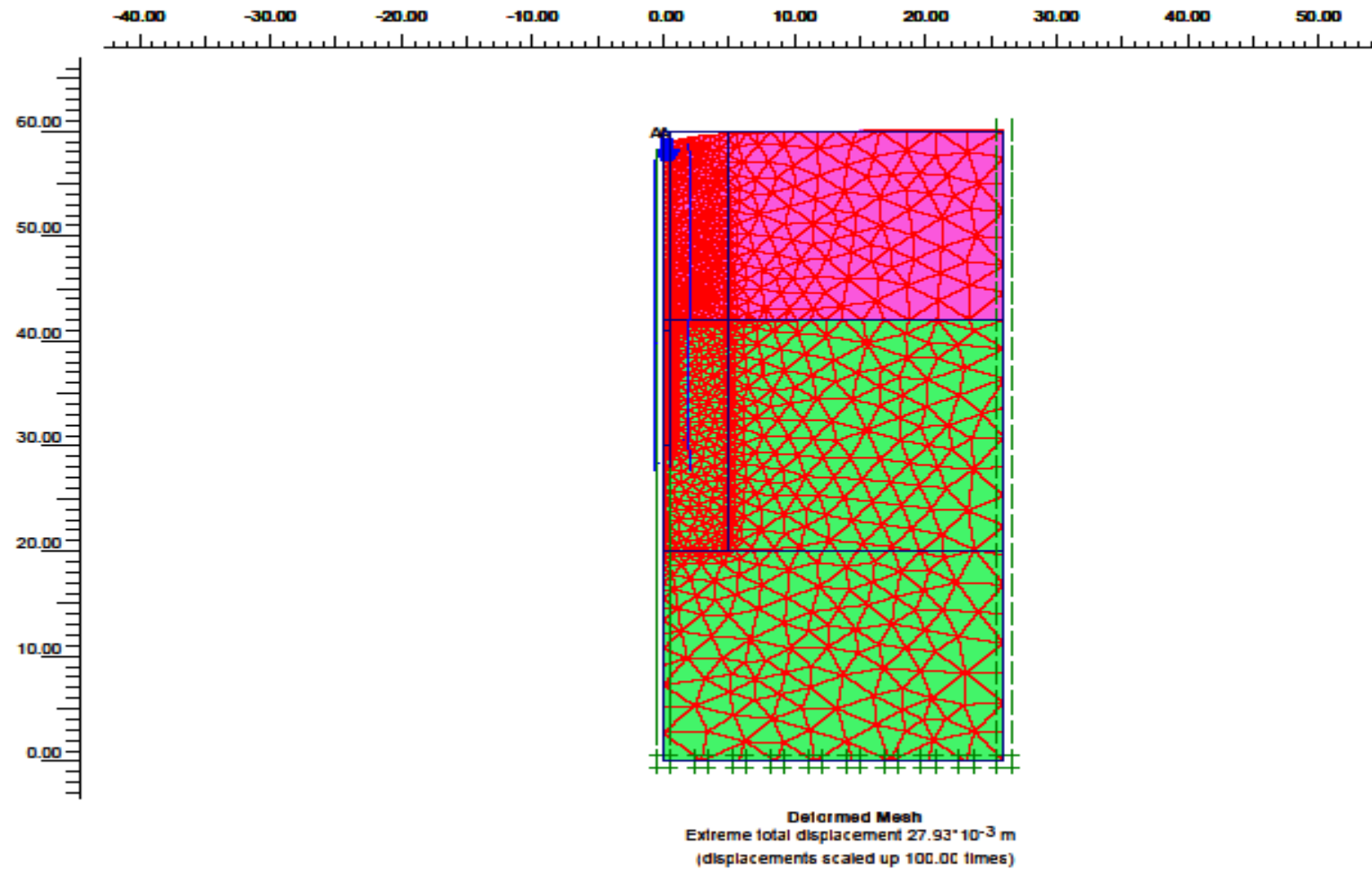


Figure C.11. Deformed mesh static case.

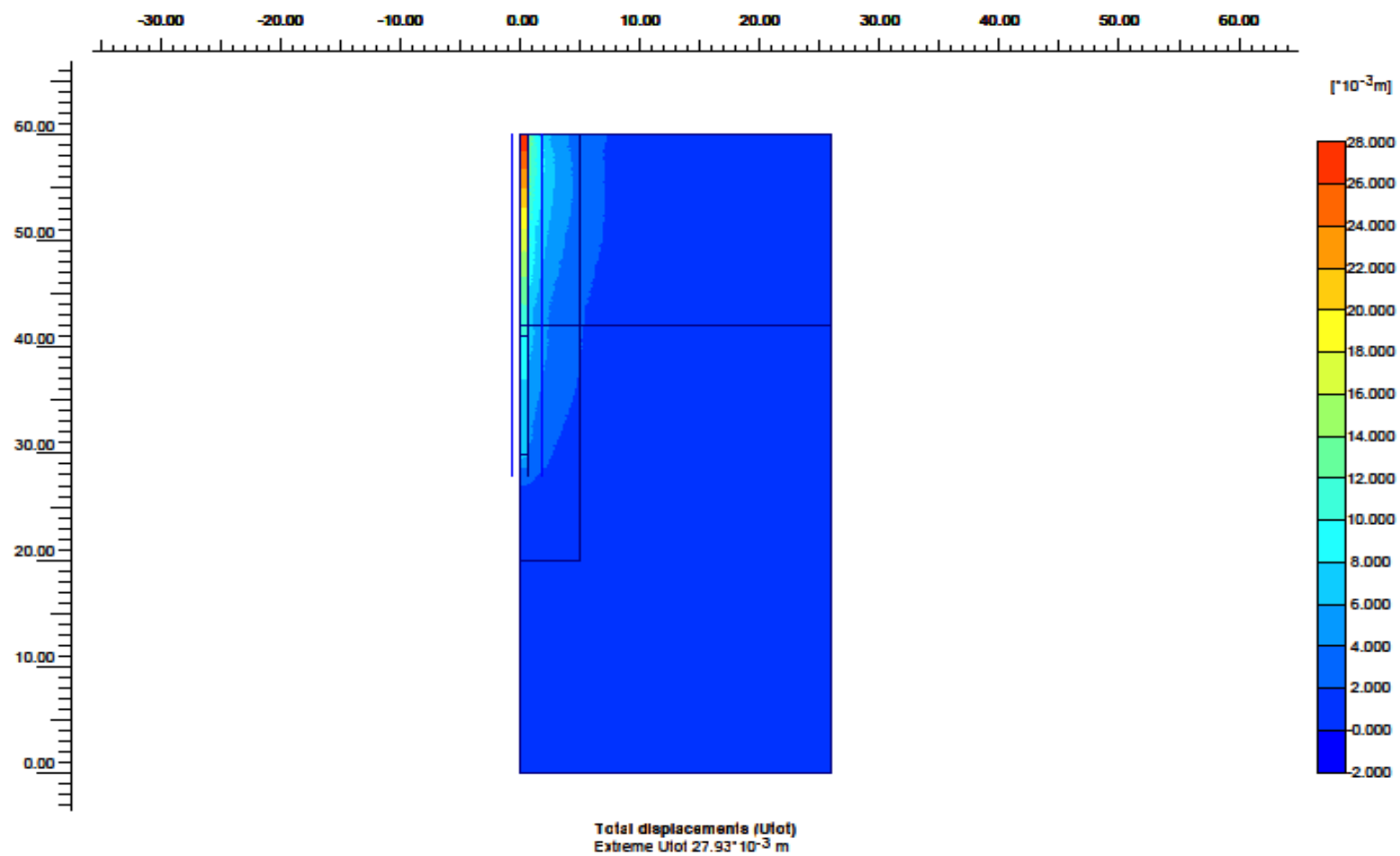


Figure C.12. Total displacements static case.

Figure C.13. Horizontal displacements static case.

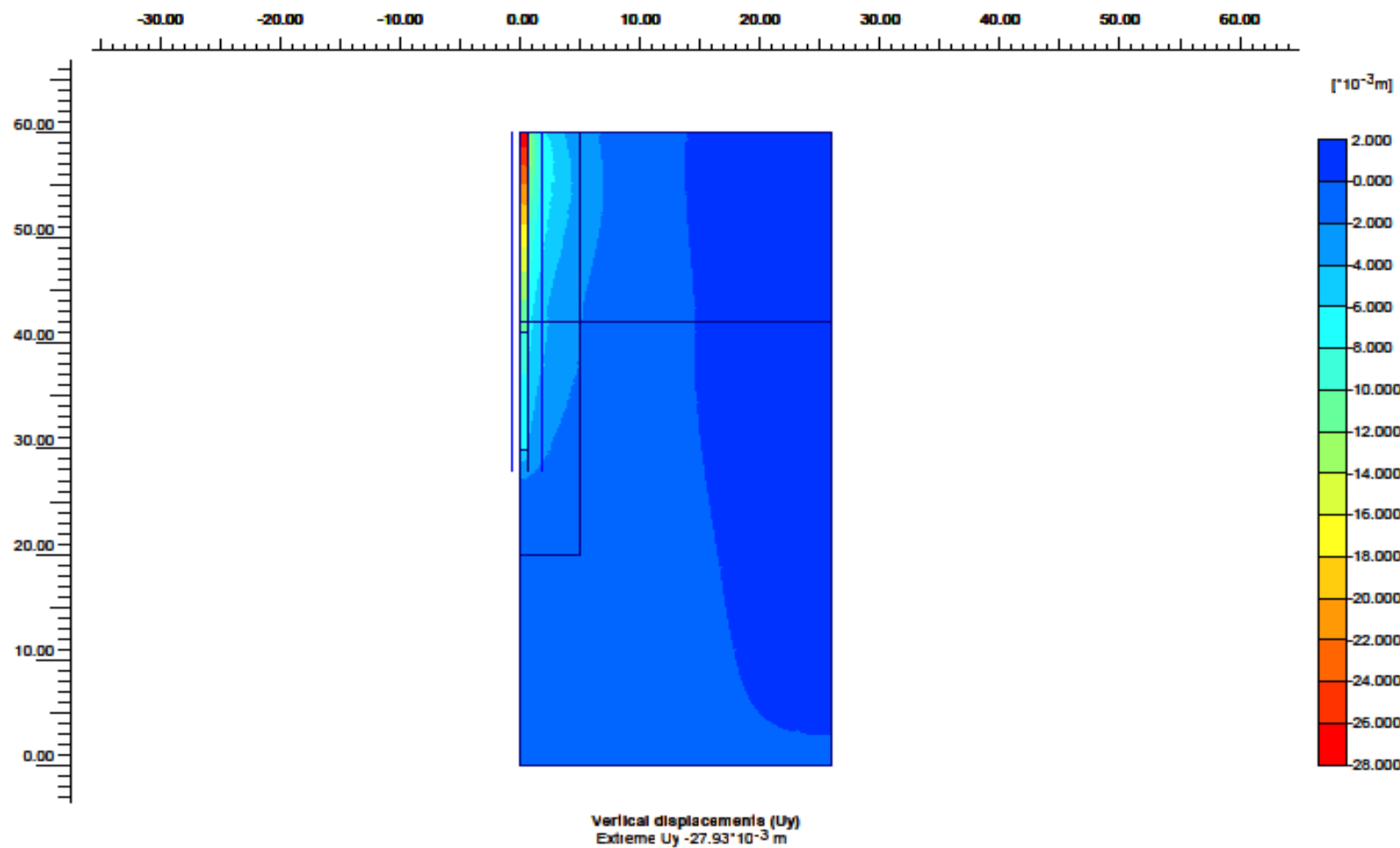


Figure C.14. Vertical displacements static case.

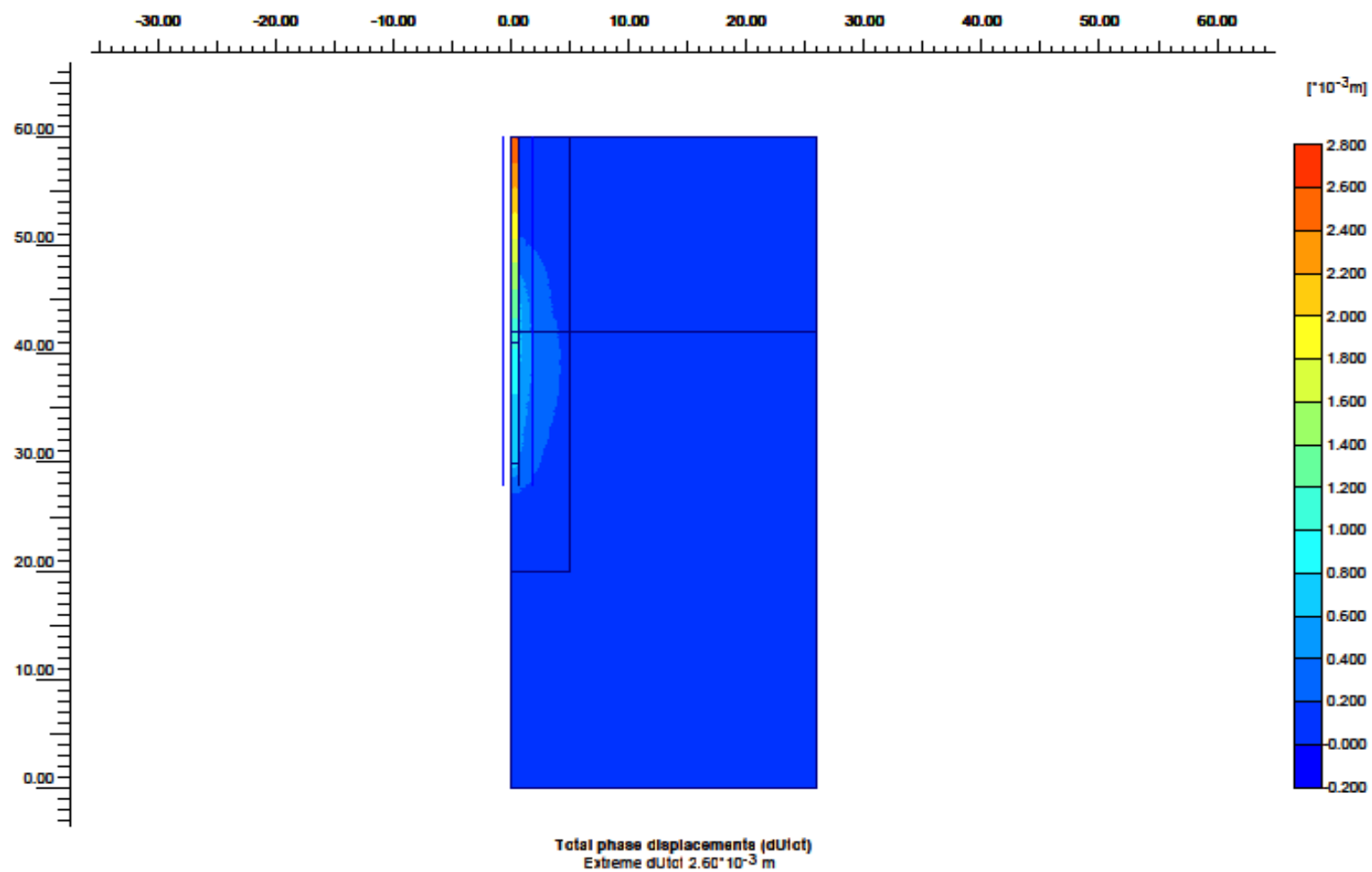


Figure C.15. Total phase displacements static case.

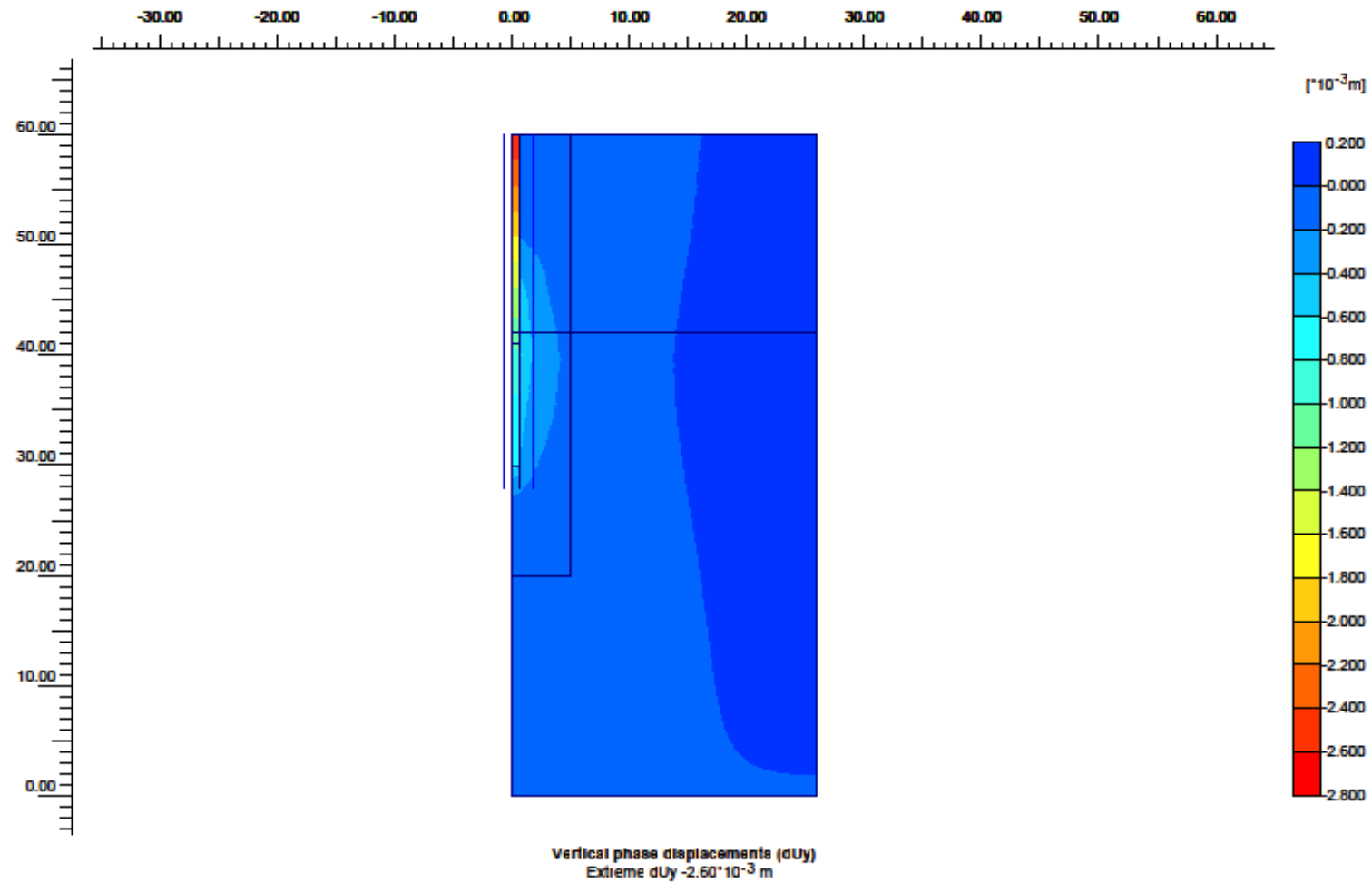


Figure C.16. Vertical phase displacements static case.

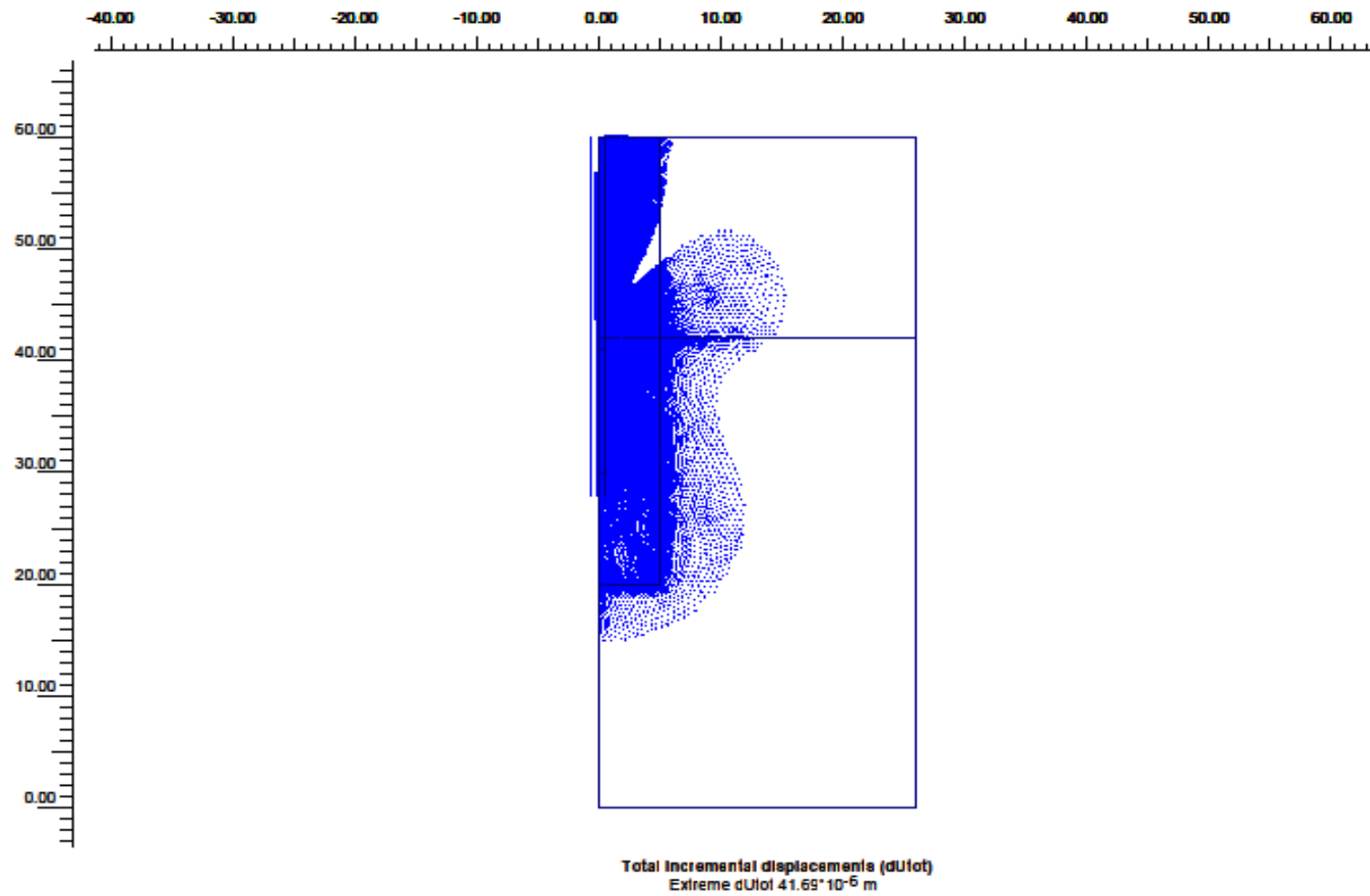


Figure C. 17. Total incremental displacements static case.

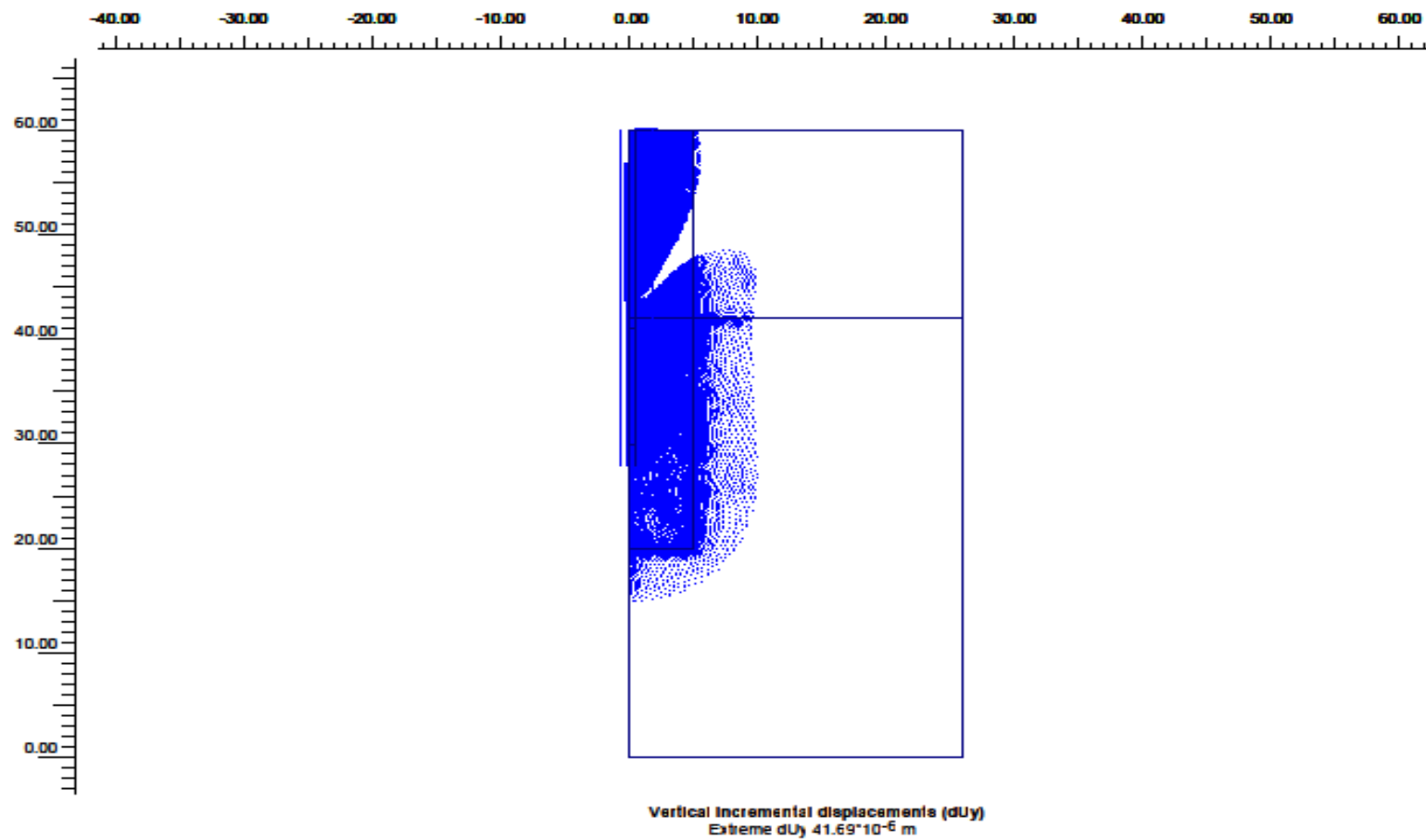


Figure C.18. Vertical incremental displacements static case.



Time-Osterberg Cell Displacement

TP1 - Sky Tower - Istanbul, Turkey

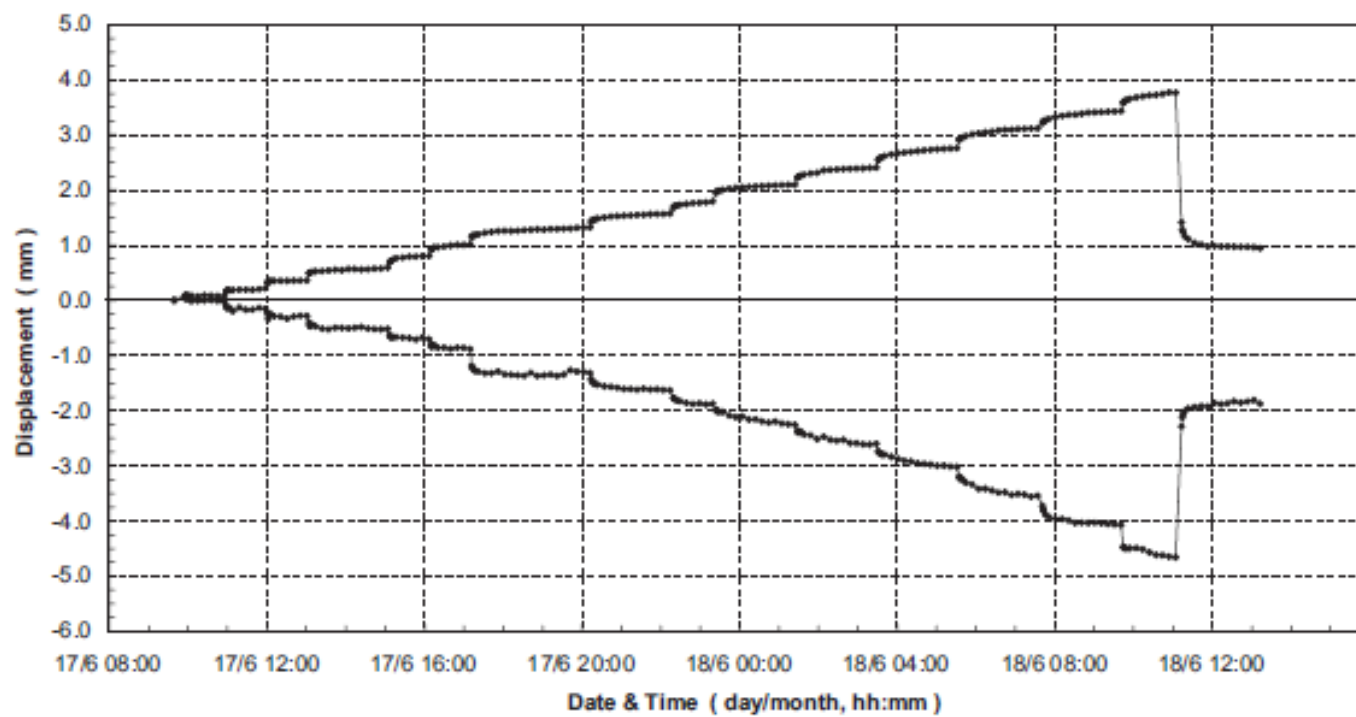


Figure C.19. Time versus Displacement.



Osterberg Cell Load-Displacement

TP1 - Sky Tower - Istanbul, Turkey

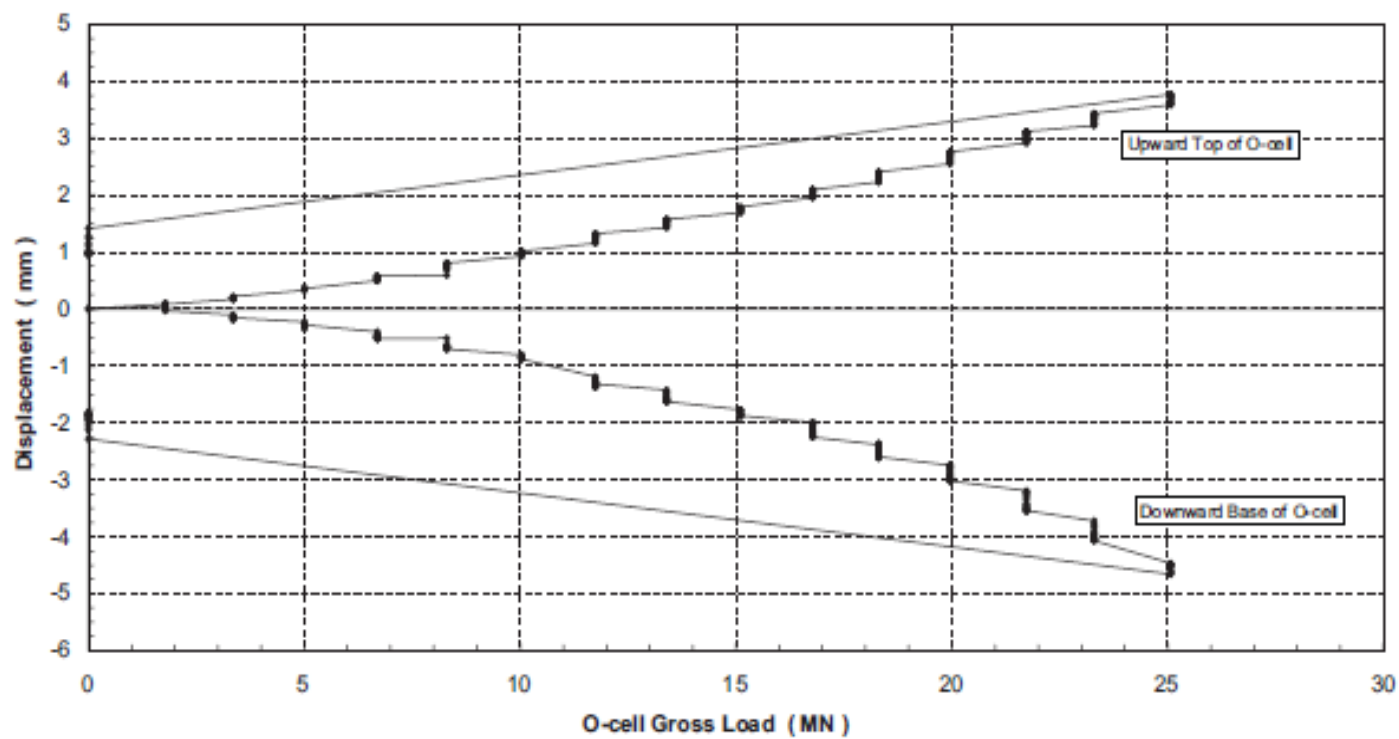


Figure C.20. Load versus Displacement.



Time-Osterberg Cell Load
TP1 - Sky Tower - Istanbul, Turkey

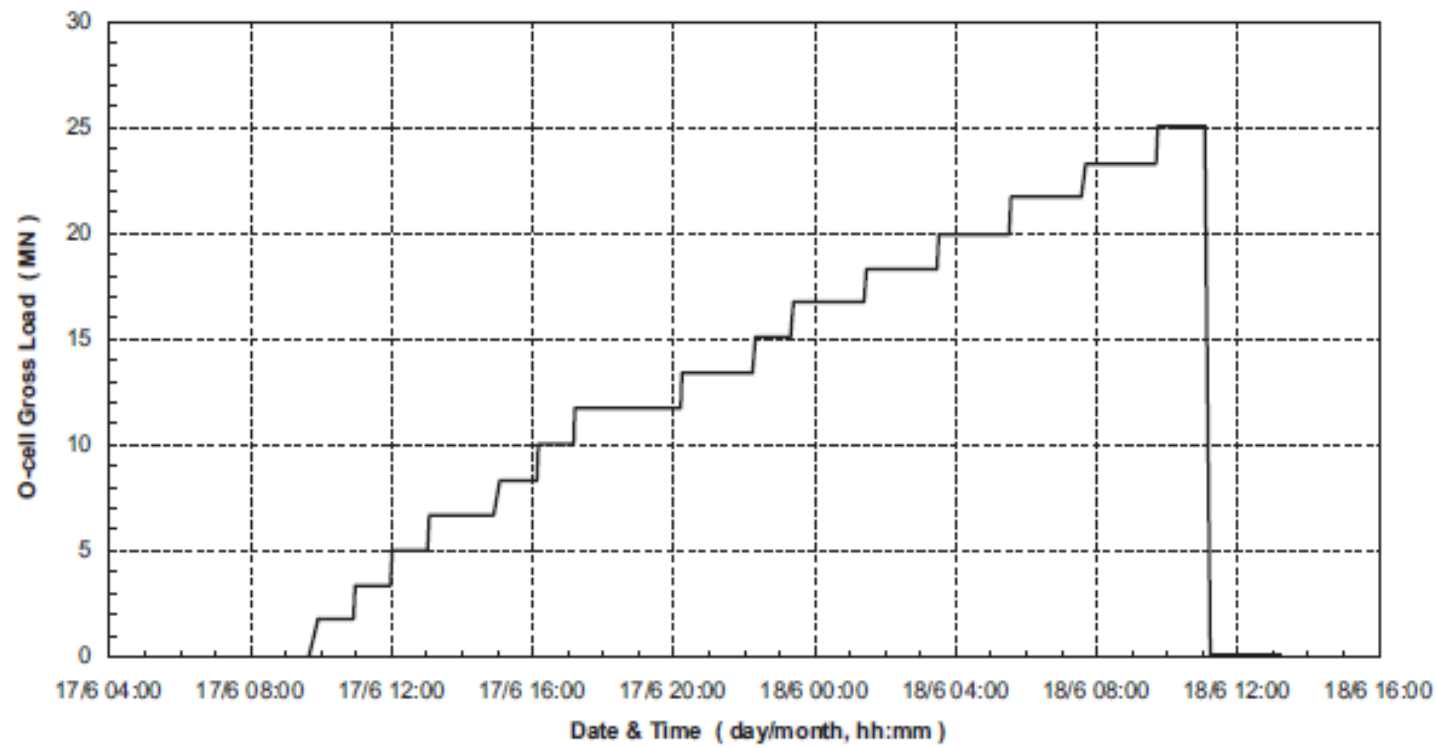


Figure C.21. Time versus Load.

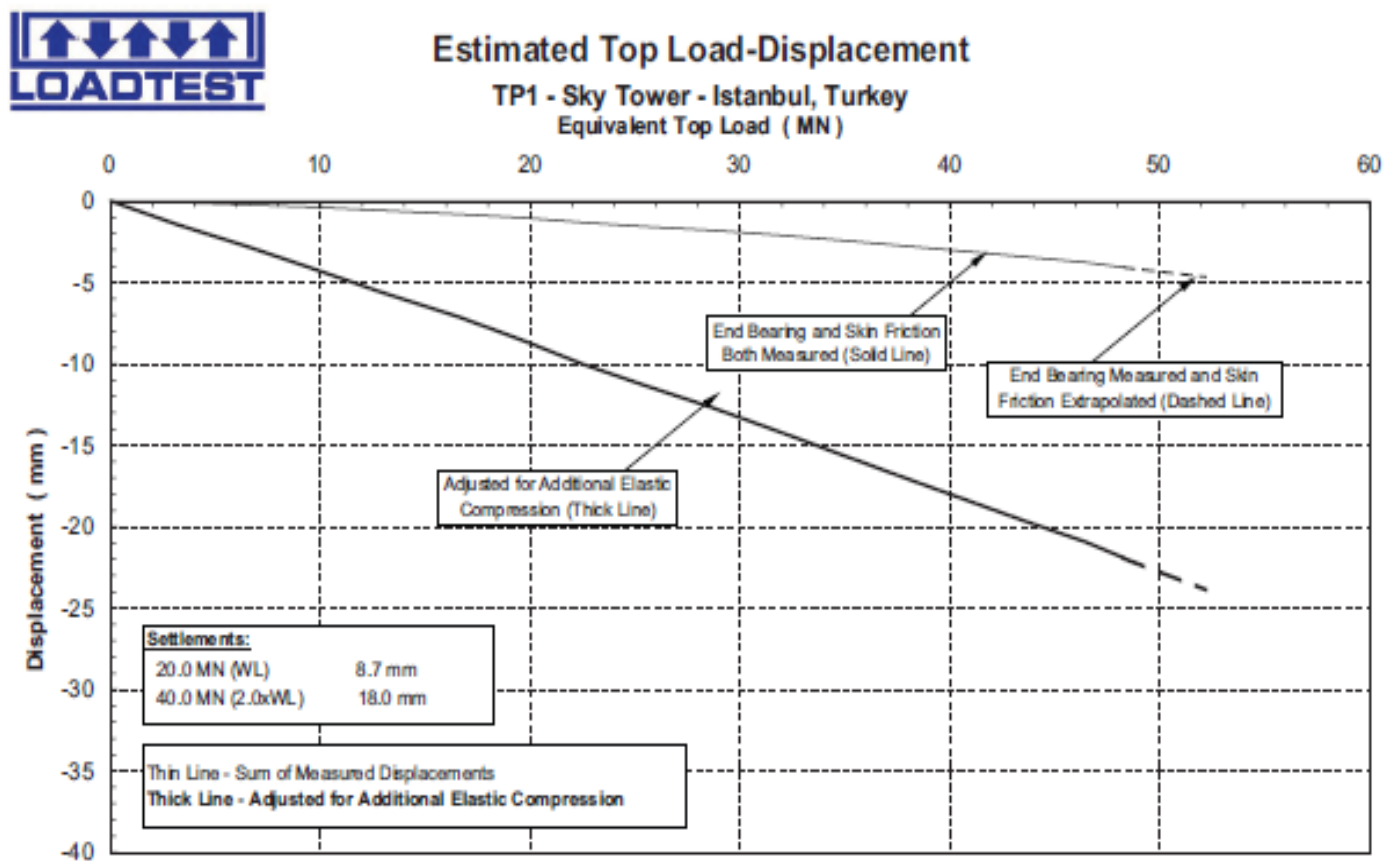


Figure C.22. Estimated Top Load Displacement.

APPENDIX D

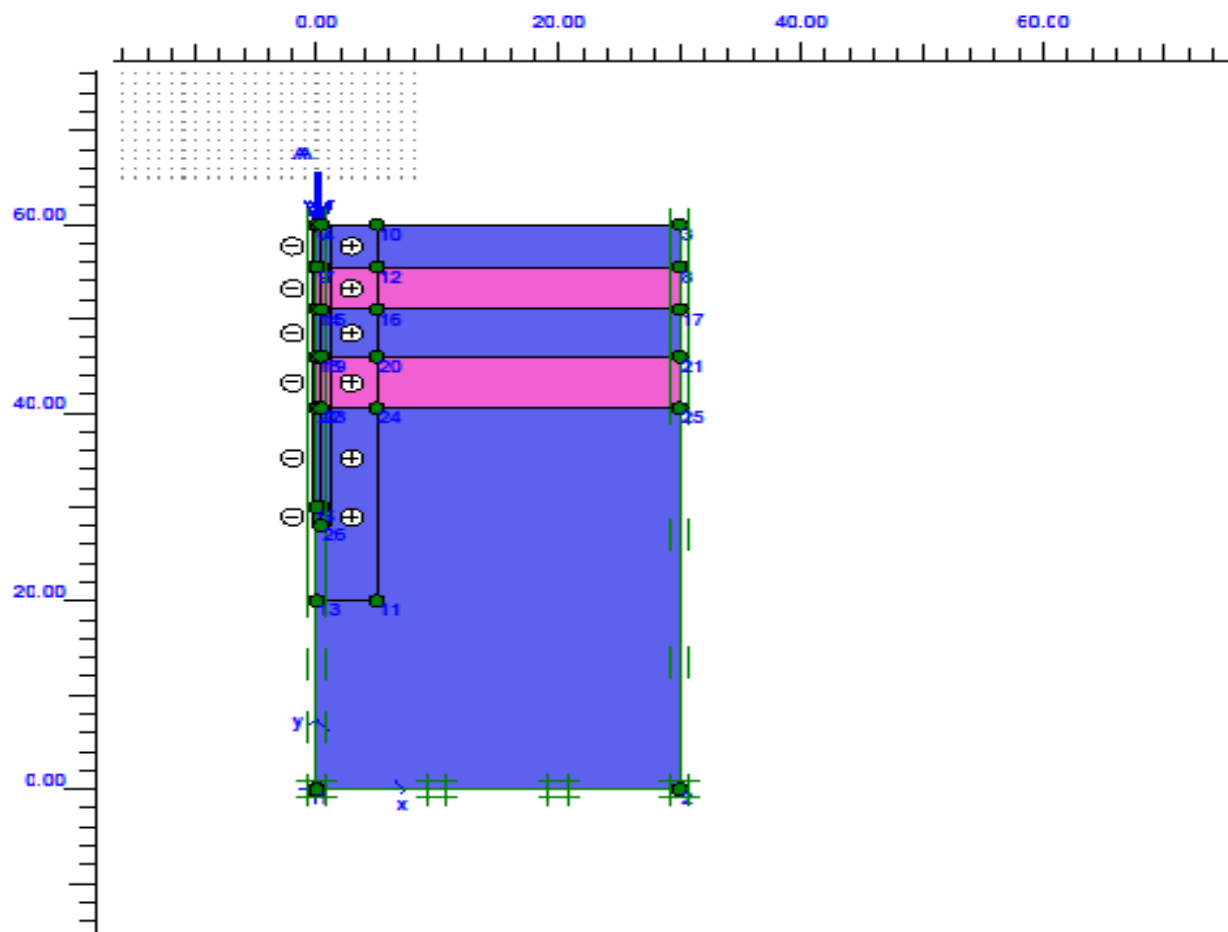


Figure D.1. Finite element model.

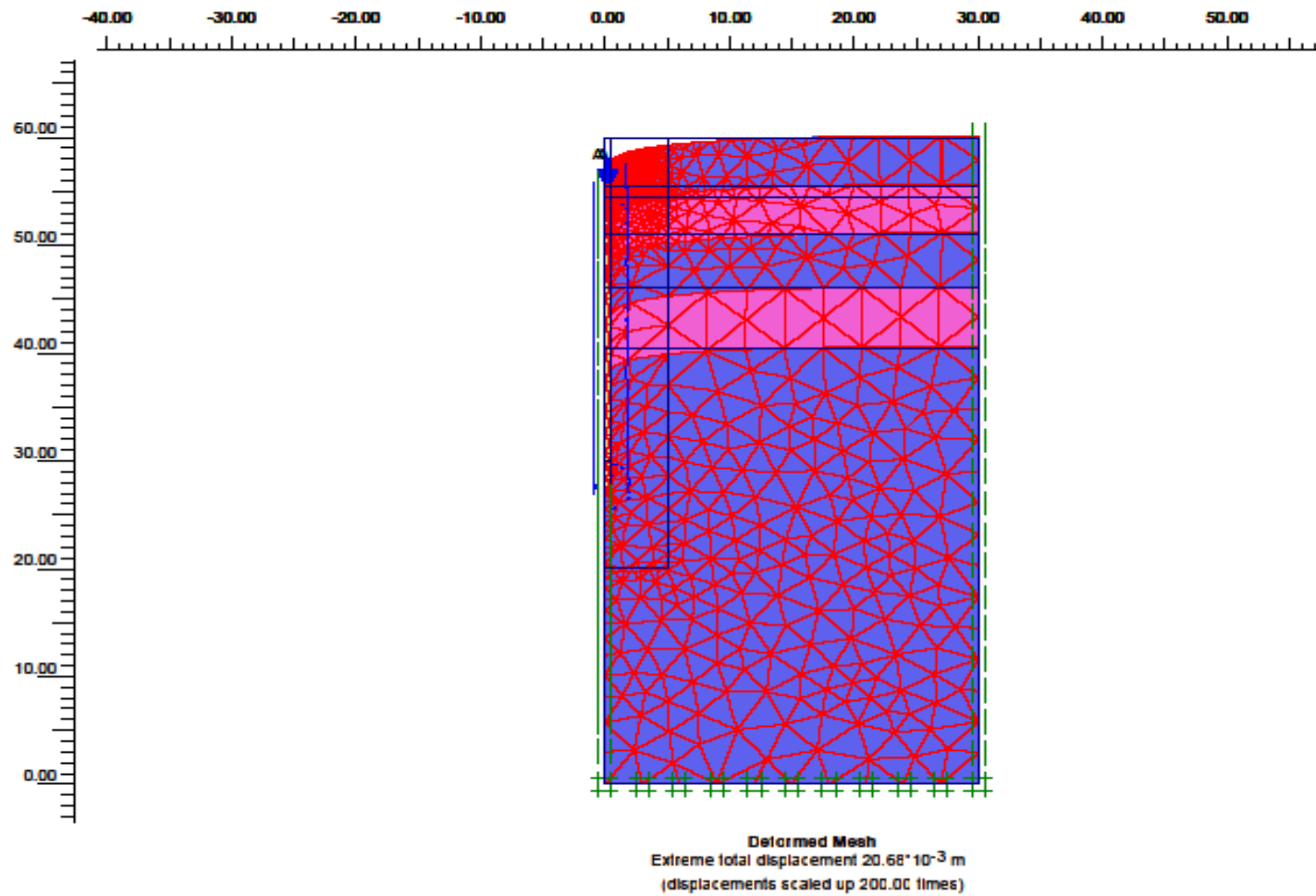


Figure D.2. Deformed mesh.

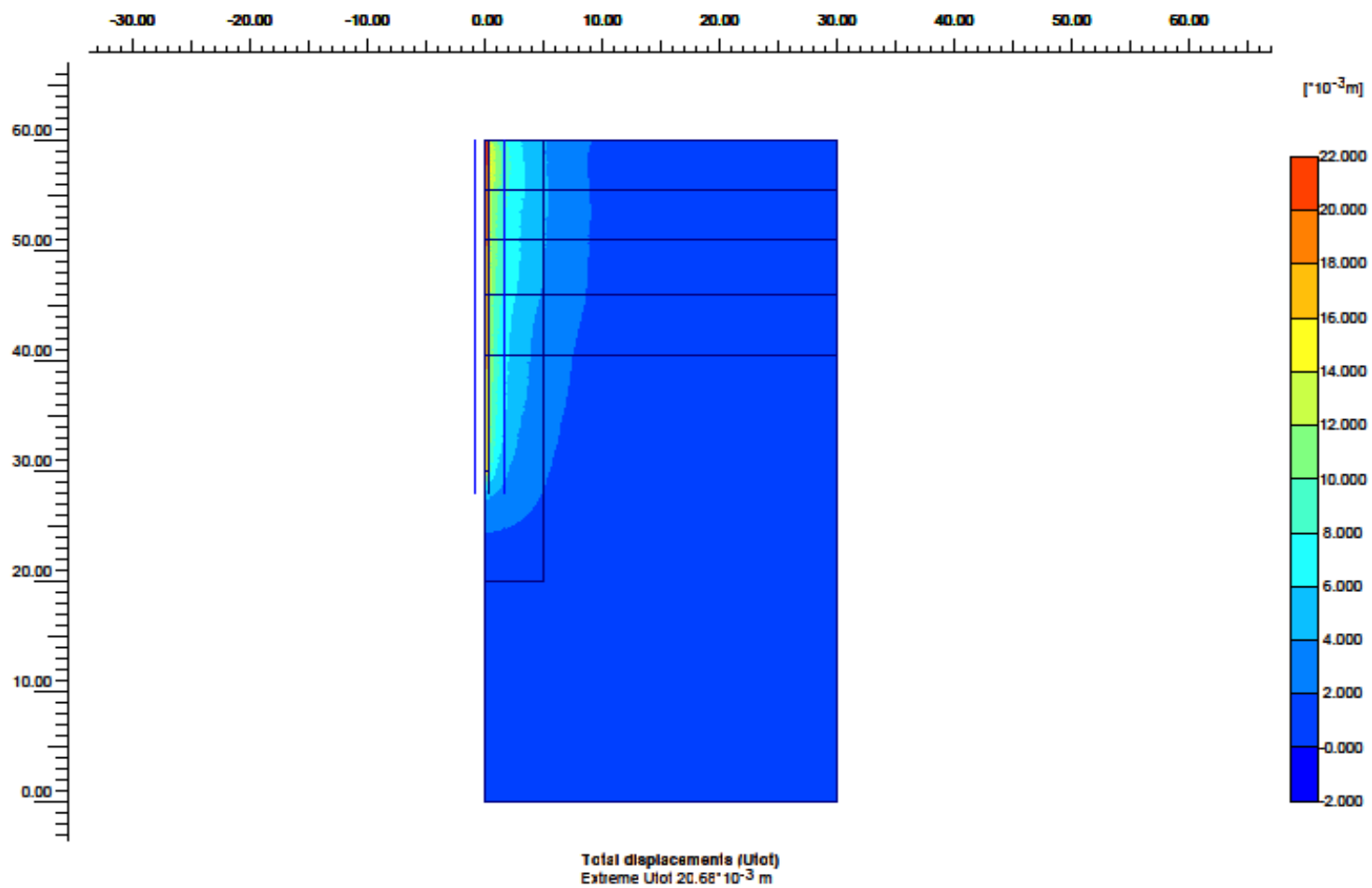


Figure D.3. Total displacements.

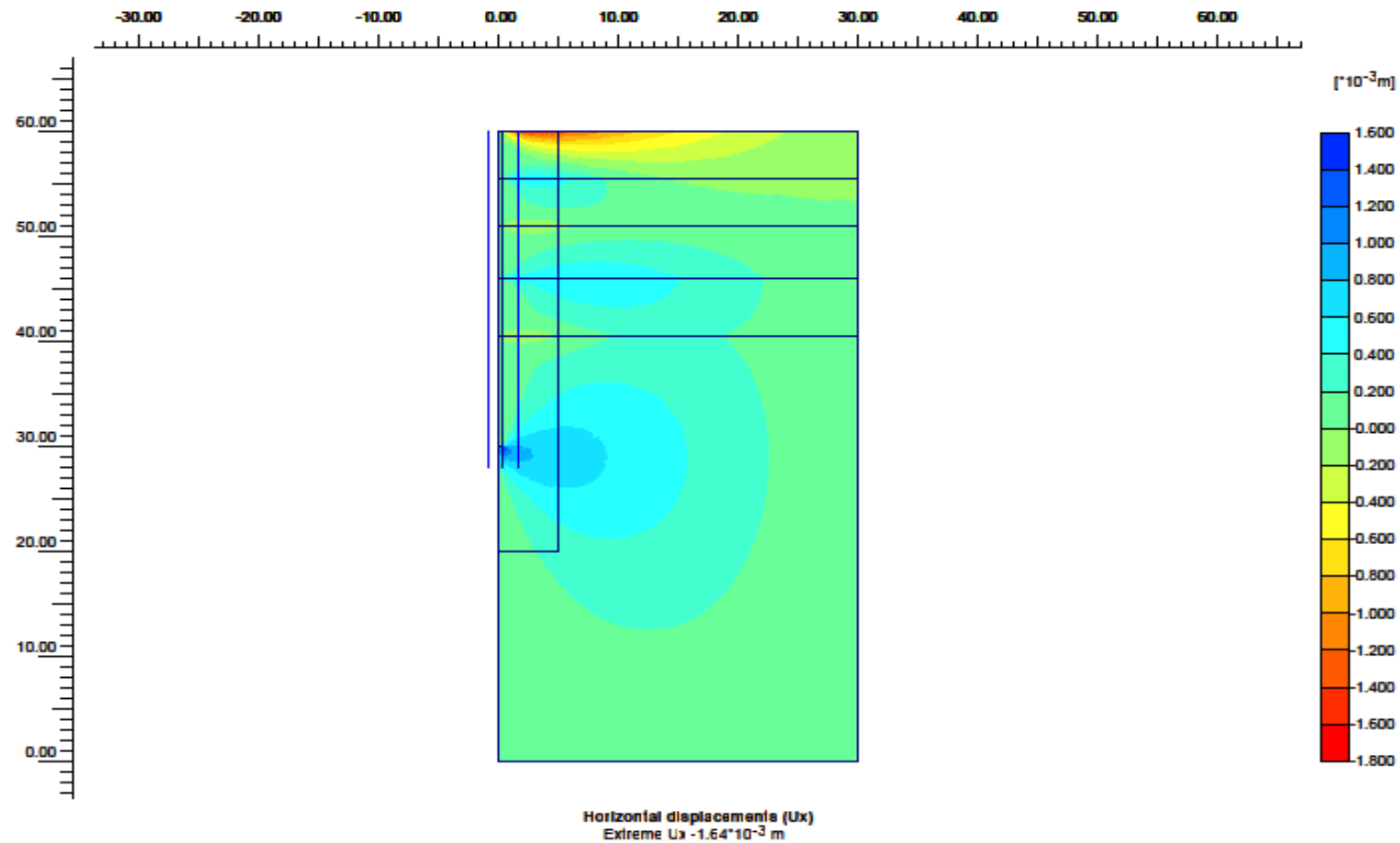


Figure D.4. Horizontal displacements.

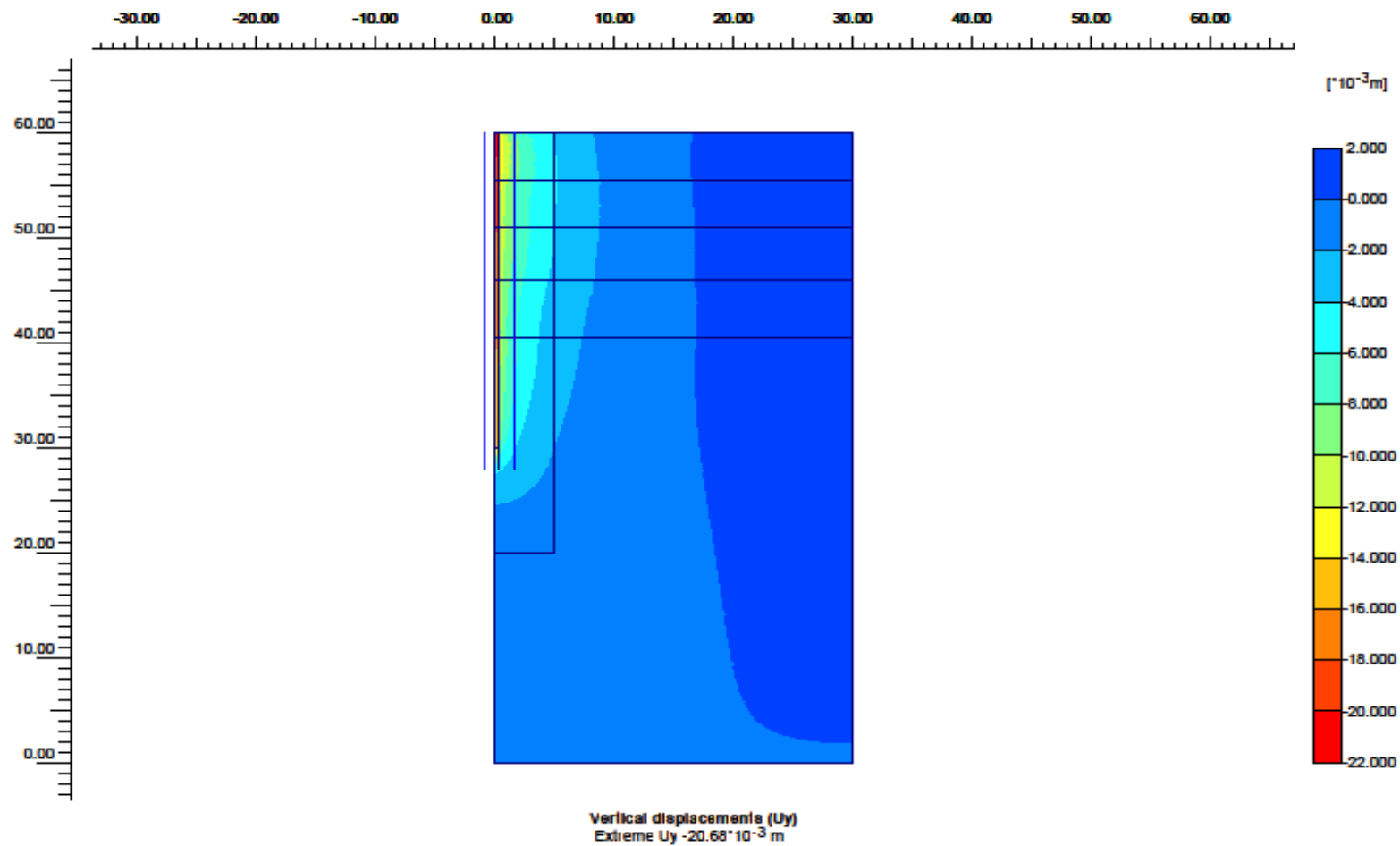


Figure D.5. Vertical displacements.

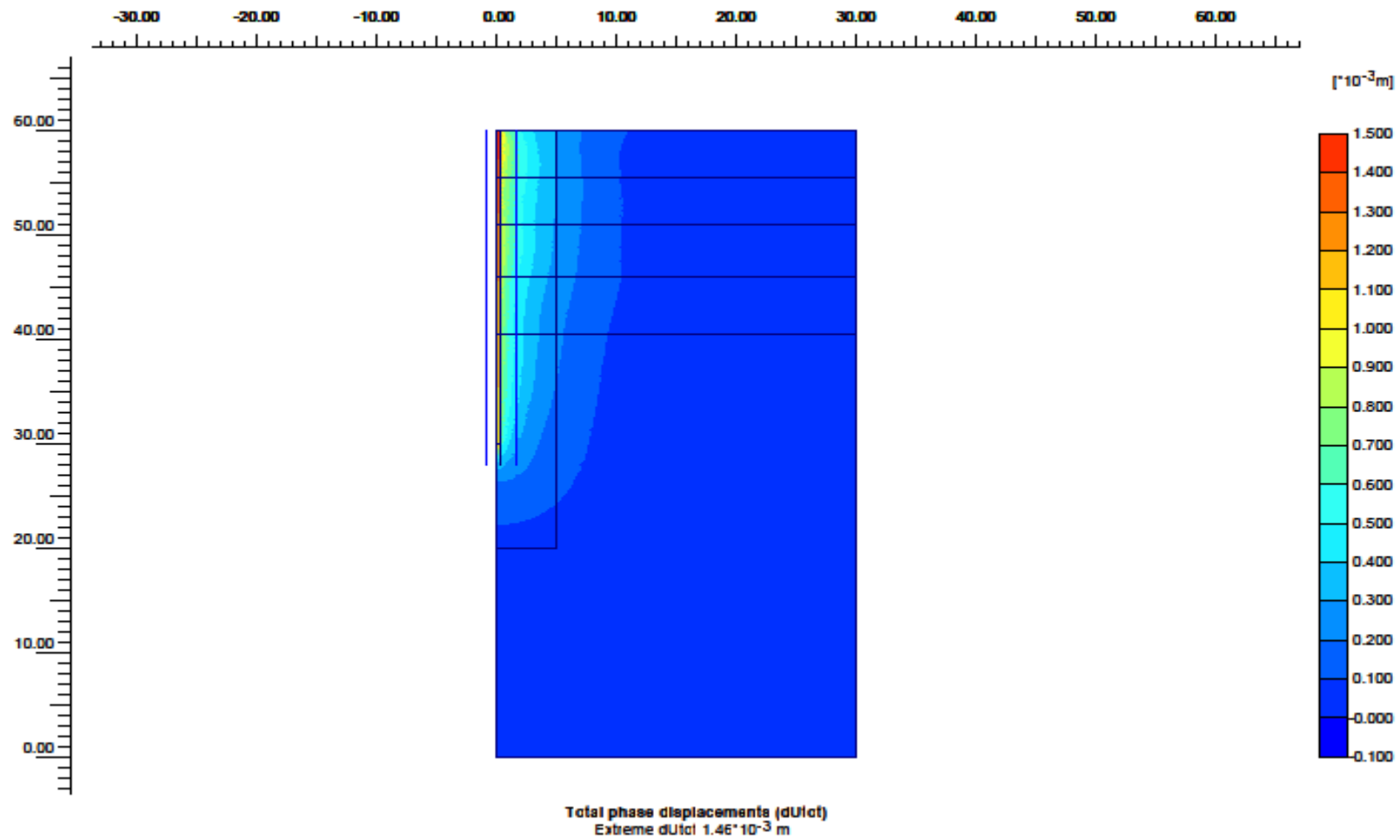


Figure D.6. Total phase displacements.

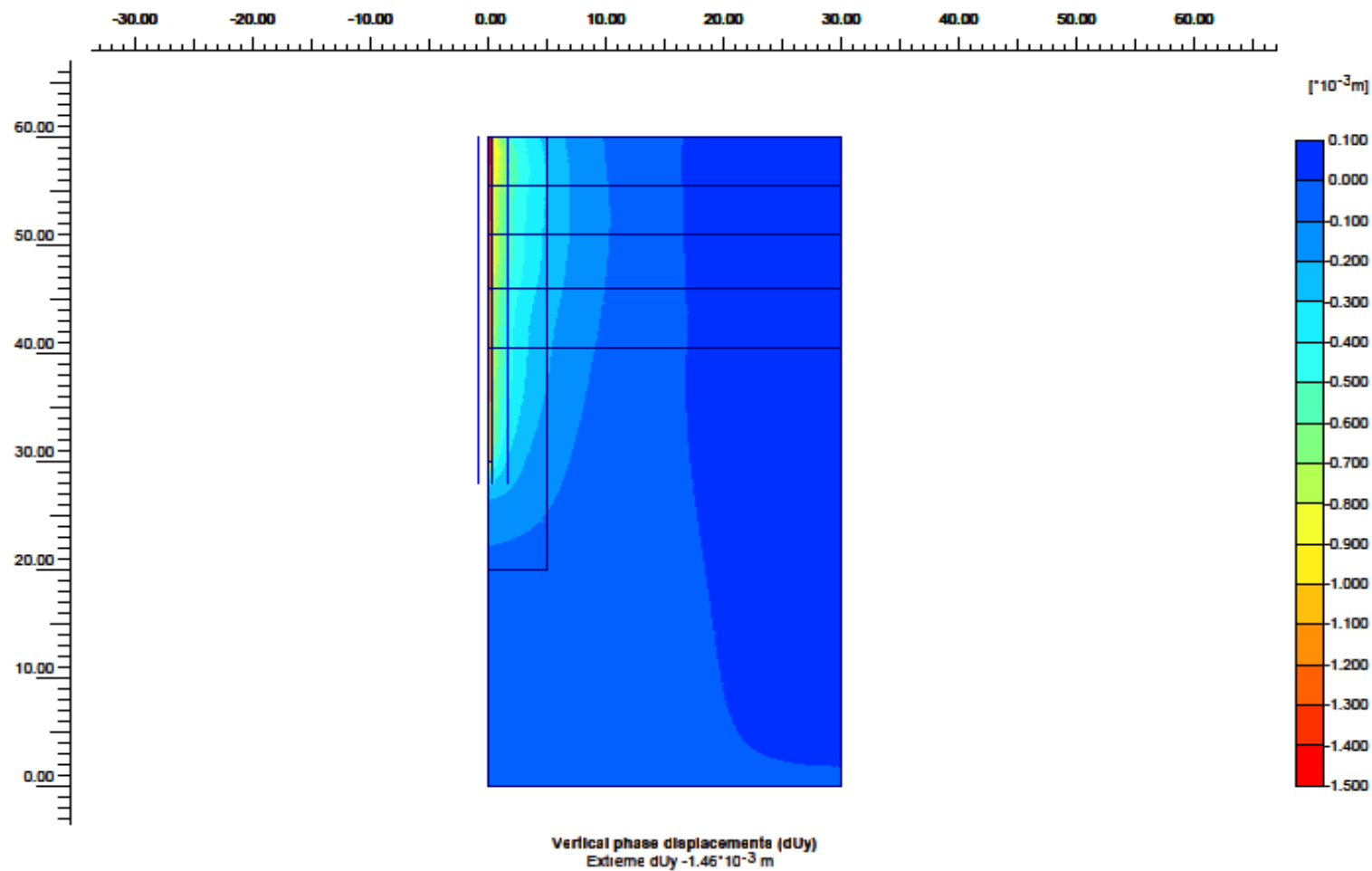


Figure D.7. Vertical phase displacements.

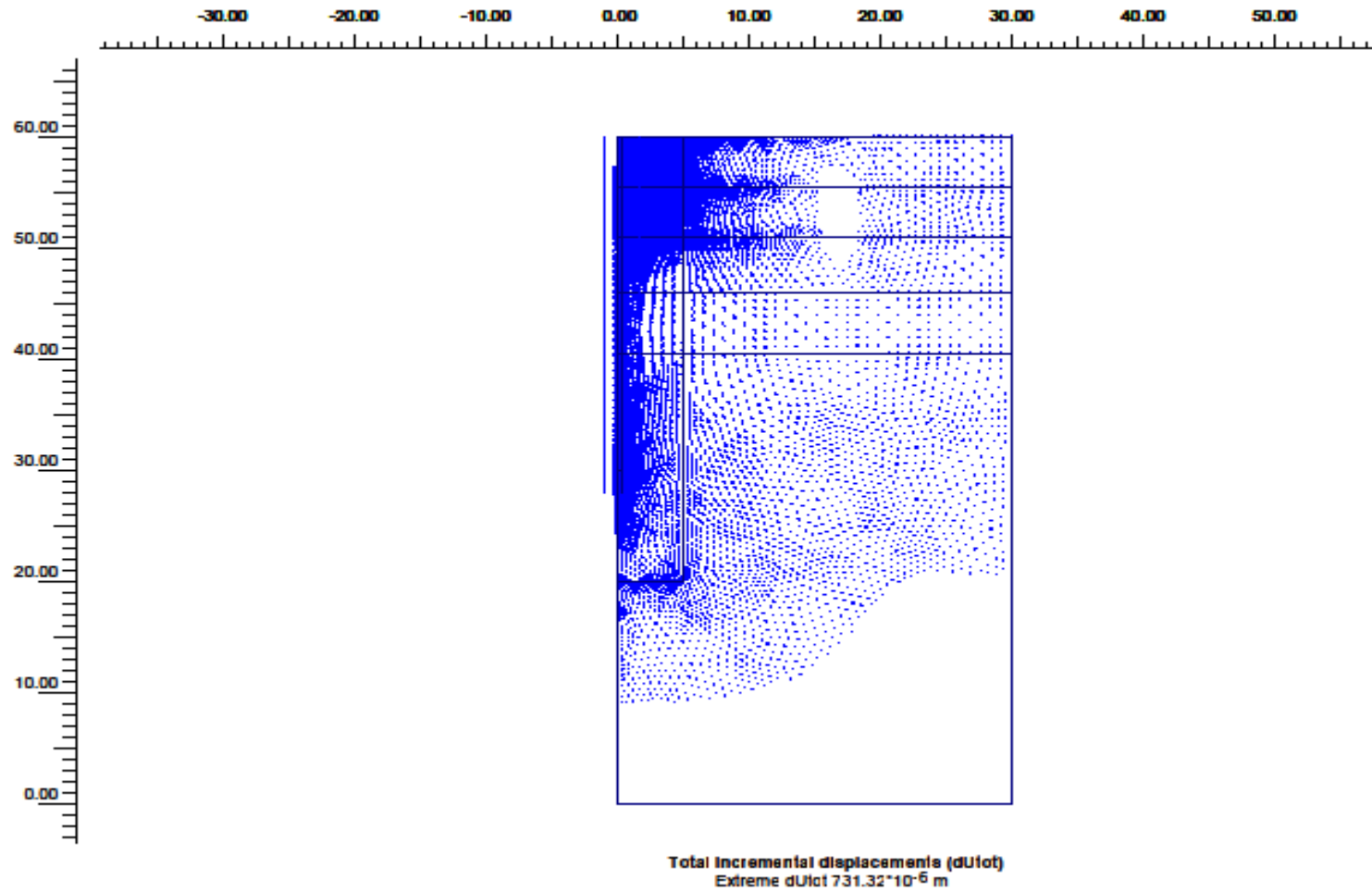


Figure D. 8. Total incremental displacements.

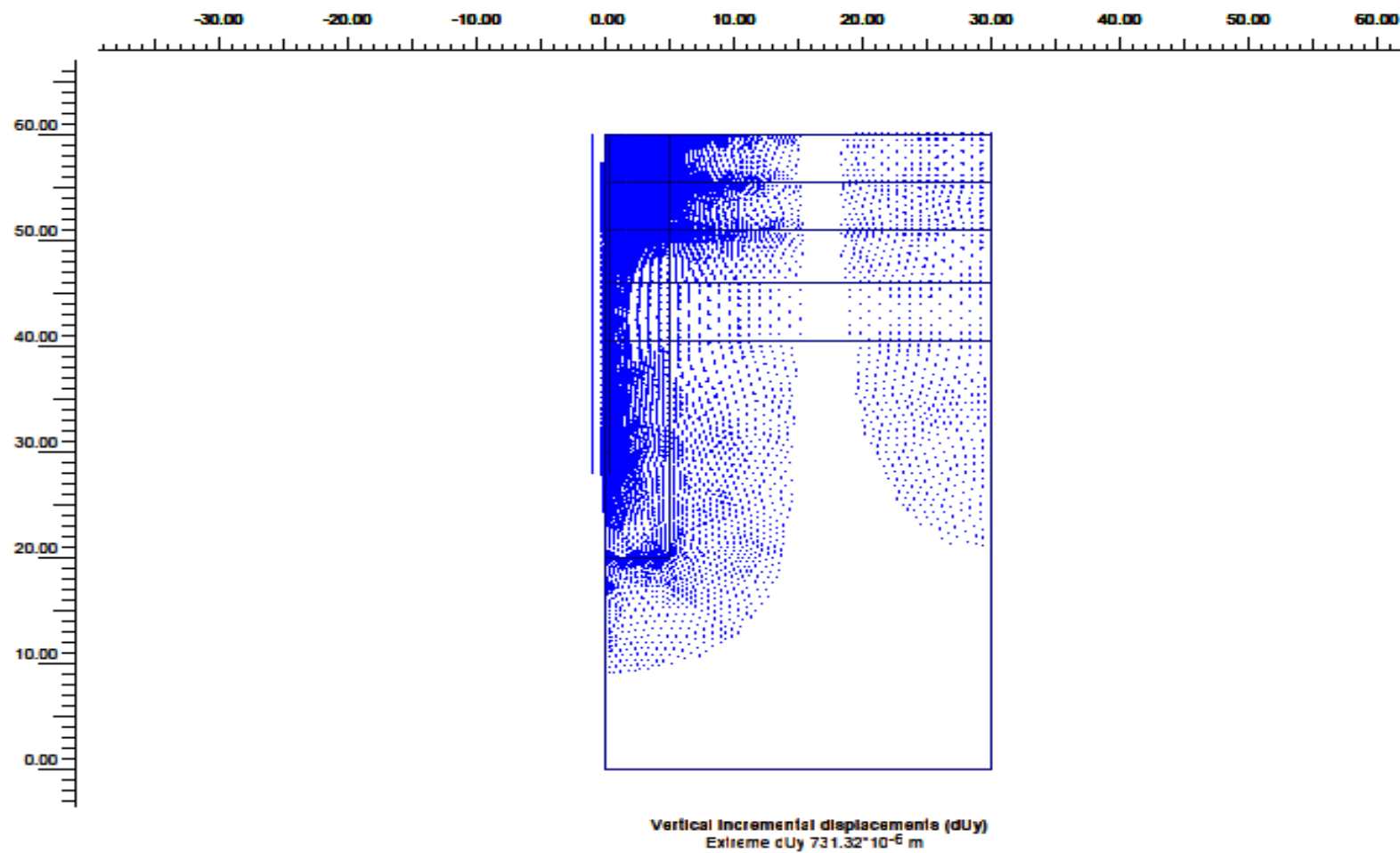


Figure D.9. Vertical incremental displacements.

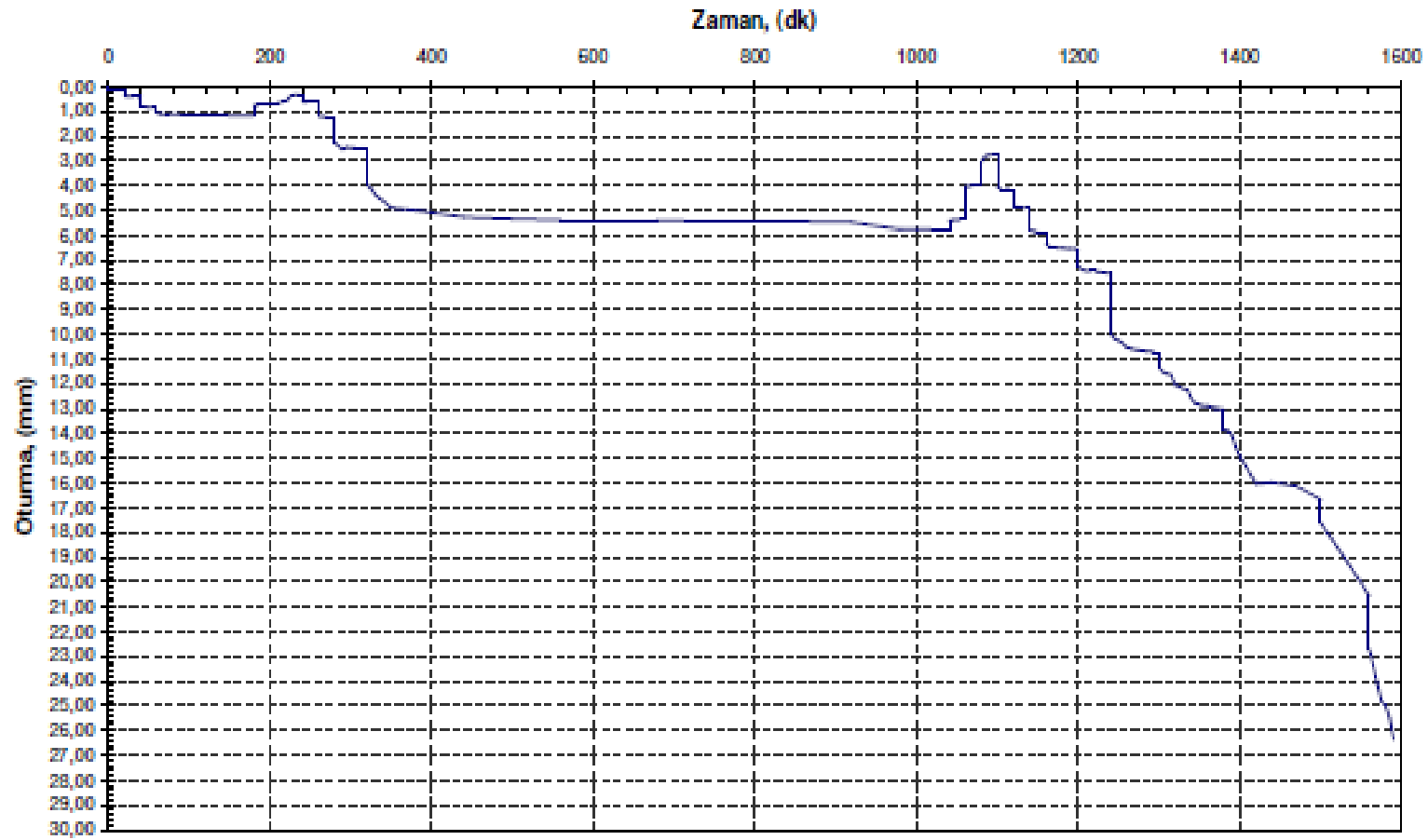


Figure D.10. Time versus Displacement.

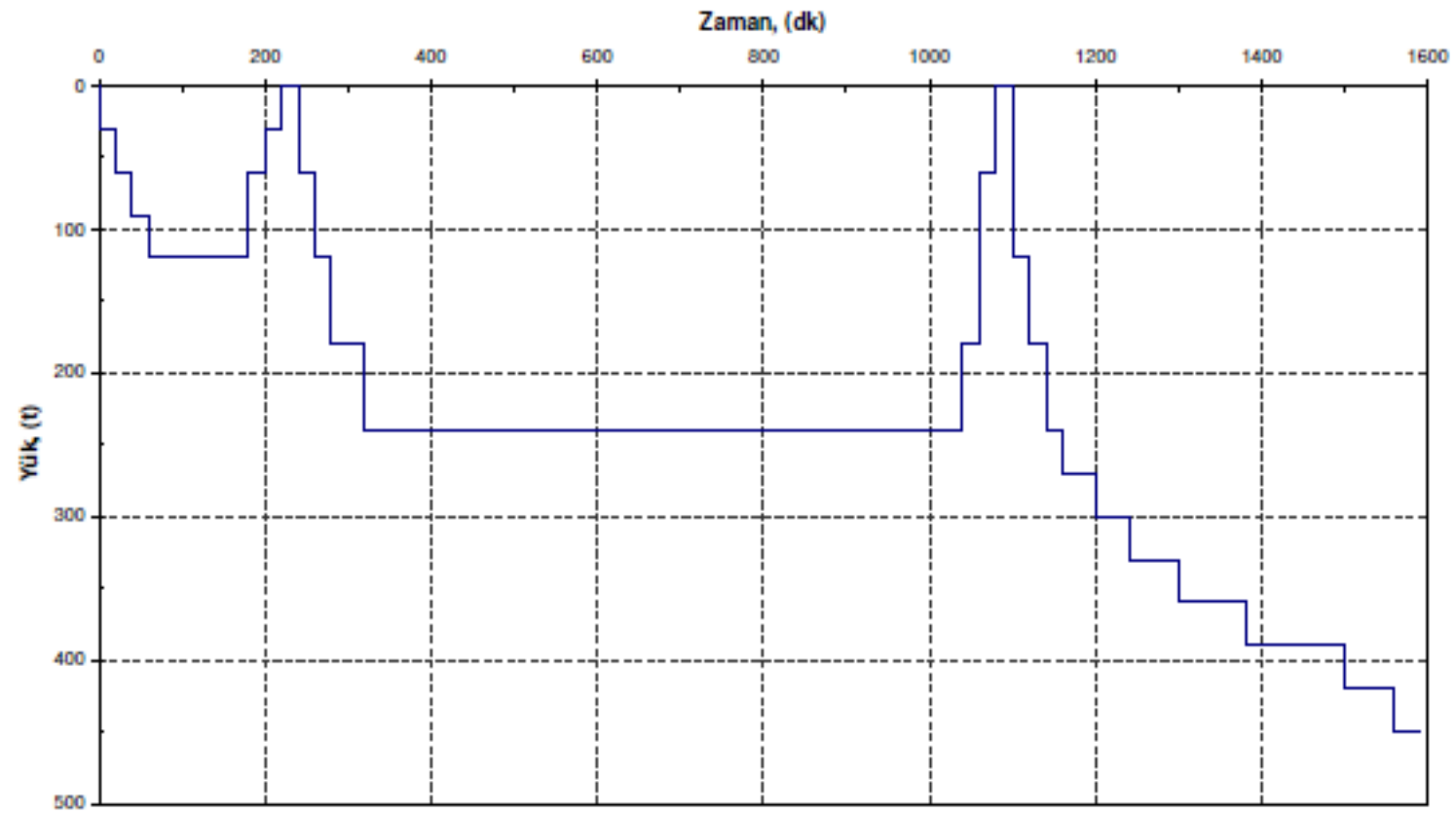


Figure D.11. Time versus Load.

APPENDIX E

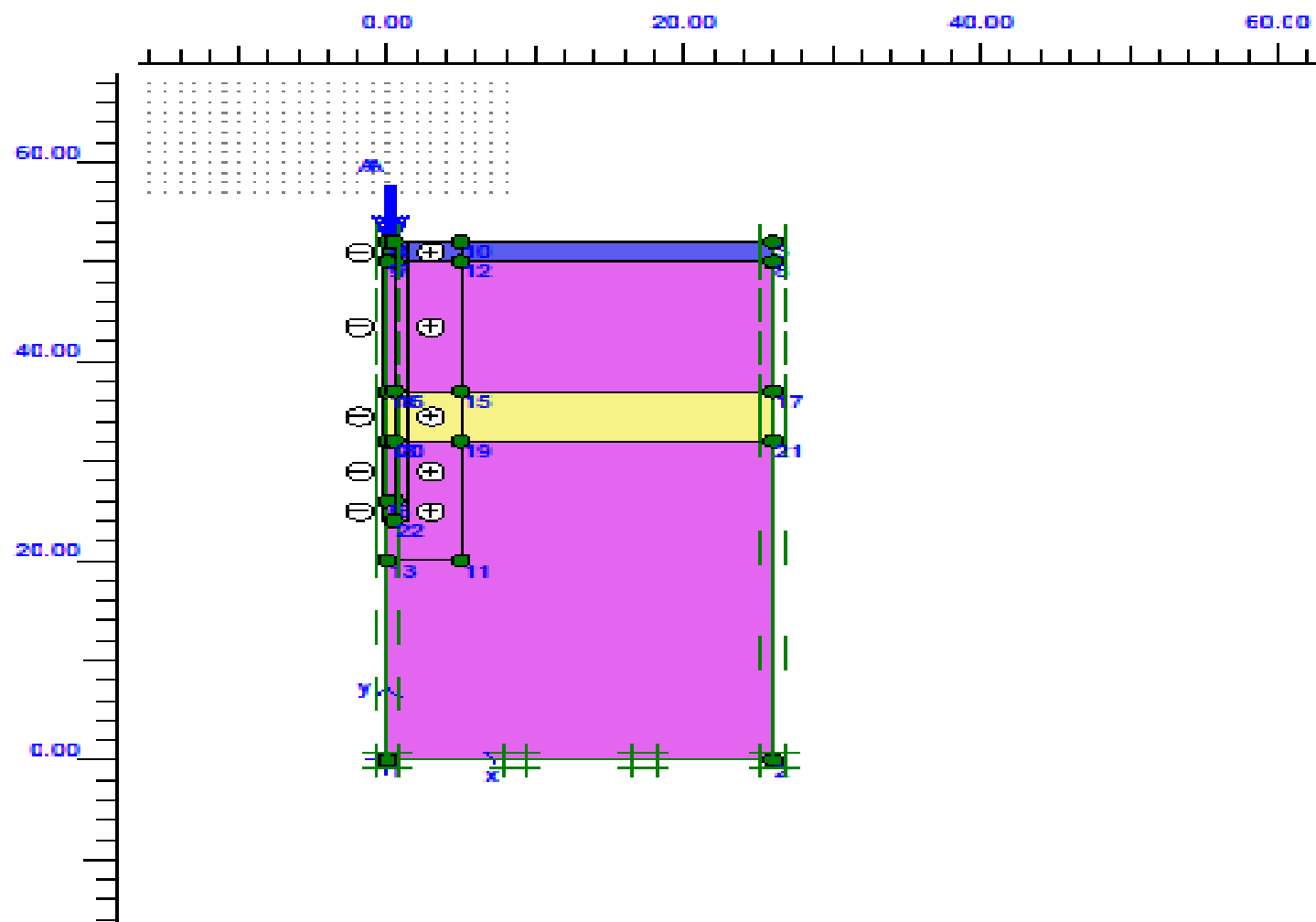


Figure E.1. Finite element model.

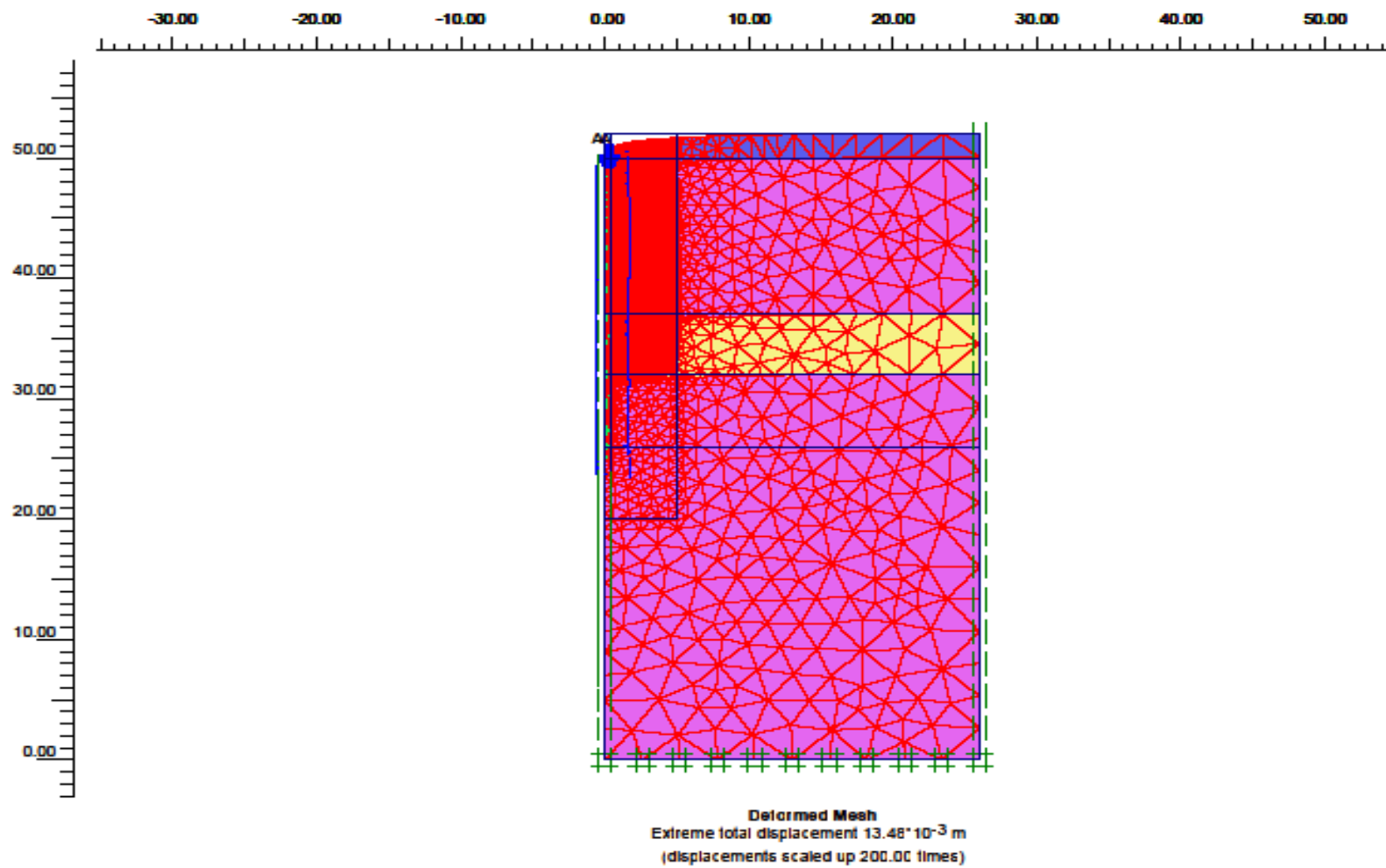


Figure E.2. Deformed mesh.

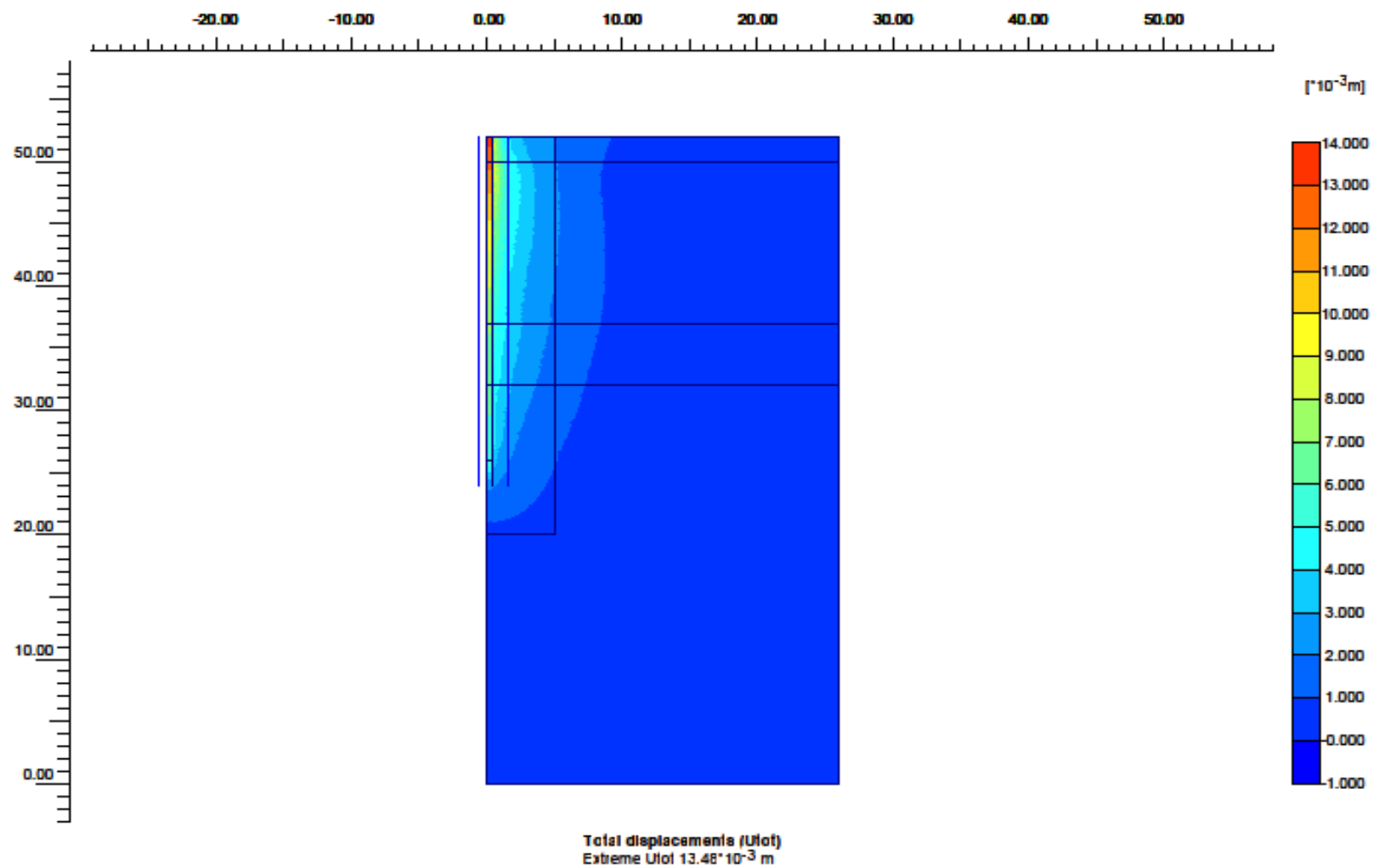


Figure E.3. Total displacements.

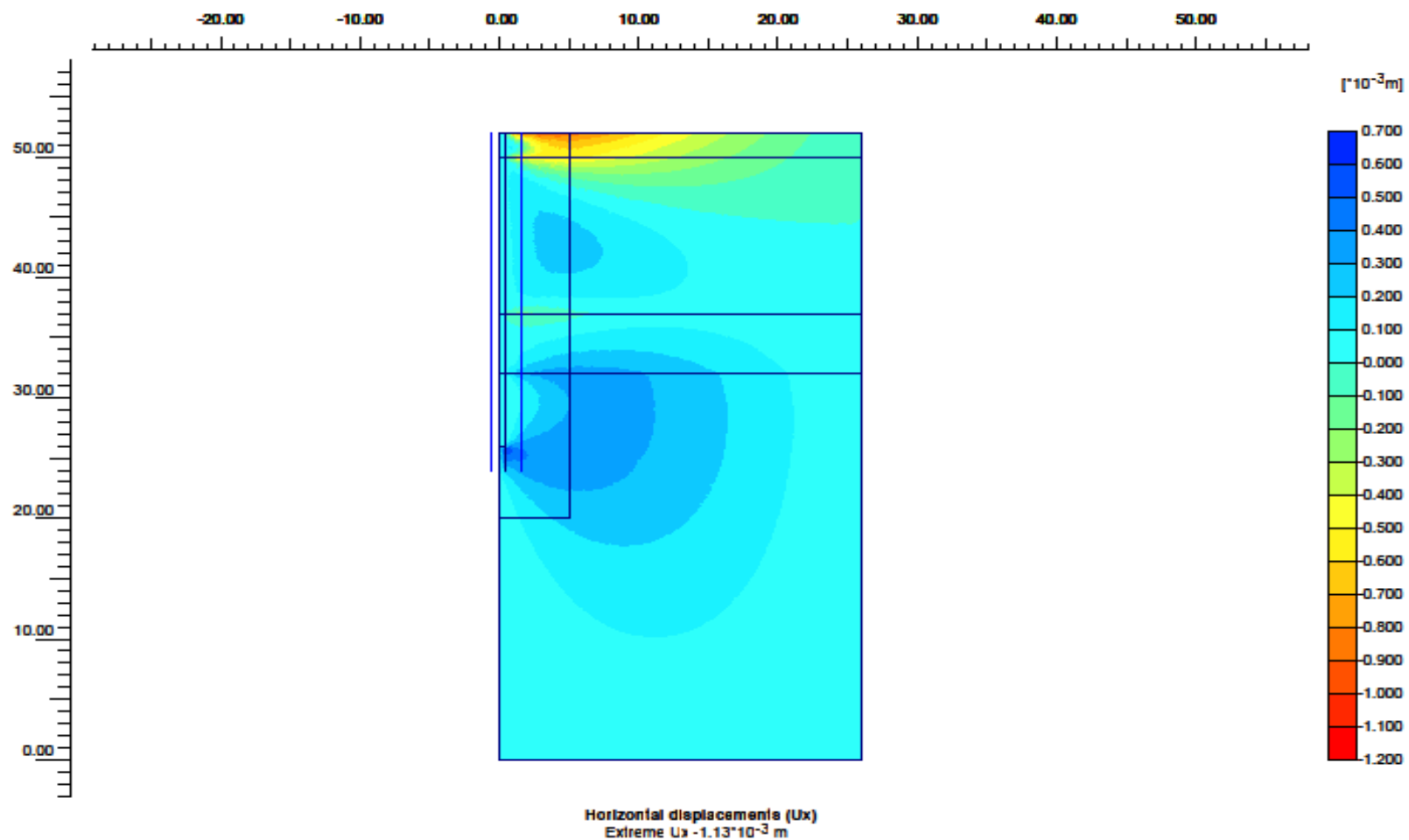


Figure E.4. Horizontal displacements.

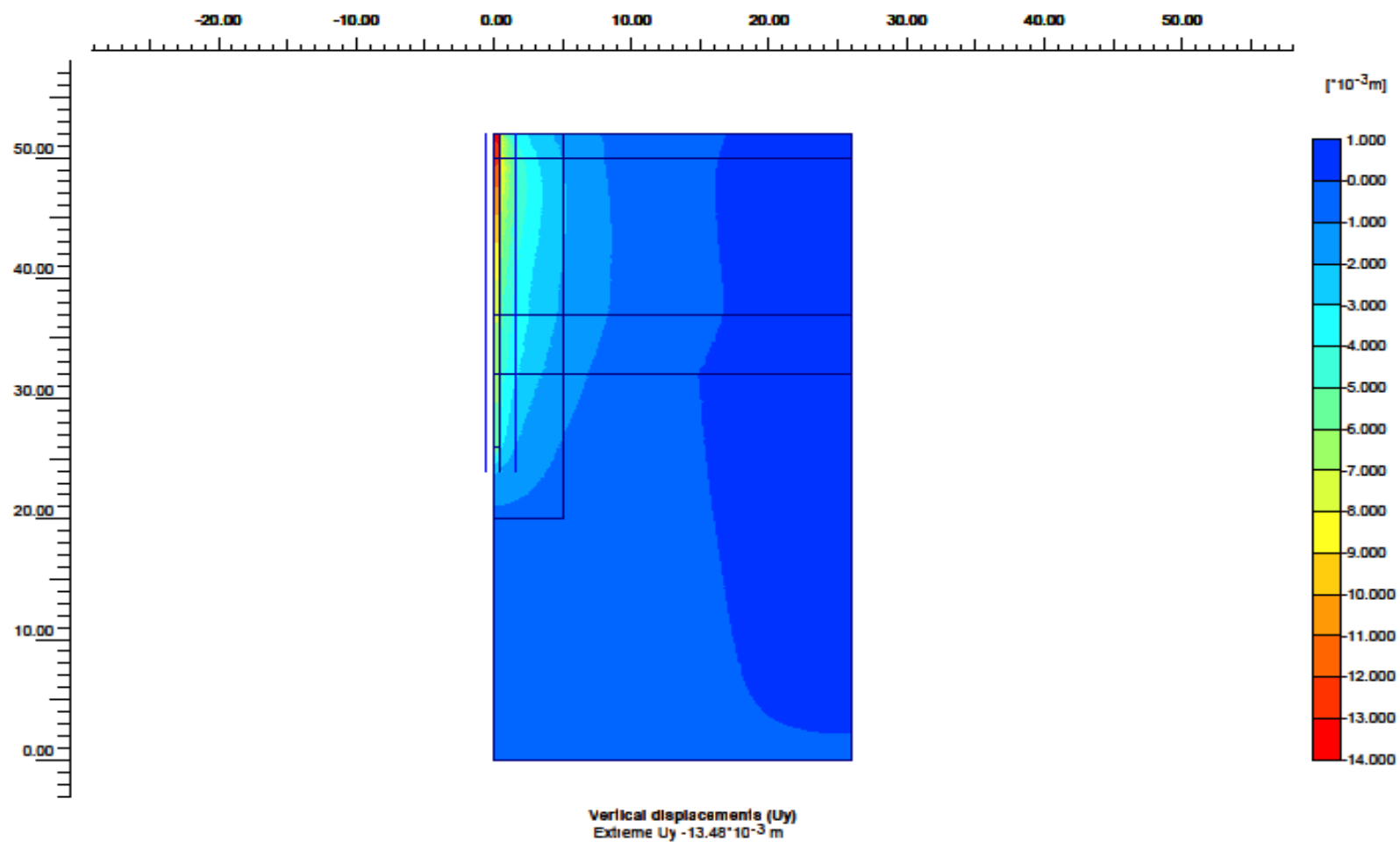


Figure E.5. Vertical displacements.

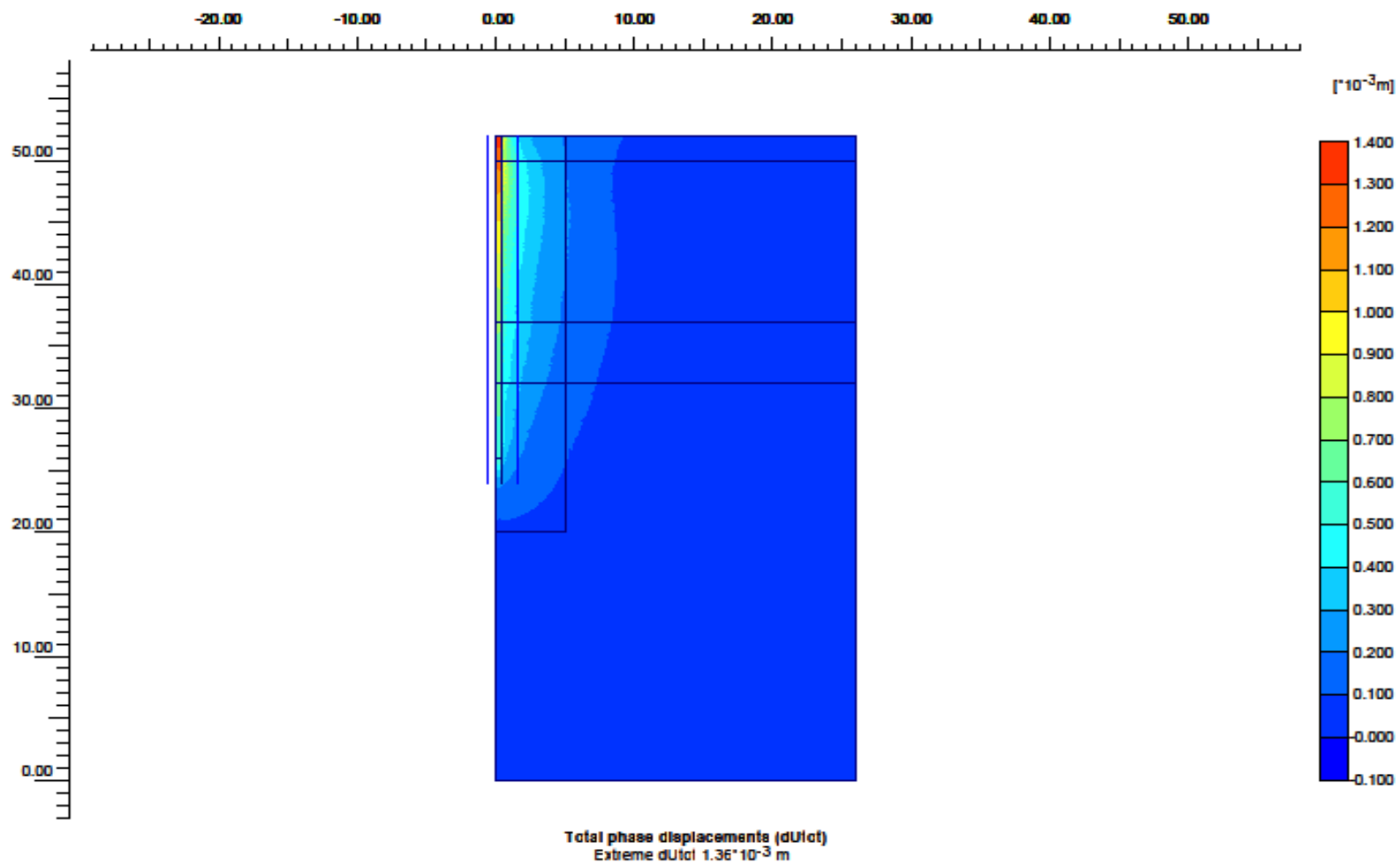


Figure E.6. Total phase displacements.

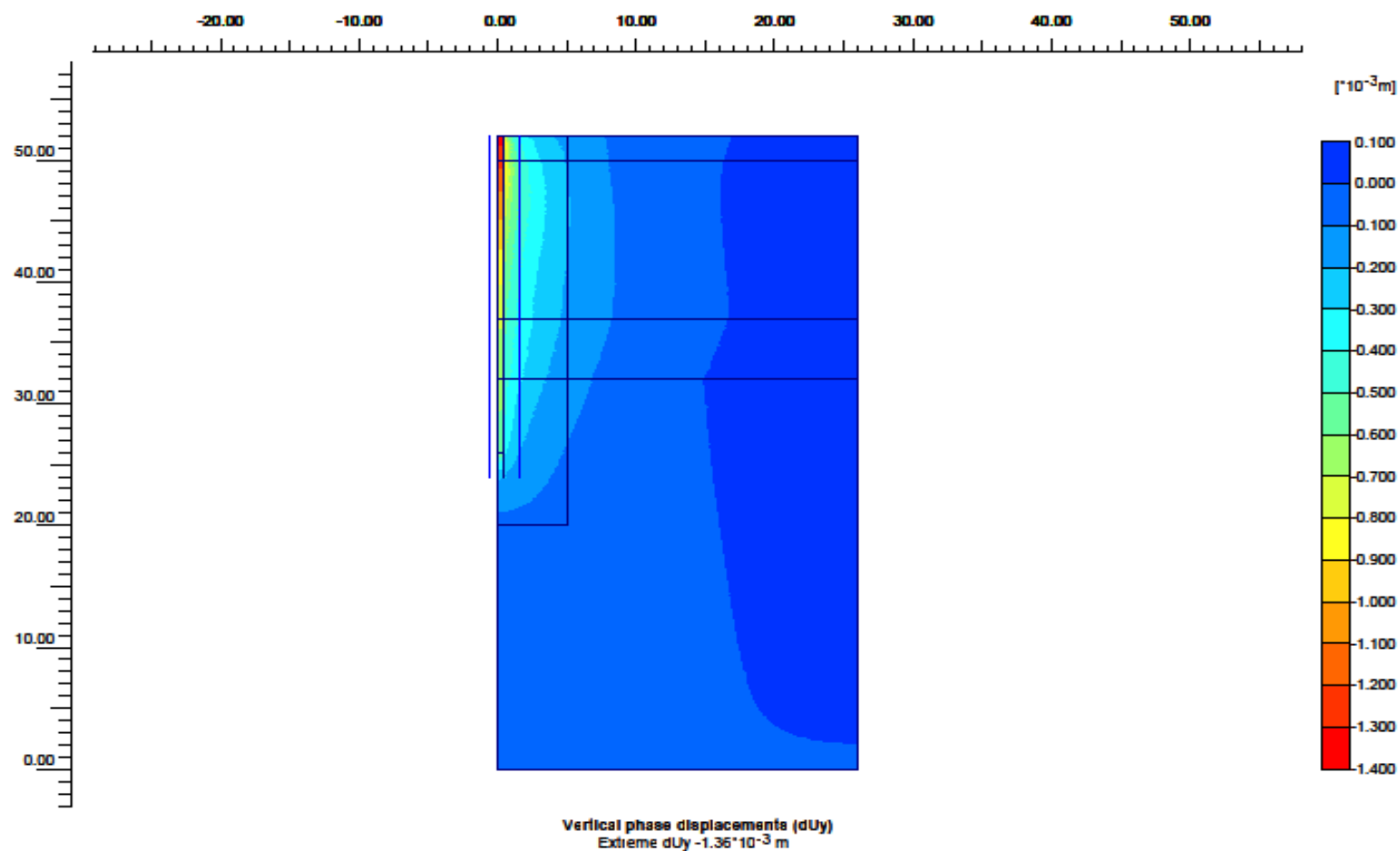


Figure E.7. Vertical phase displacements.

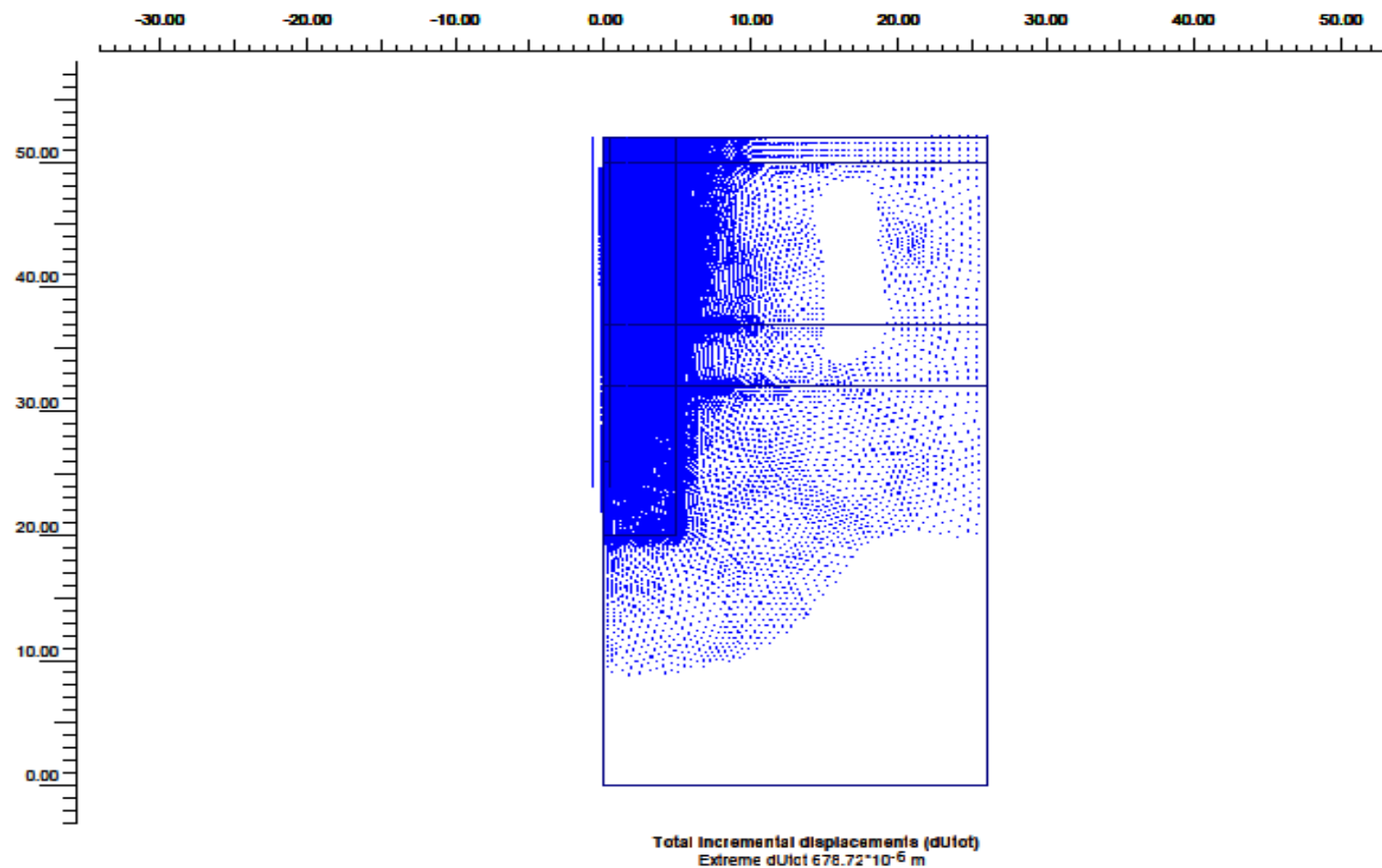


Figure E.8. Total incremental displacements.

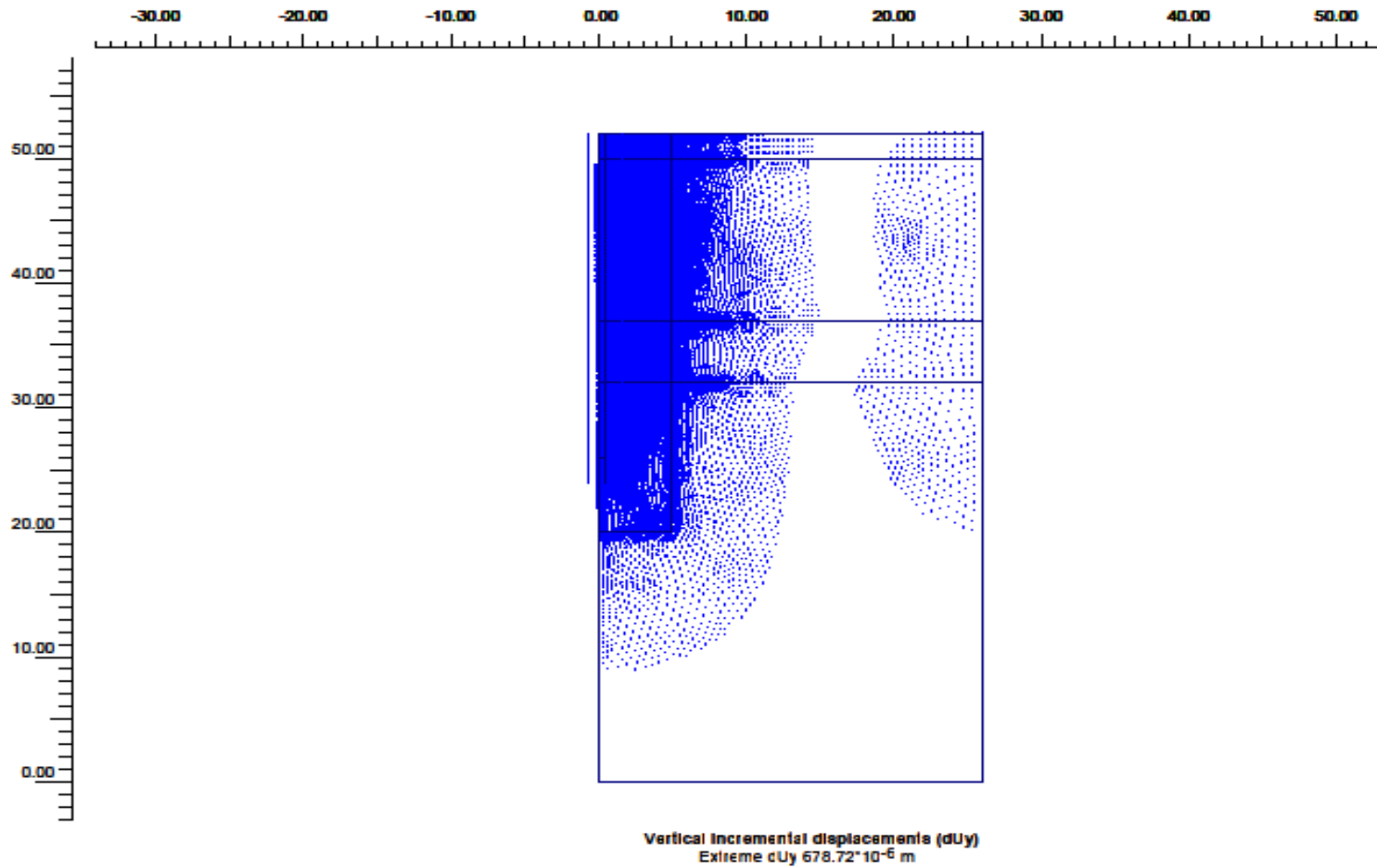


Figure E.9. Vertical incremental displacements.

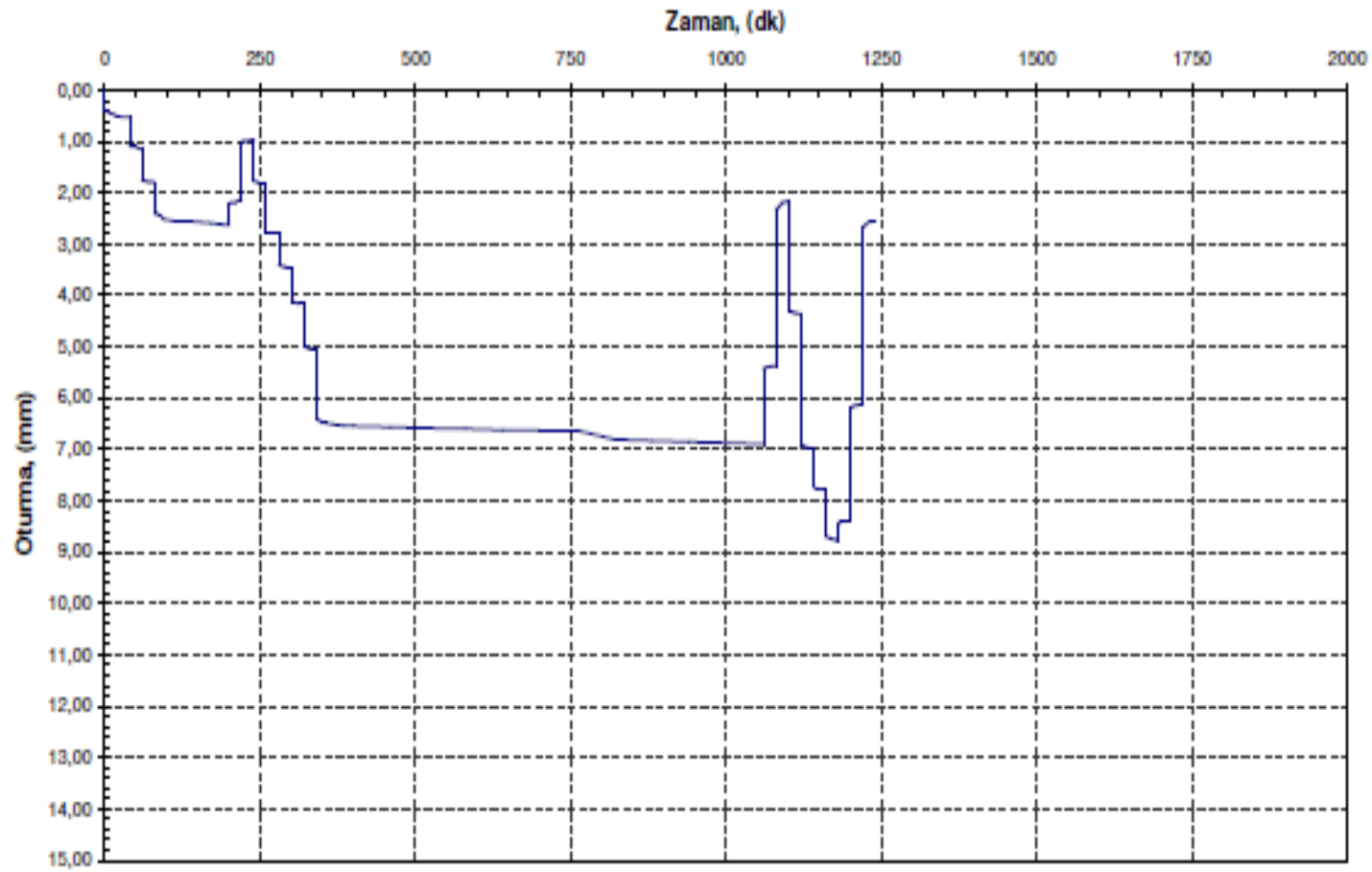


Figure E.10. Time versus Displacement.

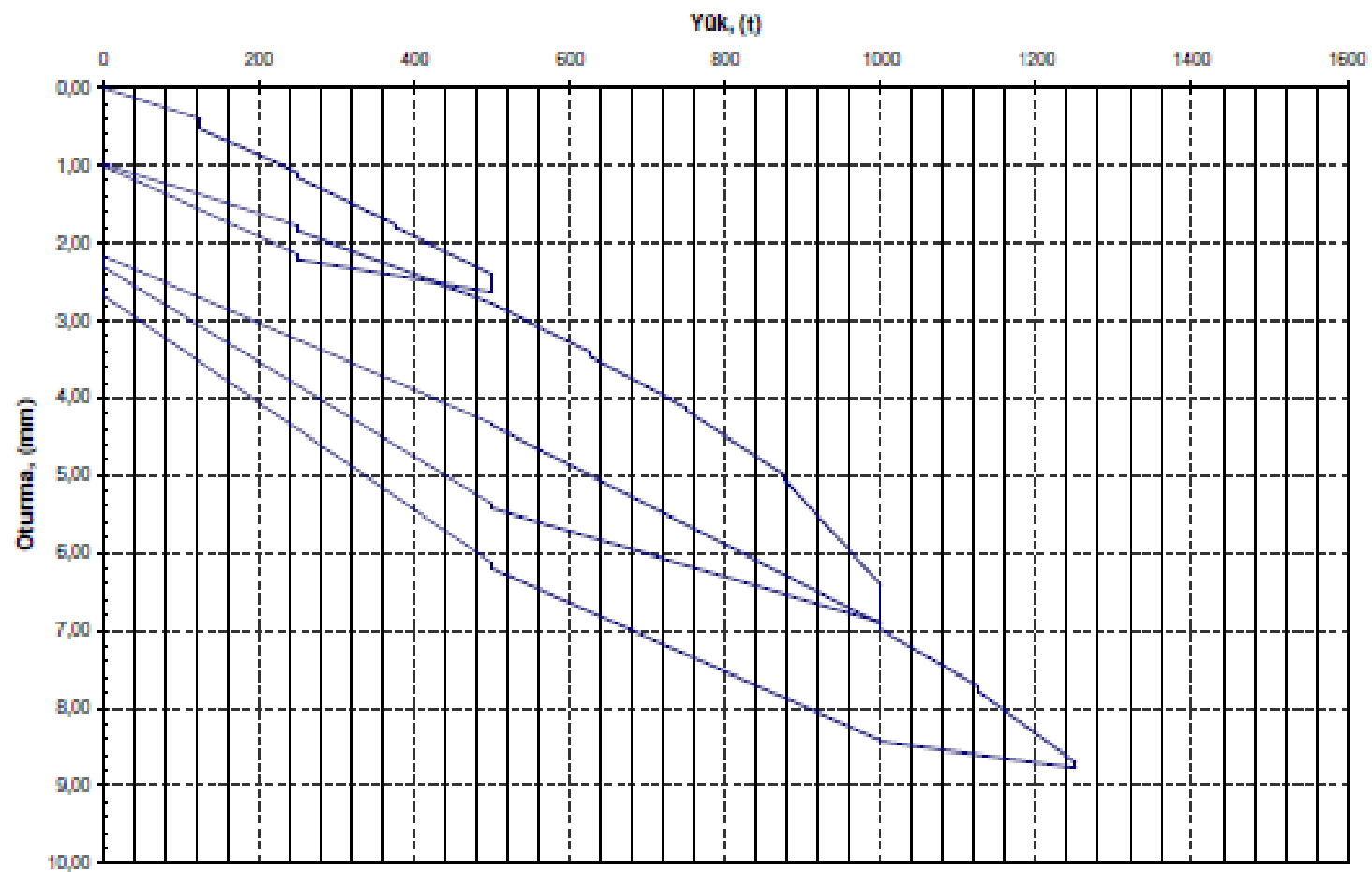


Figure E.11. Load versus Displacement.

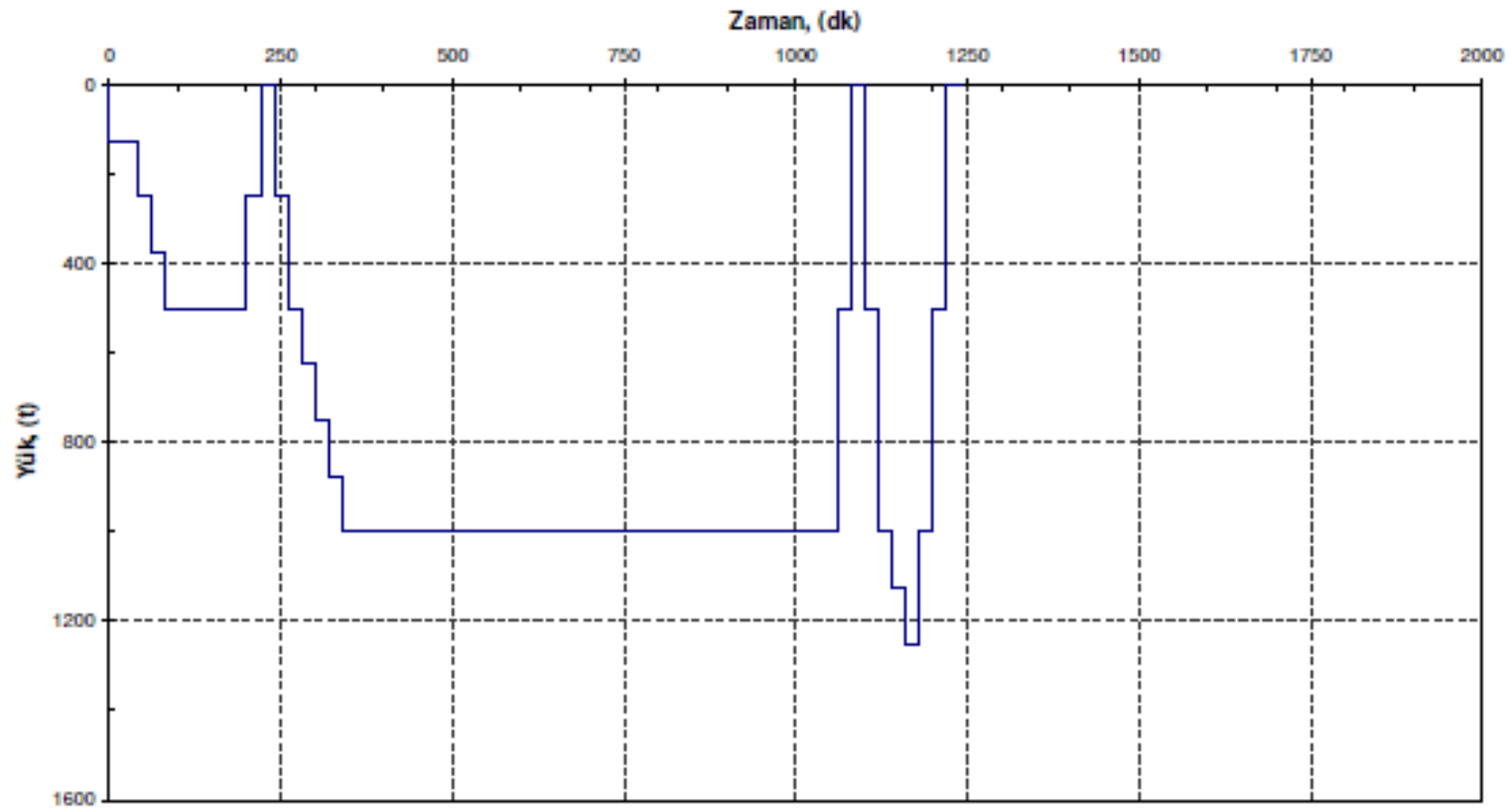


Figure E.12. Time versus Load.

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