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Application of Submerged Grouted Anchors in Sheet Pile Quay Walls

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ABSTRACT

Sheet pile walls are one of the oldest earth retaining structures constructed to retain earth, water or any other fill material in civil engineering projects. They are used in a wide variety of both temporary and permanent building applications, including excavation support system, cofferdams, cut-off walls under dams, slope stabilization, waterfront structures, and flood walls. Sheet pile walls are one of the most common types of quay walls used in port construction. The worldwide increases in utilization of large ships for transportation have created an urgent need of deepening the seabed within port areas and consequently the rehabilitation of its wharfs. Several methods can be used to increase the load-carrying capacity of sheet-piling walls. The use of additional anchored tie rods grouted into the backfill soil and arranged along the exposed wall height is one of the most practical and appropriate solutions adopted for stabilization and rehabilitation of the existing quay wall. The Ravenna Port Authority initiated a project to deepen the harbor bottom at selected wharves. An extensive parametric study through the finite element program, PLAXIS 2D, version 2012 was carried out to investigate the enhancement of using submerged grouted anchors technique on the load response of sheet-piling quay wall. The influence of grout-ties area, length of grouted body, anchor inclination and anchor location were considered and evaluated due to the effect of different system parameters. Also a comparative study was conducted by Plaxis 2D and 3D program to investigate the behavior of these sheet pile quay walls in terms of horizontal displacements induced along the sheet pile wall and ground surface settlements as well as the anchor force and calculated factor of safety. Finally, a comprehensive study was carried out by using different constitutive models to simulate the mechanical behavior of the soil to investigate the effect of these two models (Mohr-Coulomb and Hardening Soil) on the behavior of these sheet pile quay walls.

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CHAPTER 1
INTRODUCTION

1.1 INTRODUCTION

Anchor piles and sheet piles and ground anchors have been used in the construction industry since early last century and a considerable amount of technical study have been performed and these studies revealed significant technical knowledge and construction expertise. They are used in a wide variety of both temporary and permanent building applications, including retaining walls, slope stabilization, seepage control barriers, river and canal frontages, quays, wharves, sea walls, dock and harbor works, permanent foundations and ground reclamation works. The digging of an excavation in the ground causes stress changes in the ground. These stress changes indicates a variety in the stress distribution particularly around the excavation. These stress changes caused by the roof and wall pressures due to the excavation bring out the displacements around the excavation which can cause the deformations and loosening of soil especially on the surface of the slope, retaining walls and cliff walls. One of the most important cases is to control these displacements and deformations with the help of some excavation supports. The greatest use of prestressed anchors with piles or sheet piles is in the support of both temporary and permanent excavations.

The main purpose in an excavation is to help rock to support itself. The subject of piling and ground anchoring can be considered in some details and one of the important subjects is the displacement of the anchor piles due to the excavation and by applying anchorage the deformations and soil movements can be kept under control. Load-deformation behavior of anchor piles and sheet piles and anchors is a straightforward topic to discuss but also a general look to examine in detail some various factors. The performance of an anchored structure depends on how the anchor develops load.

There are also some other ground exploration and site investigations are needed which are required before designing the anchored structure. The general geology of the site and the topographical features affect the design and construction. Details of the various soil and rock strata and ground water tables may affect the anchorage during construction. By means of some laboratory tests on the soil and rock samples, in-situ tests, soil and rock mechanics investigations will also help to select the proper anchorage.

Today a large number of stabilization methods are available. Within this study the behavior of the anchor sheet pile wall is investigated and a FEM analysis is carried out. The main objective is to investigate the anchored sheet pile wall behaviors and also to compare the behavior of these sheet piles predicted by 2D and 3D models. Finally, the behavior of anchored sheet pile walls investigated by different constitutive models. Geological and design parameters and considerations, the observed anchored sheet pile wall behaviors, anchor prestress results are obtained from the design and the study is performed by constituting a theoretical model and this model is incorporated into a Windows based program called Plaxis. PLAXIS is a robust simulator of geotechnical problems which works on the basis of finite element method. This software is capable of solving a wide range of problems, from simple linear analysis to highly complex nonlinear simulation, particularly, through considering the effect of soil-structure interaction.

1.2 STATEMENT OF THE PROBLEM

When the ground is excavated the main matter is the stabilization or in some cases the rehabilitation of the walls around the opening in case of the stability of the

superstructure and the other structures which are constructed before. (e.g. buildings next to the cliff walls, motorways above a tunnel, terminals, etc.)
There are also some natural effects which can always present instability problems and have to be considered before the stabilization study. The most important considerations are quantification of the ground material, particularly joints and fissures, understanding of the water pressures, weathered or unweathered rock conditions, landslide and earthquake conditions, etc. A detailed geological study is also required to figure out these parameters and to make a decision about the stabilization method of the ground. When the effect of the stabilization is considered the study about the stabilization method becomes more important to determine the optimum design, construction and cost studies.
There are many stabilization methods available in civil and mining constructions. The most used excavation supports are rock bolts and ground anchorages. There are also many types of bolt and anchor types and during this study some of the anchor types will be mentioned and as pointed out previously the anchor sheet piles will be considered and the behavior of the anchored pile and excavation walls will be investigated in the scope of stabilization and rehabilitation. The study will make progress within the numerical analysis study results.
To determine the parameters of the soil due to the Mohr-Coulomb (MC) and Hardening Soil (HS) criteria during excavation and to study the interaction of these parameters it is very important to estimate the problems which can occur during the excavation or rehabilitation like landslide, slope or failure increase of strain, etc. During the construction of the anchor sheet piles and anchors these parameters have to be studied carefully and one of the most important subjects is the anchor arrangement and spacing of anchors which can cause failures that are mentioned above unless it is designed and constructed properly.

1.3 SCOPE AND OUTLINE OF THE THESIS

Investigation of the stabilization and rehabilitation problems that are mentioned above is possible with the determination of the design parameters and interaction of the parameters which effects the deformations into the ground during the excavation, stabilization and rehabilitation studies.
In this study the displacements and internal forces (bending moments and anchor forces) of the anchored sheet pile wall is investigated which are constructed at the Ravenna Port for the stabilization and rehabilitation of the excavation. The

deformations into the ground, internal forces and the stability of the structure are studied and investigated to bring up a conclusion. The anchorages are applied for the stabilization after the sheet piles are constructed and the constructing of additional grouted anchor rows for rehabilitation of the structure were still on process.
In this study which the anchor type, anchor arrangement, sheet pile designs, geological considerations, field and laboratory studies, are predicted and certain, the main objective is to investigate the anchored sheet pile wall stability and evaluation of the behavior by using the Mohr-Coulomb (MC) and Hardening Soil (HS) constitutive models. A FEM analysis is considered in the manner of the stabilization and rehabilitation of the excavation.

In the first chapter, the problem is defined and a scope of the thesis is described.

Chapter Two gives the background information and some literature survey about the sheet pile walls and their applications and there are also some definitions given in this chapter.

Chapter Three describes the anchorage system and the type of ground anchors and also about the anchored wall system and design concepts for these anchored walls has been discussed.

In Chapter Four a parametric study of rehabilitation of sheet pile quay walls in Ravenna Port with the usage of grouted anchors has been done and the results have been discussed.

Chapter Five gives a comparison of a 2D and 3D model for the rehabilitation of these anchored sheet pile walls.

In Chapter Six the behavior of an anchored sheet pile wall for the stabilization of an excavation has been investigated and the influence of the constitutive models has been evaluated.

Finally, conclusions derived from this study and the recommendations for further studies are provided in Chapter Seven.

CHAPTER 2
SHEET PILE WALLS

2.1 INTRODUCTION

Sheet pile walls are retaining walls constructed to retain earth, water or any other fill material. These walls are thinner in section as compared to masonry walls. (Murthy, 2008) Sheet-pile walls are widely used for both large and small waterfront structures, ranging from small pleasure-boat launching facilities to large dock structures where ocean-going ships can take on or unload cargo. A pier jutting into the harbor, consisting of two rows of sheetpiling to create a space between that is filled with earth and paved, is a common construction.

Sheetpiling is also used for beach erosion protection; for stabilizing ground slopes, particularly for roads; for shoring walls of trenches and other excavations; and for cofferdams. When the wall is under about 3 m in height it is often cantilevered (Fig. 2.1a); however, for larger wall heights it is usually anchored using one or more anchors. The resulting wall is termed an *anchored sheet-pile* wall or *anchored bulkhead*. Several of the more common wall configurations are illustrated in Fig. 2.1. The alternative shown in Fig. 2.1d of using continuous rods for parallel sheet-pile walls may be considerably more economical than driving pile anchorages, even for tie rod lengths of 30 to 40 m (Bowles, 1996).

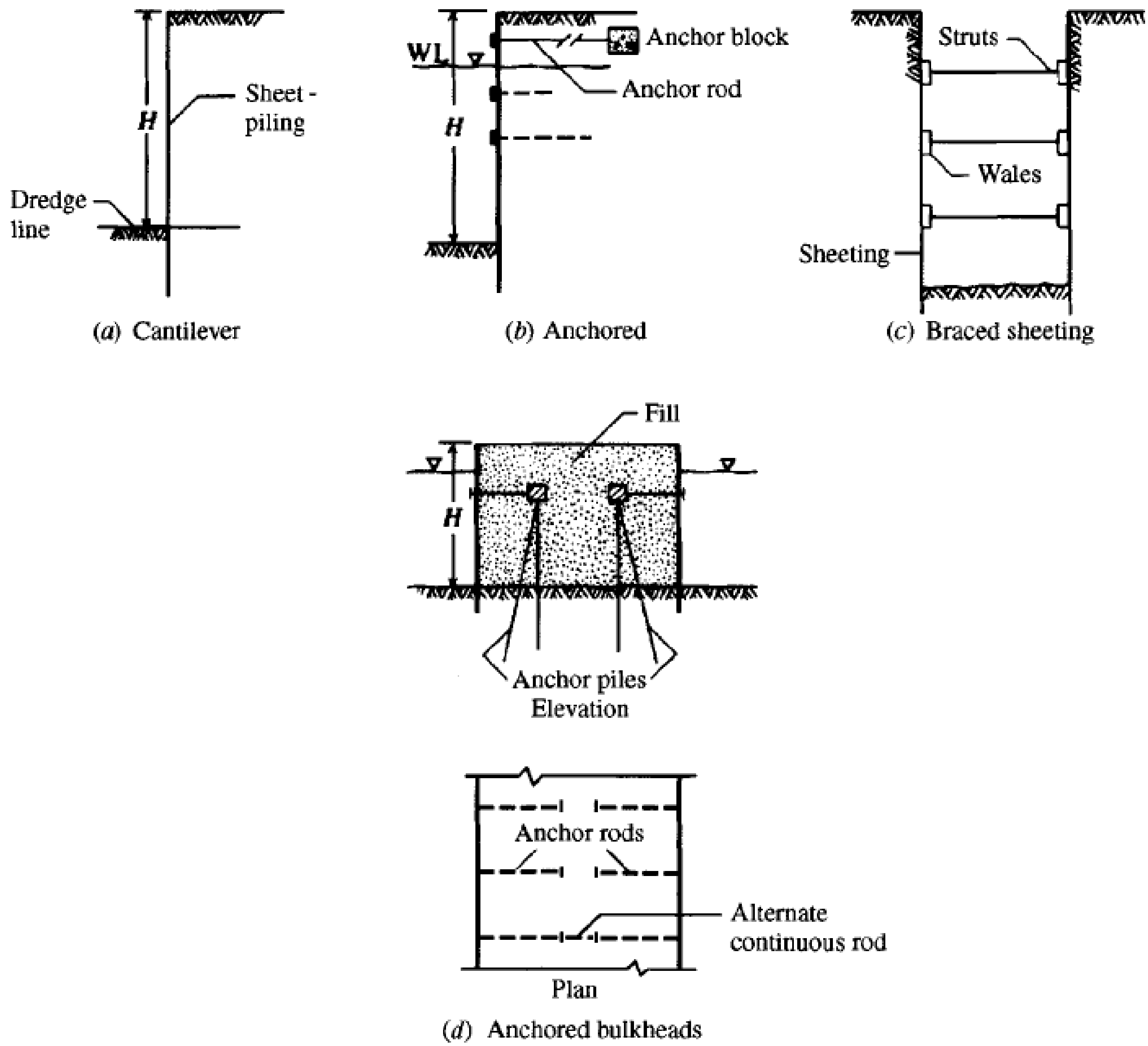


Figure 2.1 *Sheet-pile structures (Bowles, 1996)*

2.2 TYPES AND MATERIALS USED FOR SHEETPILING

Sheet piles may be of timber, reinforced concrete or steel. Allowable design stresses are often higher than in general building construction and may be from about 0.65 to 0.90 f_y for steel and wood. Reinforced concrete design stresses may be on the order of $0.75f'_c$ for unfactored loads. The design stress actually used will depend on engineering judgment, effect of wall failure (site importance factor), and the local building code.

2.2.1 Timber Sheetpiling

Timber piling is used for short spans and to resist light lateral loads such as free-standing walls of H < 3 m. It is more often used for temporarily braced sheeting to prevent trench cave-ins during installation of deep water and sewer lines. If timber sheeting is used in permanent structures above water level, preservative treatment is necessary, and even so the useful life is seldom over 10 to 15 years (Bowles, 1996). At present timber is little used except in temporary retaining structures owing to both the scarcity of timber—particularly of large cross section—and cost. Several timber piling shapes are shown in Fig. 2.2, of which the Wakefield and V groove piling have been and are the most used. Dimensions shown are approximate and you will have to use what is currently available.

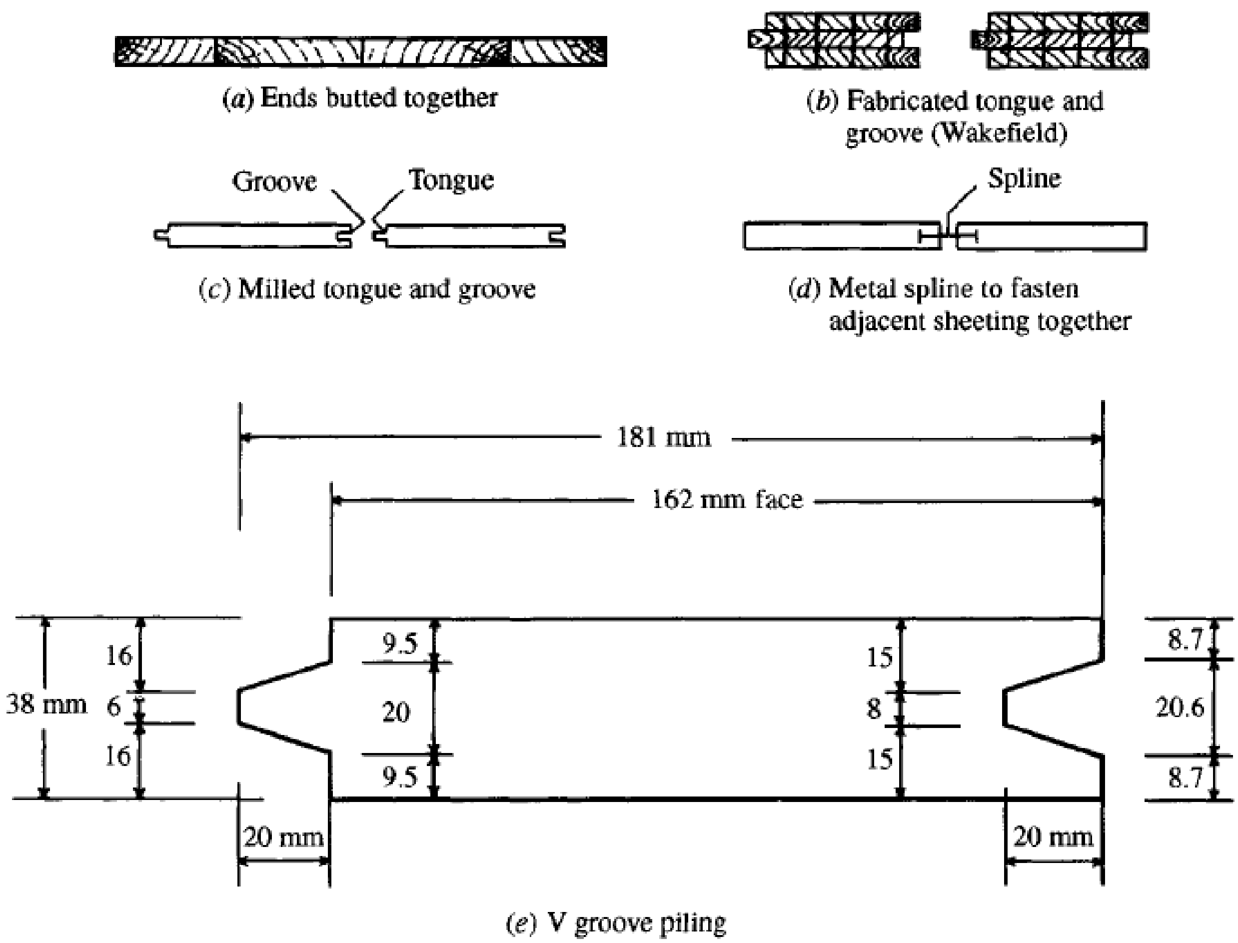


Figure 2.2 *Timber sheet piles*

It is common to see low timber walls treated with wood preservative in use along waterfronts. A substantial amount of timber piling—mostly fast-growing pine—is still used for protection where the piling is driven, then surrounded with stabilizing blocks or boulders (termed groins) to catch sand from the ocean side to maintain beaches. Here the intent is for the wall eventually to become covered with sand from tidal action. Strength is not the primary concern for this use, so if the wood lasts long enough to become buried, the purpose of the wall has been accomplished.

If wood sheetpiling is being considered, the soil type is a major factor. Almost any driving requires interfacing the pile hammer with a driving cap over the timber to minimize top damage. Driving in hard or gravelly soil tends to damage or even split the pile tip. Damage can sometimes be avoided by driving and pulling a steel mandrel or the like or by using a water jet to create a "predrilled" hole to reduce the driving resistance.

2.2.2 Reinforced Concrete Sheetpiling

These sheet piles are precast concrete members, usually with a tongue-and-groove joint. Even though their cross section is considerably dated (see Fig. 2.3), this form is still used. They are designed for service stresses, but because of their mass, both handling and driving stresses must also be taken into account. The points are usually cast with a bevel, which tends to wedge the pile being driven against the previously driven pile.

The typical dimensions shown in Fig. 2.3 indicate the piles are relatively bulky. During driving they will displace a large volume of soil for an increase in driving resistance. The relatively large sizes, coupled with the high unit weight (γ_c = 23.6 kN/m^3) of concrete, mean that the piles are quite heavy and may not be competitive with other pile types unless they are produced near the job site.

Dimensions and reinforcing bars shown in Fig. 2.3 are typical, but currently produced piles will contain bars that are available to the producer at casting time.

If the joints are cleaned and grouted after they have been driven, a reasonably watertight wall may be obtained (Bowles, 1996). However, if the wall is grouted, expansion joints may be required along the wall at intervals that are multiples of the section width.

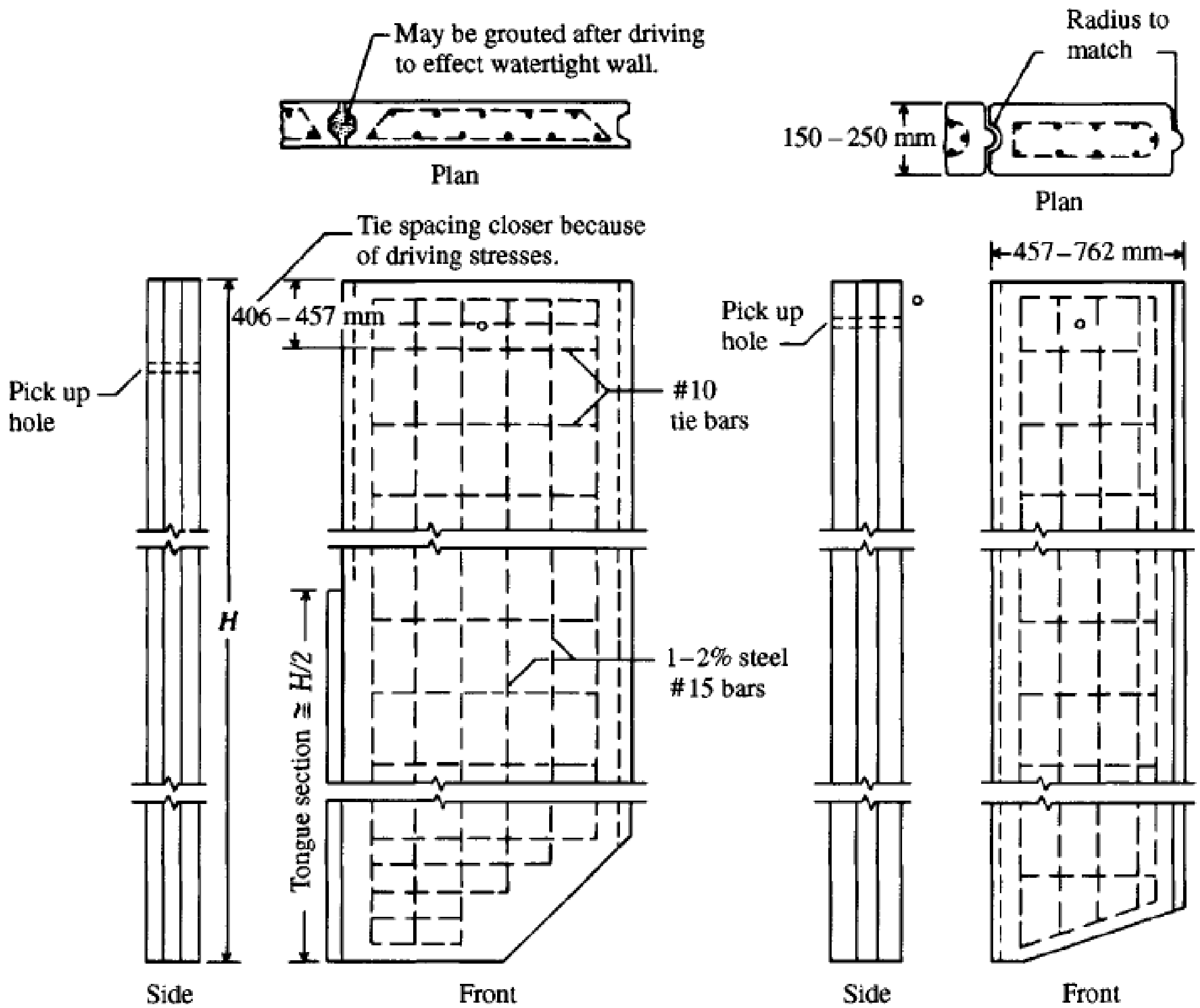


Figure 2.3 *Typical details of reinforced concrete sheet piles.*

2.2.3 Steel Sheetpiling

The most common types of piles used are steel sheet piles. Steel sheet piles are available in the market in several shapes. Some of the typical pile sections are shown in Fig. 2.4. The archweb and Z-piles are used to resist large bending moments, as in anchored or cantilever walls. Where the bending moments are less, shallow-arch piles with corresponding smaller section moduli can be used. Straight-web sheet piles are used where the web will be subjected to tension, as in cellular cofferdams. The ball-and-socket type of joints, Fig. 2.5 (d), offer less driving resistance than the thumb-and-finger joints, Fig. 2.5 (c).

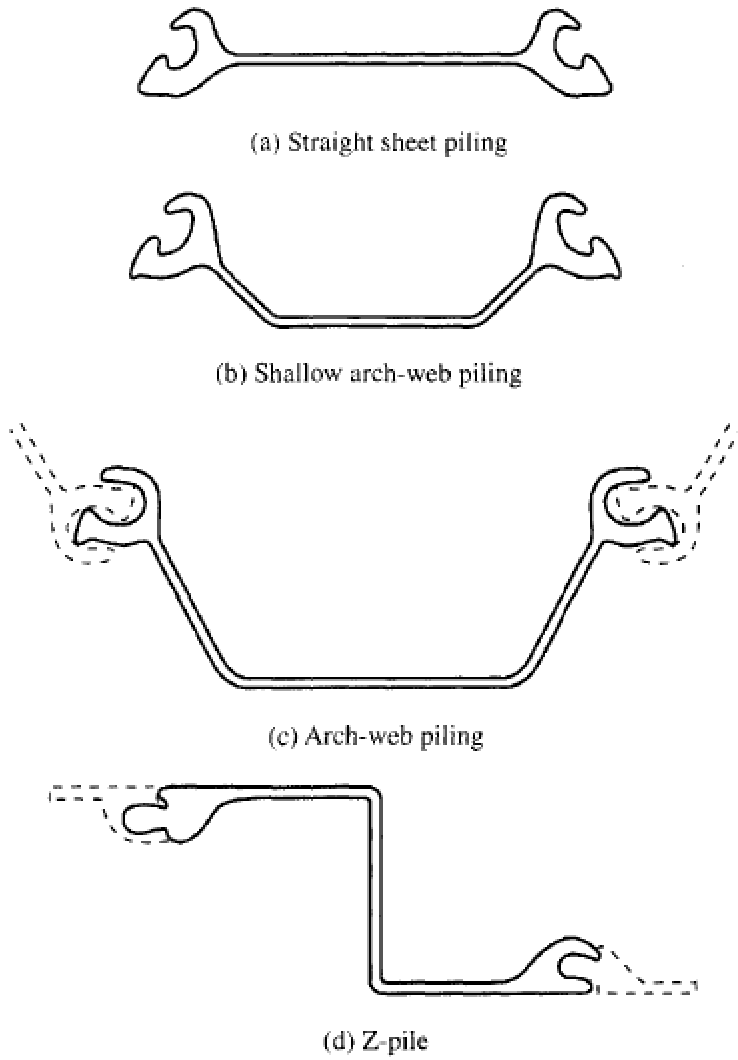


Figure 2.4 *Steel Sheet pile sections*

Steel piles possess several advantages over the other types. Some of the important advantages are (Bowles, 1996):

1. It is resistant to the high driving stresses developed in hard or rocky material.
2. It is relatively lightweight.
3. It may be reused several times.
4. It has a long service life either above or below water if it is provided with modest protection according to NBS (1962), which summarizes data on a number of piles inspected after lengthy service. Watkins (1969) provides some guidance for considering corrosion of sheetpiling in sea water.

5. It is easy to increase the pile length by either welding or bolting. If the full design length cannot be driven, it is easy to cut the excess length using a cutting torch.
6. Joints are less apt to deform when wedged full with soil and small stones during driving.
7. A nearly impervious wall can be constructed by driving the sheeting with a removable plug in the open thumb-and-finger joint. The plug is pulled after the pile is driven, and the resulting cavity is filled with a plastic sealer. The next pile section is then driven with the intersecting thumb or ball socket displacing part of the plastic sealer from the prefixed cavity. When the piling is driven in pairs, sealing the intermediate joint by prefilling may not provide a 100 percent impervious joint. Sellmeijer et al. (1995) describe an experimental wall project using this general approach but with European-produced piling, which has a slightly different joint configuration than the standard "thumb-and-finger" or "ball-and-socket" interlocks of piling produced in the United States (see Fig. 2.5).

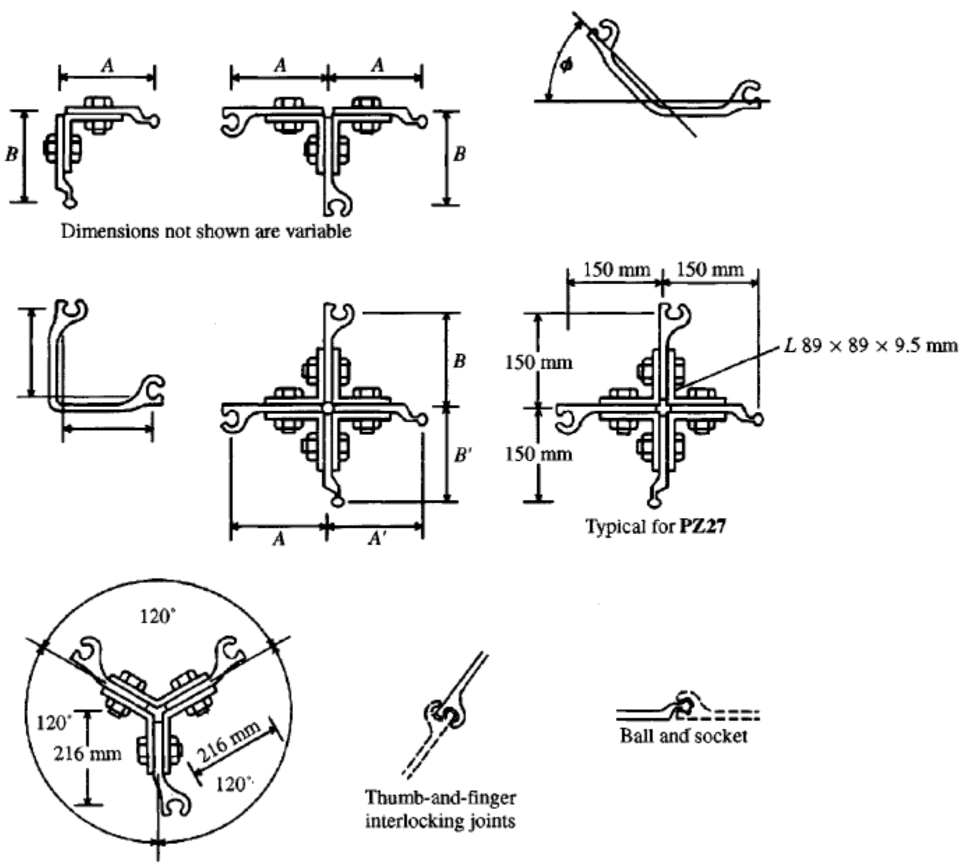


Figure 2.5 *Typical fabricated or rolled sheet-pile joints.*

Figure 2.5 illustrates several angle sections and joints that can be fabricated from cut pieces of sheetpiling; these are for illustration, as other joints can be produced. The crosses and wyes shown are used in cellular cofferdams; the angles and bends are used for direction changes in the wall.

When the stiffness capacity of the available Z piles is insufficient, the box sections (Fig. 2.6a) or the soldier-Z-pile combination of Fig. 2.6b might be used.

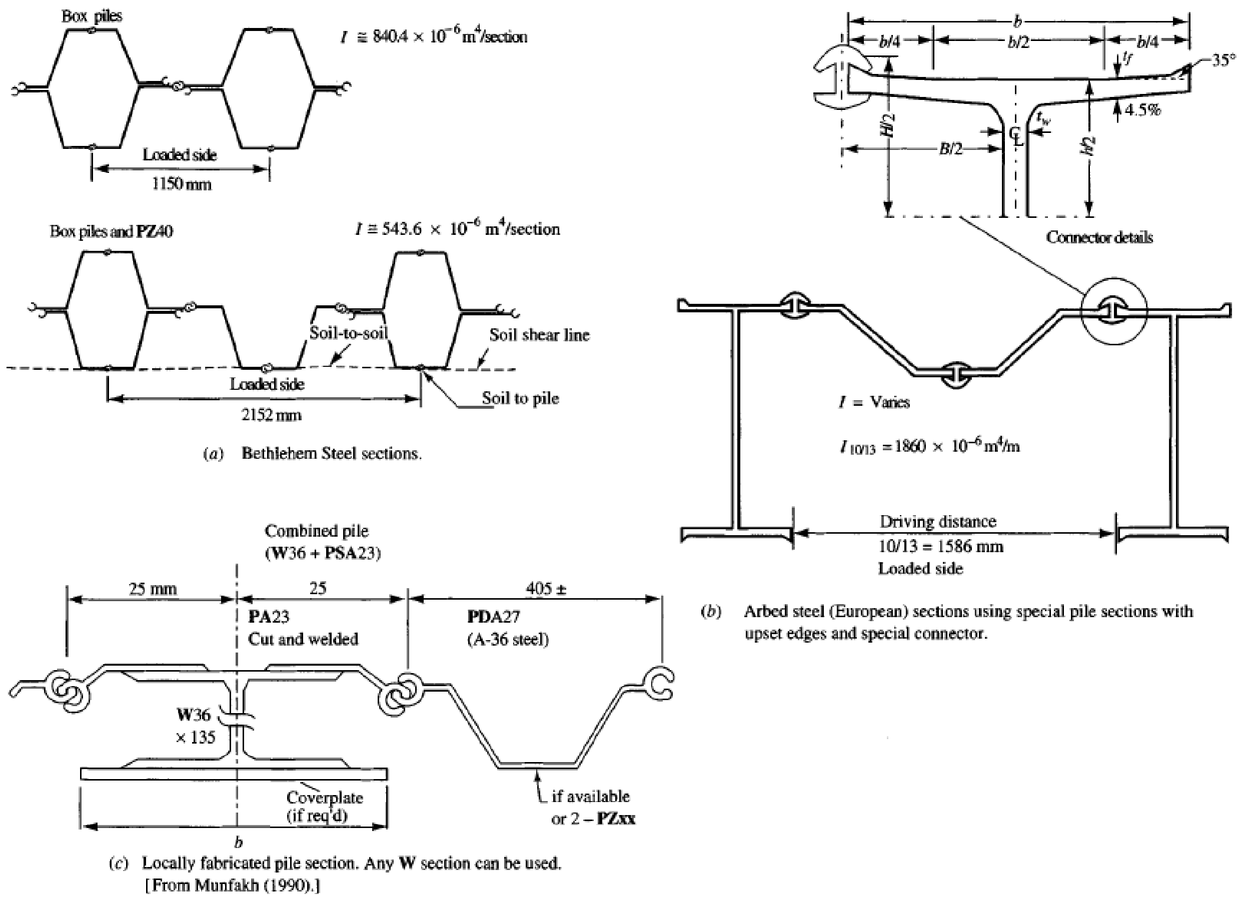


Figure 2.6 *Typical Built-up pile sections used where standard rolled shapes do not have adequate bending stiffness.*

2.2.4 Composite Sheet-Pile Walls

Walls may be constructed using composite construction. The soldier beam-wood lagging combination is an example. Other examples include use of soldier beams on some spacing with sheetpiling used between the spacings. For corrosion protection one might encase the upper part of steel sheetpiling in concrete after it is driven, with the concrete extending from below the water line to the pile top. A wood facing might also be used, or the lower part of the sheeting could be made of steel and the upper part of a different material-wood or concrete (Bowles, 1996). Since steel is relatively durable in most waterfront installations, the principal composite construction consists in using a mix of soldier beams and sheet piles or built-up box pile sections.

2.2.5 Fiber reinforced polymer

Sheet piling may also be manufactured of a synthetic fiber-reinforced polymer (FRP). This type of sheet pile is also referred to as fiberglass or composite sheet pile. A FRP product consists of fiber reinforcement and a polymer resin matrix. The fiber reinforcement typically consists of glass reinforcing fibers. Because of the method of manufacturing, the mechanical properties (strength, modulus of elasticity) may vary with orientation. Due to the potential strength of FRP sheet pile and its resistance to corrosion, FRP could be considered for applications requiring wall heights higher than allowed for vinyl sheet pile or for areas with high corrosion potential for steel. The required strength, modulus of elasticity, and anisotropic nature of the material must be considered in the design.

2.2.6 Vinyl

Vinyl sheet pile is available in sections of similar shape to Z-shaped steel sheet pile. Vinyl has lower strength and modulus of elasticity than steel and is, therefore, limited to lower wall heights or anchored walls. Vinyl sheet pile may be manufactured by monoextrusion of all virgin material or coextrusion of recycled material, coextruded with a virgin material coating to provide resistance to ultraviolet light.

2.3 SHEET PILE STRUCTURES

Sheet piles may conveniently be used in several civil engineering works as shown in Fig. 2.7. They may be used as:

1. Cantilever sheet piles
2. Anchored bulkheads
3. Braced sheeting in cuts
4. Single cell cofferdams
5. Cellular cofferdams, circular type

6. Cellular cofferdams (diaphragm)

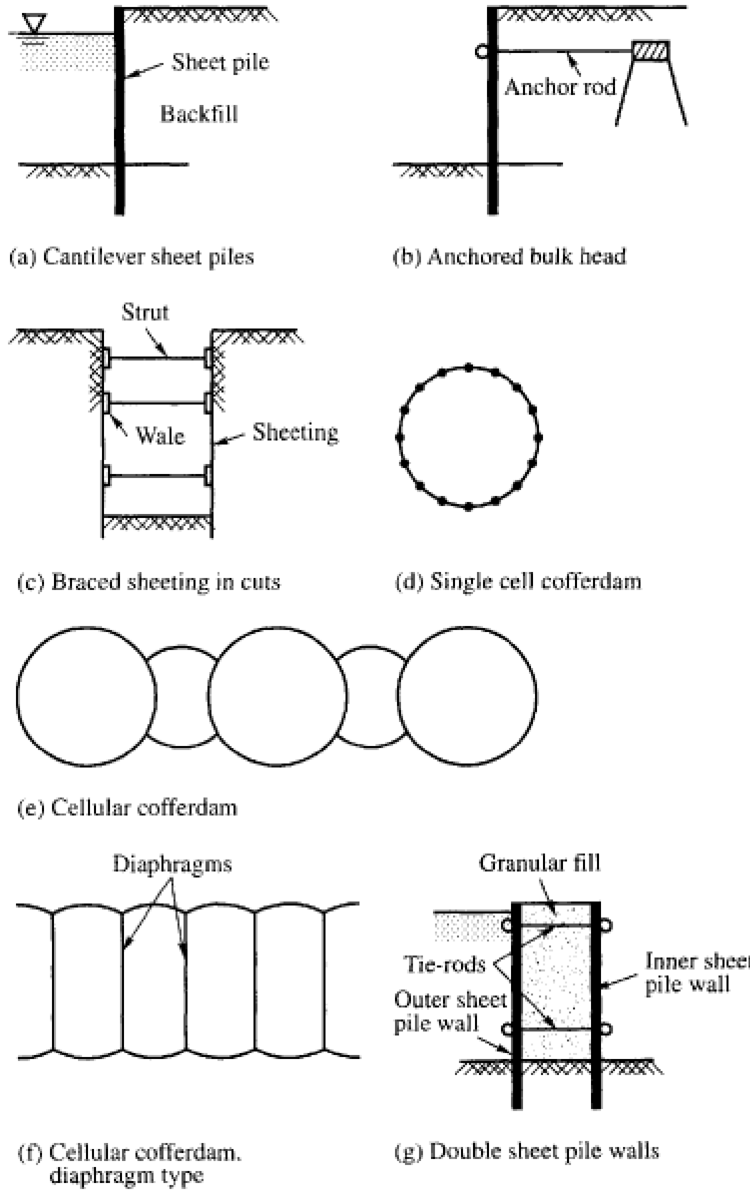


Figure 2.7 *Use of sheet piles*

Anchored bulkheads Fig. 2.7 (b) serve the same purpose as retaining walls. However, in contrast to retaining walls whose weight always represent an appreciable fraction of the weight of the sliding wedge, bulkheads consist of a single row of relatively light sheet piles of which the lower ends are driven into the earth and the upper ends are anchored by tie or anchor rods. The anchor rods are held in place by anchors which are buried in the backfill at a considerable distance from the bulkhead.

Anchored bulkheads are widely used for dock and harbor structures. This construction provides a vertical wall so that ships may tie up alongside, or to serve as a pier structure, which may jet out into the water. In these cases sheeting may be required to laterally support a fill on which railway lines, roads or warehouses may be constructed so that ship cargoes may be transferred to other areas. The use of an anchor rod tends to reduce the lateral deflection, the bending moment, and the depth of the penetration of the pile.
Cantilever sheet piles depend for their stability on an adequate embedment into the soil below the dredge line. Since the piles are fixed only at the bottom and are free at the top, they are called cantilever sheet piles. These piles are economical only for moderate wall heights, since the required section modulus increases rapidly with an increase in wall height, as the bending moment increases with the cube of the cantilevered height of the wall. The lateral deflection of this type of wall, because of the cantilever action, will be relatively large. Erosion and scour in front of the wall, i.e., lowering the dredge line, should be controlled since stability of the wall depends primarily on the developed passive pressure in front of the wall.

2.4 CANTILEVER SHEET PILE WALLS

The principle with a cantilever sheet pile wall is that it is not anchored or strutted. It has to be driven to a depth under the excavation bottom where the active and the passive earth pressures balancing each other, i.e. moment and horizontal equilibrium occurs. The required depth of the sheet pile wall under the excavation bottom is based on moment equilibrium, defined in Equation (2.1). (Person and Sigstrom, 2010)

$$P_p \times h_p > P_A \times h_A \qquad (2.1)$$

where the parameters are defined in Figure 2.8. Horizontal equilibrium is assumed to be satisfied with an extra 20 % of embedding. A cantilever sheet pile wall rotating around a point at the lower part of the sheet pile wall.

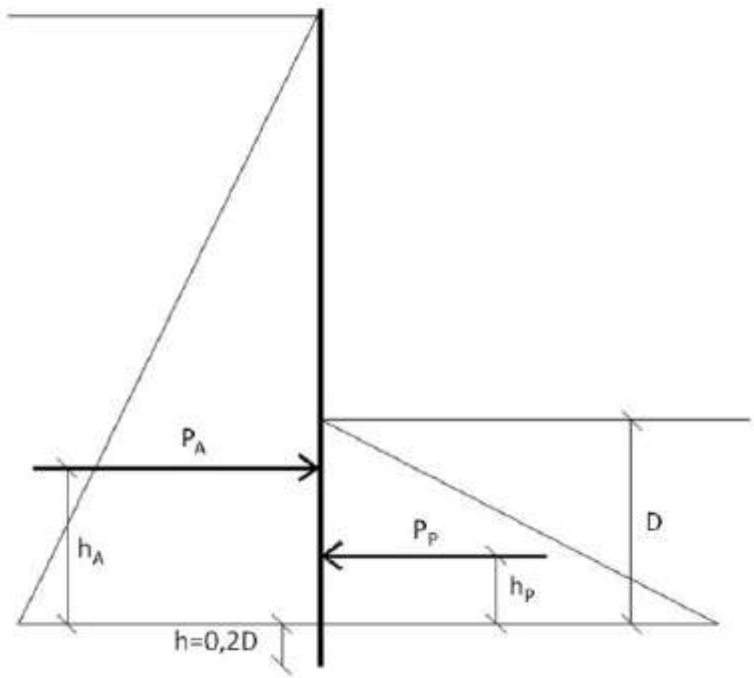


Figure 2.8 *Earth pressure against a cantilever sheet pile wall.*

2.5 ANCHORED BULKHEAD

To design a sheet pile wall, several empirical and semi-empirical methods have been developed, all of which are based on the classical lateral earth pressure theories. Several methods have been developed in the design of sheet pile wall; however the tow most common methods are the Free-earth support method and Fixed-earth support method. The main different between this methods lies in the influence with which the depth of embedment has on the deflected shape of the wall. (Dan Saliran and Salahuddin, 2009)

a) Free-earth support method

The Free-earth support method is based on the assumption that the sheet pile is embedded to a sufficient depth into the soil to prevent translation, but not rotation at the toe and a pinned support is assumed. This condition and simplified assumptions of active (from filled side) and passive pressure on the free side below the dredge line are shown in Fig. 2.9. For the supported wall, a strut (prop) or tie near the top of the wall provides the other support. Compare to Fixed-earth support method under similar set of conditions, the relative length of pile required is less but the maximum bending moments are higher.

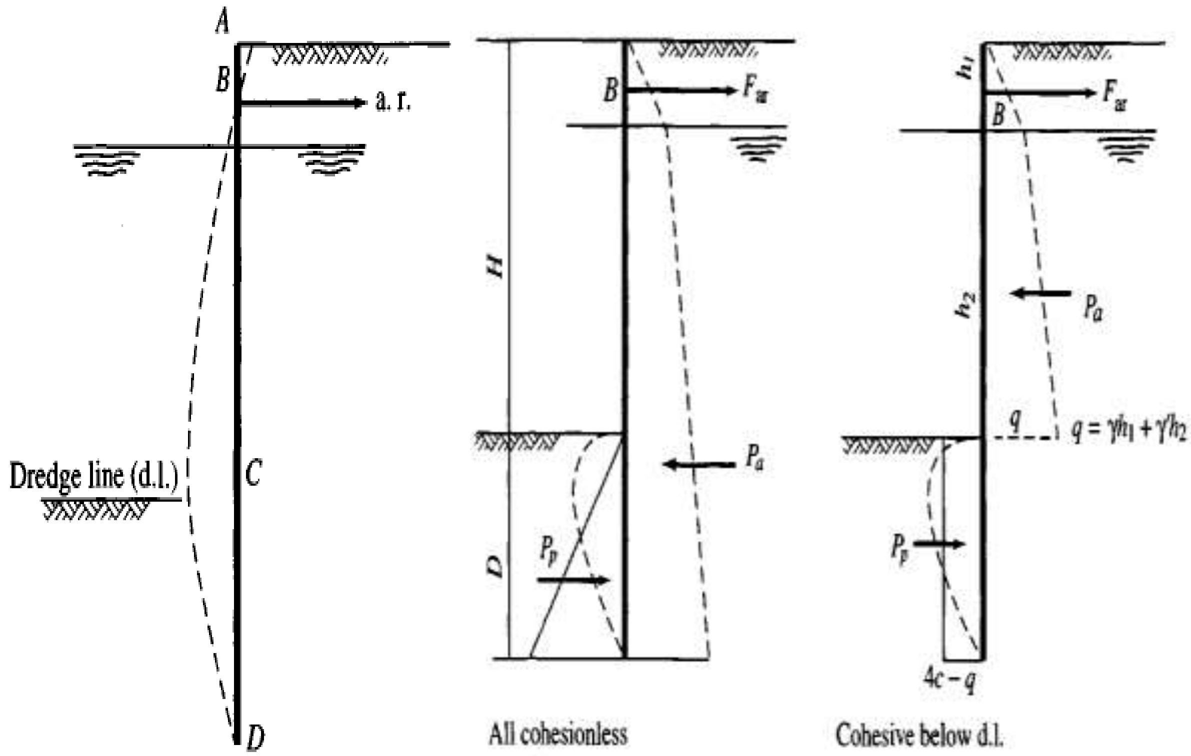


Figure 2.9 *Free-earth support deflection line (qualitative) and assumed and probable (dashed) soil resistance and active earth pressure profile for "free-earth" support method (Bowles, 1996).*

b) Fixed-earth support method

A wall designed using Fixed-earth support principles are embedded sufficiently deep enough so that at the foot of the wall, both translation and rotation are prevented and fixity is assumed. Fig. 2.10 illustrated the deflected shape of an anchored sheet pile wall. The effect of toe fixity is to create a fixed end moment in the wall, reducing the maximum bending moments for a given set of conditions but at the expense of increased pile length. The design method used (whether Free-earth support or Fixed-earth support Method) should also consider the effects of hydrostatic pressures and surcharge loads, which are usually added to the soils.

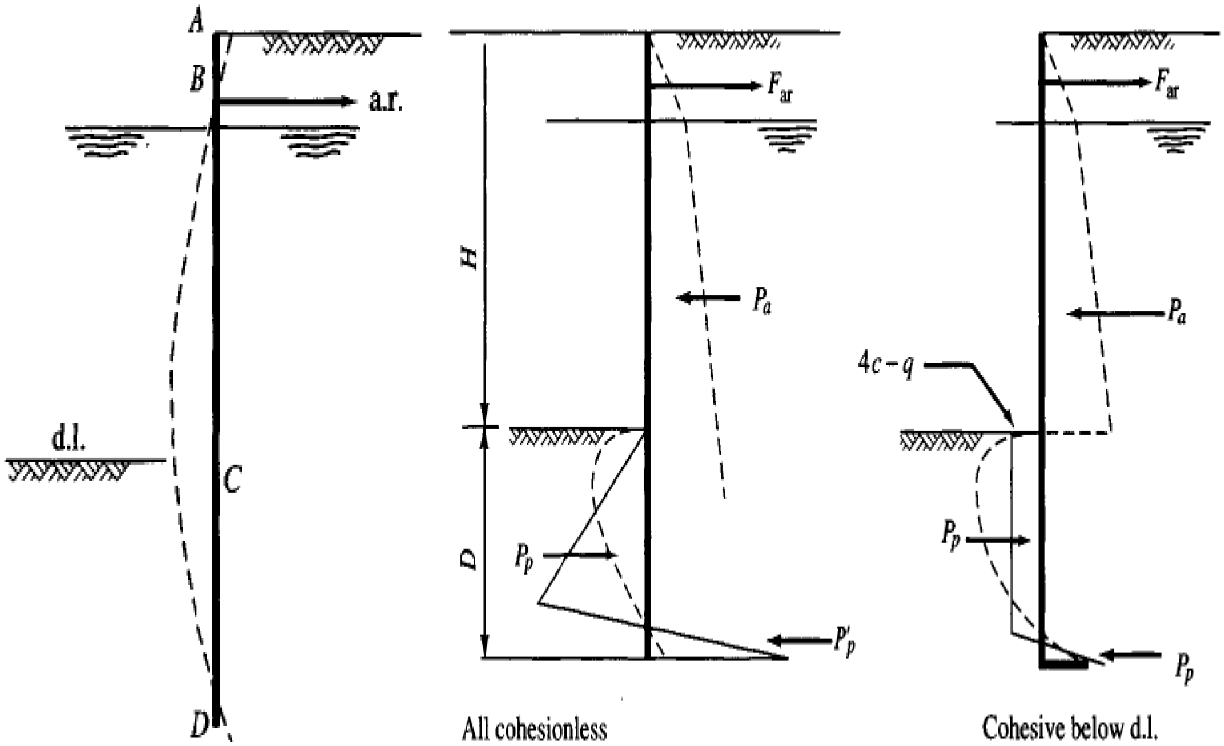


Figure 2.10 *Fixed-earth support deflection line (qualitative) and assumed and probable (dashed) soil resistance and active earth pressure profile for "fixed-earth" support method (Bowles, 1996).*

The design was based primarily on taking moments about the anchor rod, increasing the depth of embedment D until ΣF_h was satisfied, and then computing the resulting bending moments in the piling. A safety factor was incorporated by using a reduced K_p for passive pressure or by increasing the embedment depth D some arbitrary amount such as 20 or 30 percent. Two of the simplifications could result in errors (Bowles, 1996):

1. Unless the anchor rod elongates sufficiently, the active pressure may not fully develop, resulting in a computed anchor rod force that is too small.

2. The center of pressure below the dredge line is qualitatively shown by the dashed lines of Fig. 2.9 and 2.10 and is closer to the dredge line than assumed using the passive pressure profiles shown. The erroneous location of the center of pressure usually results in moments that are too large.

2.6 DESIGN CHARTS FOR ANCHORED BULKHEADS IN SAND

Hagerty and Nofal (1992) provided a set of design charts for determining

1. The depth of embedment
2. The tensile force in the anchor rod and
3. The maximum moment in the sheet piling

The charts are applicable to sheet piling in sand and the analysis is based on the free-earth support method. The assumptions made for the preparation of the design charts are:

1. For active earth pressure, Coulomb's theory is valid
2. Logarithmic failure surface below the dredge line for the analysis of passive earth pressure.
3. The angle of friction remains the same above and below the dredge line
4. The angle of wall friction between the pile and the soil is $\phi/2$

The various symbols used in the charts are,

h_a = the depth of the anchor rod below the backfill surface
h_1 =the depth of the water table from the backfill surface
h_2 = depth of the water above dredge line
H = height of the sheet pile wall above the dredge line
D = the minimum depth of embedment required by the free-earth support method
Ta = tensile force in the anchor rod per unit length of wall

Hagerty and Nofal developed the curves given in Fig. 2.11 on the assumption that the water table is at the ground level, that is $h_1 = 0$. Then they applied correction factors for $h_1 > 0$. These correction factors are given in Fig. 2.12. The equations for determining D, T_a and $M_{(max)}$ are:

$$D = G_d C_d H \qquad (2.2)$$

$$Ta = G_t C_t \gamma_a H^2 \qquad (2.3)$$

$$M_{(max)} = G_m C_m \gamma_a H^3 \qquad (2.4)$$

where,

G_d = generalized non-dimensional embedment = D/H for h_1, = 0

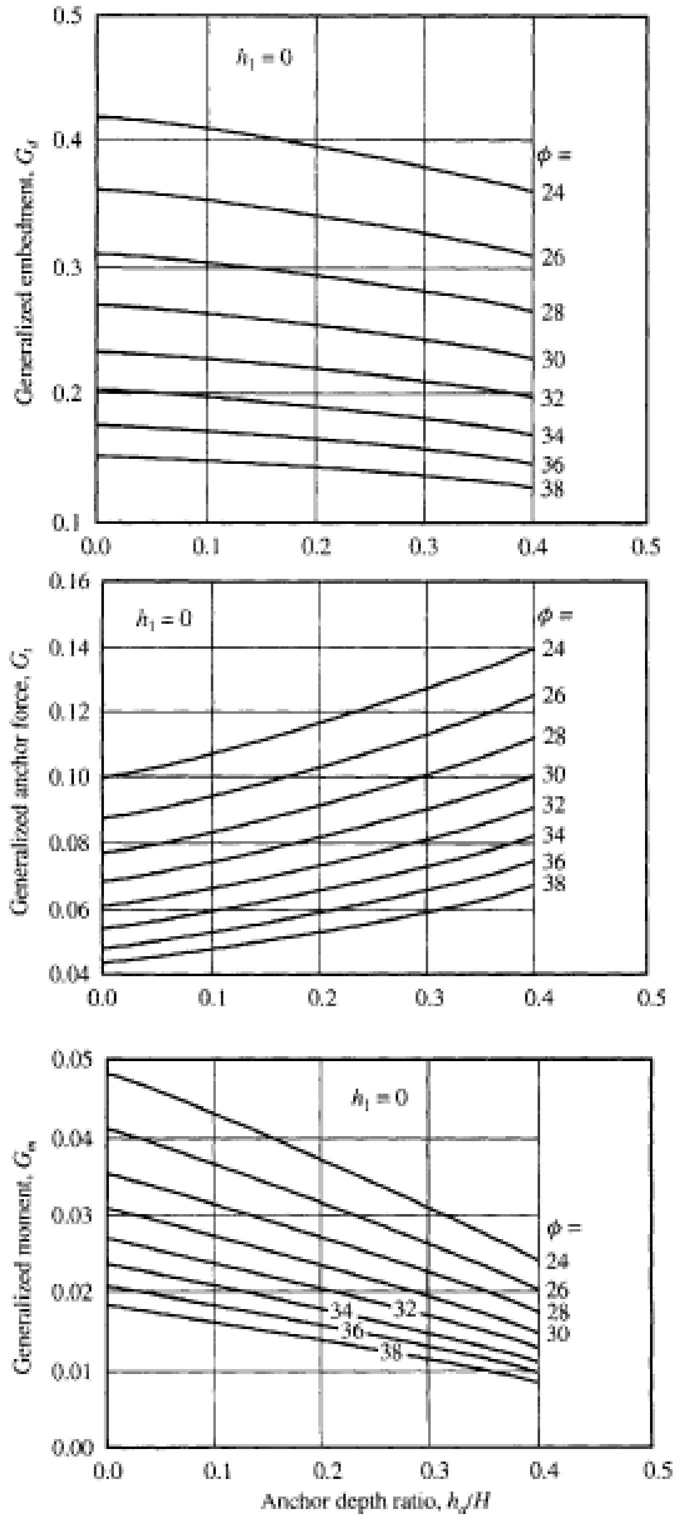


Figure 2.11 *Generalized (a) depth of embedment, Gd, (b) anchor force Gt, and (c) maximum moment Gm (after Hagerty and Nofal, 1992)*

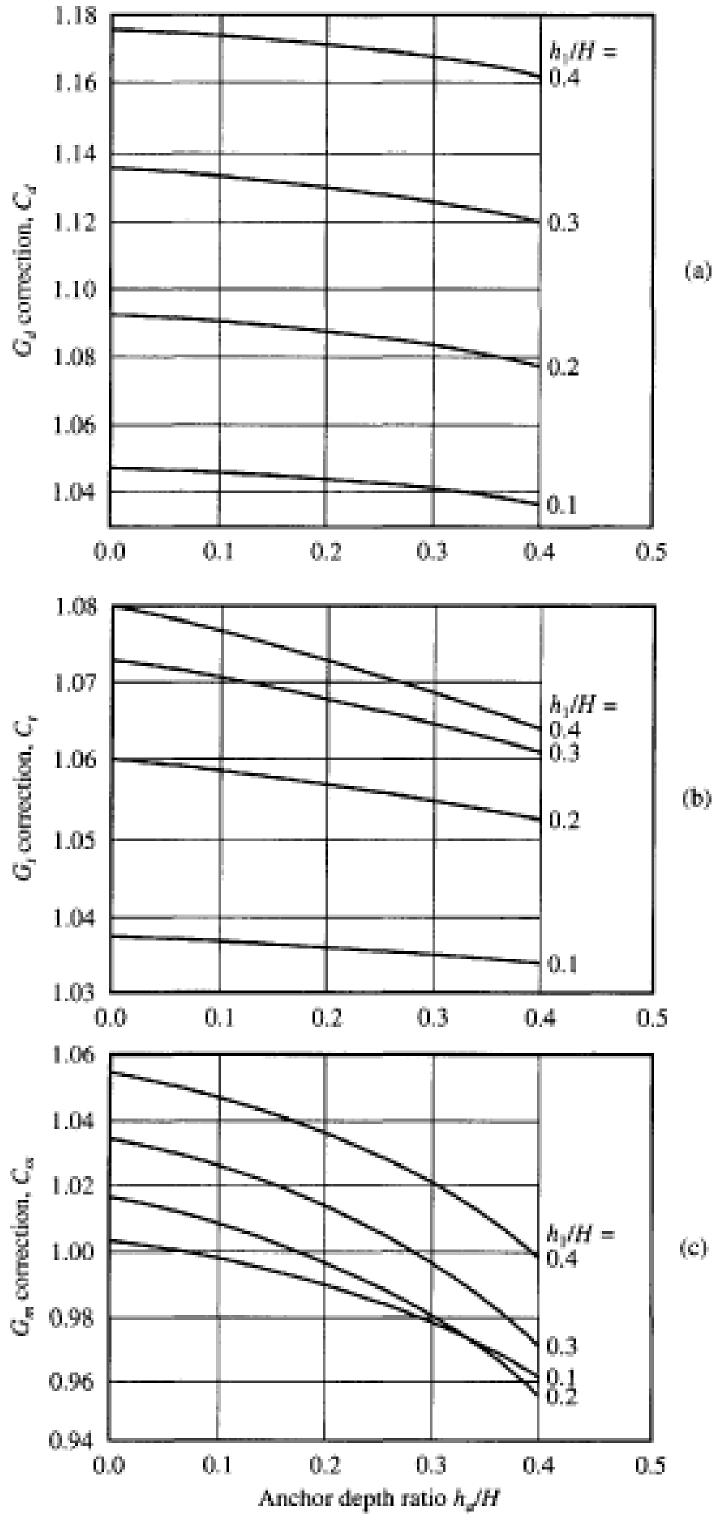


Figure 2.12 *Correction factors for variation of depth of water h1, (a) depth correction Cd, (b) anchor force correction Ct and (c) moment correction Cm (after Hagerty and Nofal, 1992)*

G_t = generalized non-dimensional anchor force = $T_a / (\gamma_a H^2)$ for $h_1 = 0$
G_m = generalized non-dimensional moment = $M_{(max)} / \gamma_a (H^3)$ for $h_1 = 0$
C_d, C_t, C_m = correction factors for $h_1 > 0$
γ_a = average effective unit weight of soil = $(\gamma_m h_1^{\ 2} + \gamma_b h_2^{\ 2} + 2\gamma_m h_1 h_1)/H^2$
γ_m = moist or dry unit weight of soil above the water table
γ_b = submerged unit weight of soil

The theoretical depth D as calculated by the use of design charts has to be increased by 20 to 40% to give a factor of safety of 1.5 to 2.0 respectively.

2.7 MOMENT REDUCTION FOR ANCHORED SHEET PILE WALLS

The design of anchored sheet piling by the free-earth method is based on the assumption that the piling is perfectly rigid and the earth pressure distribution is hydrostatic, obeying classical earth pressure theory. In reality, the sheet piling is rather flexible and the earth pressure differs considerably from the hydrostatic distribution.
As such the bending moments M(max) calculated by the lateral earth pressure theories are higher than the actual values. Rowe (1952) suggested a procedure to reduce the calculated moments obtained by the free earth support method.

Anchored Piling in Granular Soils

Rowe (1952) analyzed sheet piling in granular soils and stated that the following significant factors are required to be taken in the design

1. The relative density of the soil
2. The relative flexibility of the piling which is expressed

$$\rho = 109 \times 10^{-6} \left(\frac{\hat{H}^4}{EI}\right) \qquad (2.5a)$$

where,

ρ = flexibility number
$\hat{H}$ = the total height of the piling in m
El = the modulus of elasticity and the moment of inertia of the piling (MN-m^2) per m of wall

Eq. (2.5) may be expressed in English units

$$\rho = \frac{\hat{H}^4}{EI} \qquad (2.5b)$$

where, Ĥ is in ft, E is in lb/in^2 and I is in in^4/ft of wall.

Anchored Piling in Cohesive Soils

For anchored piles in cohesive soils, the most significant factors are (Rowe, 1952)

1. The stability number

$$N_S = \frac{c}{\gamma H}\sqrt{(1+\frac{c_a}{c})} = 1.25\frac{c}{\gamma H} \qquad (2.6)$$

2. The relative height of piling

where,

H = height of piling above the dredge line in meters
γ = effective unit weight of the soil above the dredge line = moist unit weight above water level and buoyant unit weight below water level, kN/m^3
c = the cohesion of the soil below the dredge line, kN/m^2
c_a = adhesion between the soil and the sheet pile wall, kN/m^2
$\sqrt{(1+\frac{c_a}{c})} = 1.25$ for design purposes
α = ratio between H and Ĥ
M_d = design moment
M_{max} = maximum theoretical moment

Fig. 2.13 gives charts for computing design moments for pile walls in granular and cohesive soils.

2.8 LOADS

2.8.1 Lateral earth pressure

The lateral (horizontal) earth pressure is a function of the soil properties (cohesion, phi angle, and unit weight), height of overburden, and the elevation of the water table. Earth pressures varies from an initial state referred to as at-rest, K_0, to a minimum limit state referred to as active, K_A, to a maximum limit state referred to as passive, K_P. The classical method of sheet pile design assumes development of active and passive lateral earth pressures.

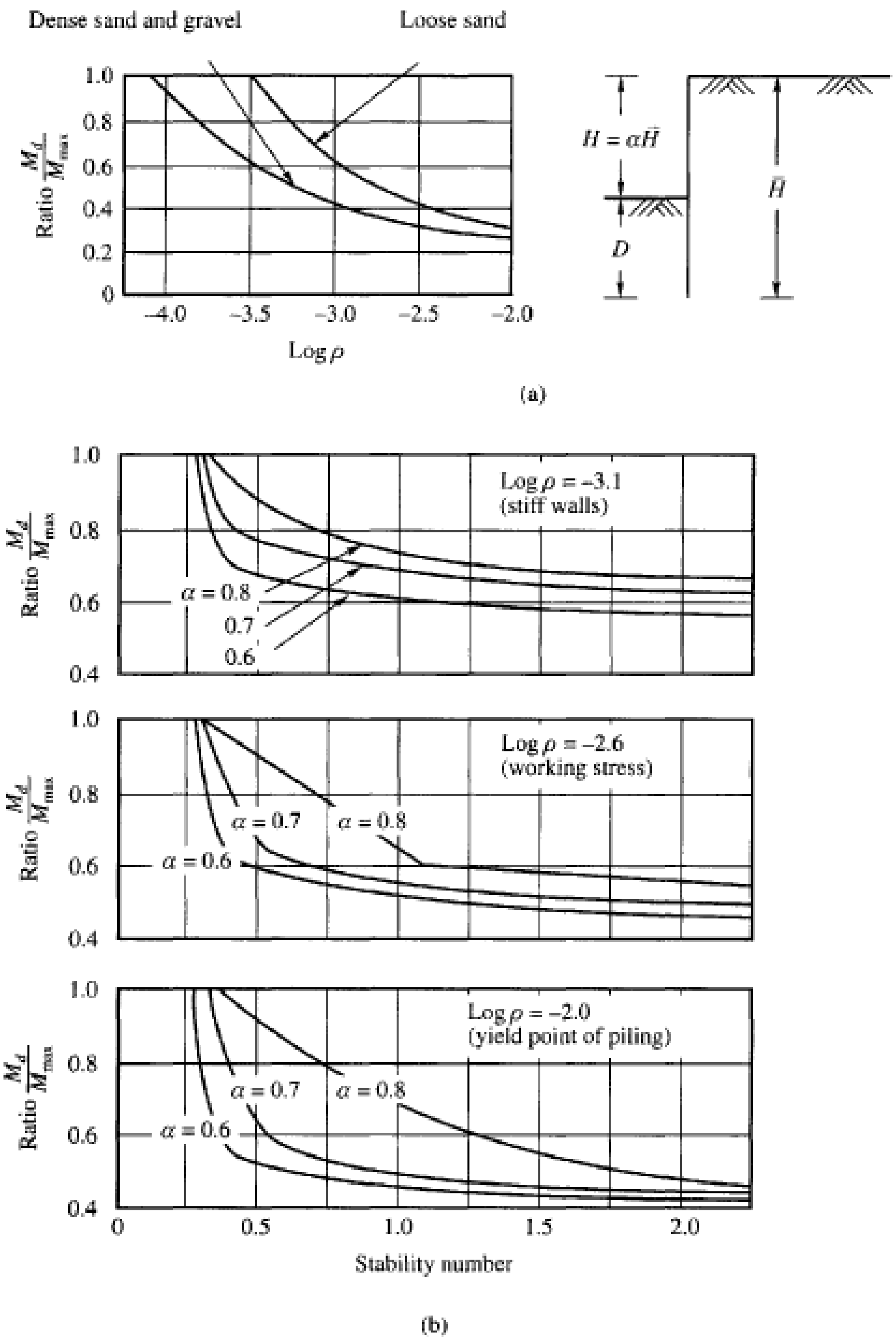


Figure 2.13 *Bending moments in anchored sheet piling by free-earth support method, (a) in granular soils, and (b) in cohesive soils (Rowe, 1952)*

Active earth pressure develops when the pile moves or rotates away from the soil allowing the soil to expand laterally (horizontally) in the direction of the pile movement (Fig. 2.14). Active earth pressure is the driving force in sheet pile stability analysis. In general, a lateral movement of approximately 1 inch is required to fully mobilize the shear resistance for each 20 feet of wall height.

Passive earth pressure develops when the pile moves or rotate towards the soil, tending to compress the soil laterally (horizontally) in the direction of the pile movement (Fig. 2.14). Passive earth pressure is the resisting force in sheet pile stability analysis. In general, a lateral movement of approximately 1 inch is required to fully mobilize the shear resistance for each 2 feet of wall height.

More rigorous analysis may be conducted, assuming that the soil behaves as a spring, with the maximum resistance equal to the active or passive lateral earth pressure, as appropriate.

Active state

Passive state

Figure 2.14 *Active and passive rotation*

2.8.2 Water loads

A difference in water level on either side of the wall creates unbalanced hydrostatic pressure, adding to the pressure forcing the wall outward (Fig. 2.15). The difference in water level may be because of a ground water table, which is higher in the bank than in the stream, a higher water level upstream of an inchannel sheet pile, or from a high stream flow which saturates the bank, followed by rapid drawdown when the water level in the stream drops faster than the water can drain from the bank.

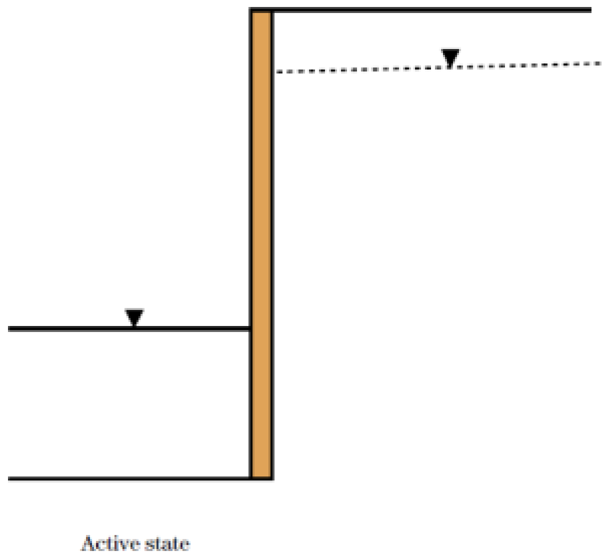


Figure 2.15 *Differential water load*

2.8.3 Surcharge

Surface surcharge (Fig. 2.16) also exerts lateral pressure on the wall, forcing the wall outward. Typical surcharge loadings may be due to equipment (parked or traveling), storage areas, construction materials, vehicles, and others. Surcharge loads are often estimated to be 100 to 200 pounds per square foot. Other surcharge loads include spoil, snow, or ice.

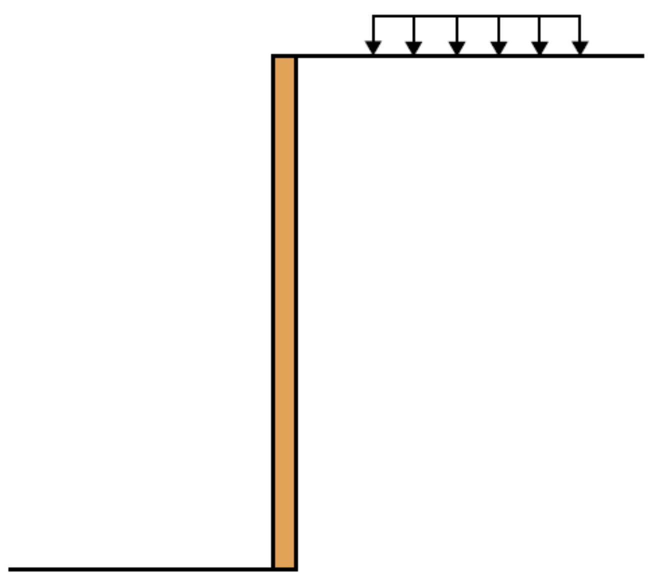


Figure 2.16 *Surface surcharge loading*

2.8.4 Wall stability

Both anchored and cantilever sheet pile must be analyzed against overturning. Wall penetration must be great enough to prevent deep-seated failure (Fig. 2.17 (USACE 1994c)) or rotational failure (Fig. 2.18 (USACE 1994c)). Deep-seated failure should be assessed by a slope stability analysis conducted by a geotechnical engineer.

The rotational stability of a cantilever wall or an anchored wall may be evaluated using methods presented in the Retaining Wall Design Guide. Penetration depths determined by the Retaining Wall Design Guide or Steel Sheet Piling Design Manual are typically increased by 30 percent to provide a factor of safety against overturning.

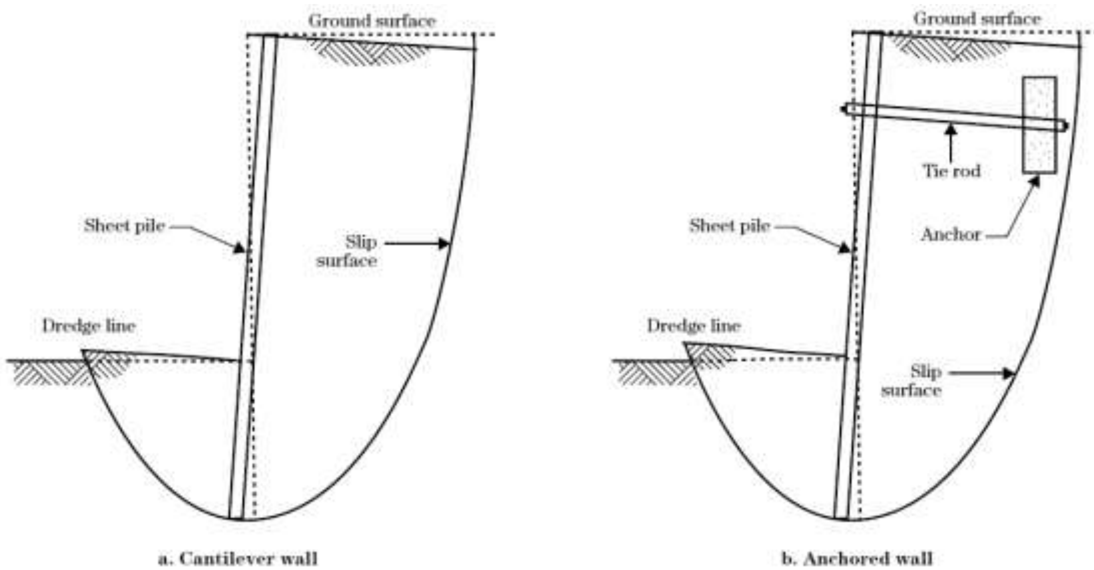


Figure 2.17 *Deep-seated failure*

2.8.5 Structural design

Sheet pile failure may also be caused by overstressing the pile (Fig. 2.19 (USACE 1994c)).

To avoid compounding factors of safety, the sheet piling and wales are designed to resist forces produced by soil pressures calculated using a factor of safety of 1.0 for both active and passive pressures (USACE 1994c). Therefore, the design bending moment, shear, and associated deflection for the sheet pile are based on a factor of safety of 1.0 for both active and passive soil pressures.

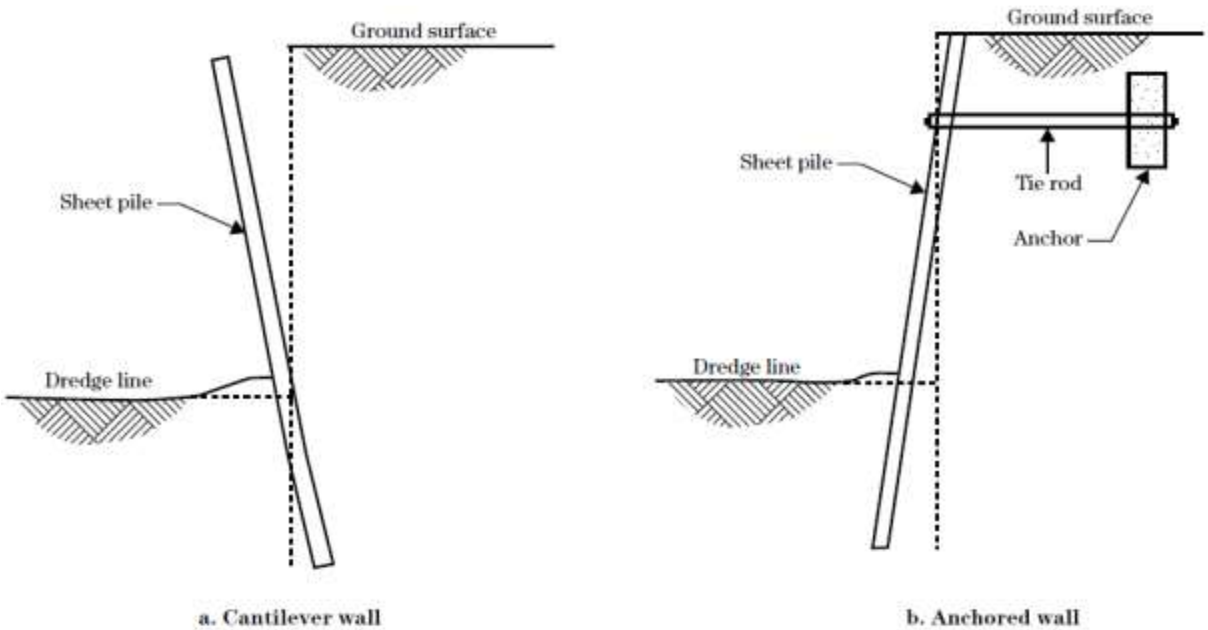


Figure 2.18 *Wall rotational failure*

No firm guidelines exist for acceptable deflection, and values ranging from 1 to 5 inches are typically considered acceptable. It is recommended that the deflection be limited to 1 to 3 inches for stream restoration and stabilization projects.

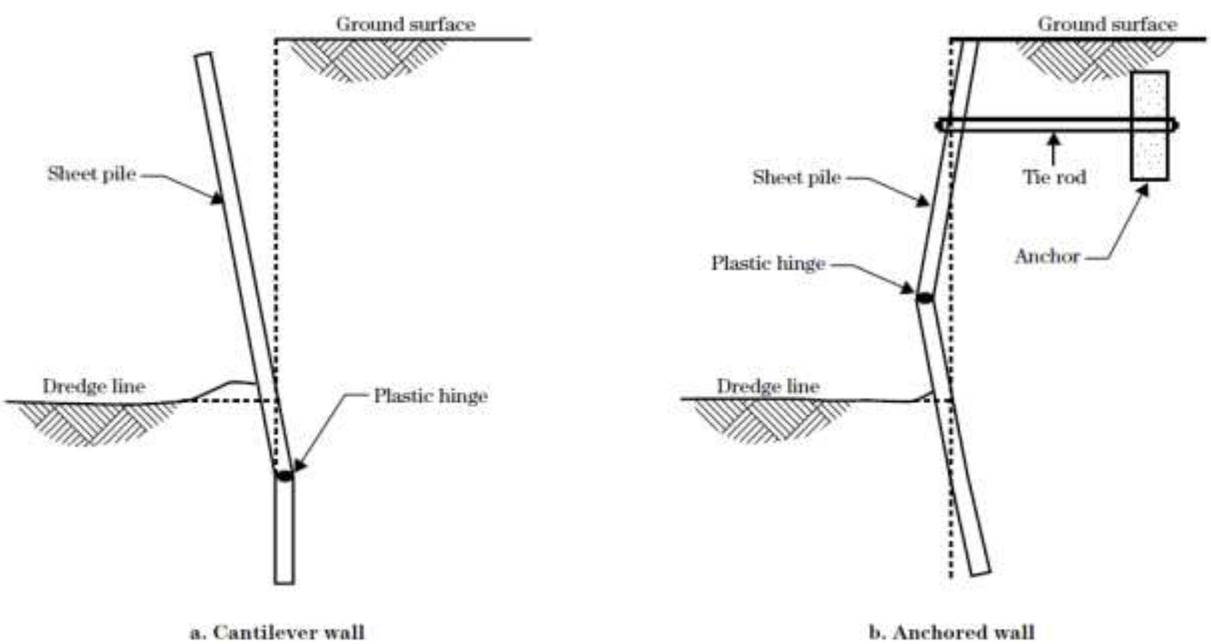


Figure 2.19 *Overstressed pile*

2.9 Construction considerations

Piling—Cold-rolled steel sheet pile sections have a weaker interlock than hot-rolled sections and may unlock while being driven in hard conditions, resulting in misalignment. A minimum pile thickness of a fourth inch is typically recommended for driveability. In tough driving conditions, such as dense to very dense sands, very stiff to hard clay soils, or soils containing significant amounts of gravel, a thicker pile should be considered. In areas where corrosion of the steel pile is a concern, a thicker pile than required structurally should be considered to allow for corrosion throughout the design life.

Equipment—Sheet pile is typically installed by driving, jetting, or trenching. Jetting is often not allowed for walls designed to retain soil. Hammers for driving may be steam, air, diesel-drop, single action, double action, differential action, or vibratory. Vibratory hammers work well in sand, silt, or softer clay soils. Harder driving conditions such as stiff clay may require an impact hammer.

Access for a crane is often required to operate the hammer. Short piles or piles in easier driving conditions may be installed with a backhoe or hammer attached to a back/track hoe. A temporary guide structure or template is recommended to ensure that the piles are driven in the correct alignment. Use of a protective cap is required with impact hammers. Protective shoes may be used on the tip of a pile in hard driving conditions.

When driving vinyl pile in stiff clays or dense sands, a steel mandrel is often driven with the vinyl pile and extracted upon completion of driving. The purpose of the mandrel is to support the vinyl pile only during driving.

Pile driving and installation—Piles should be driven with the proper size hammer for the size of pile, depth of penetration, and soil conditions. When impact hammers are used, the hammer should be appropriately sized and a protective cap utilized to prevent excessive damage to the pile. In some conditions, large impact hammers are not appropriate for driving smaller pile sections and have caused excessive damage to the pile. A smaller impact hammer may work better in these situations.

Alignment—Piles should be maintained in alignment during driving. Sheet pile should not be driven more than an eighth inch per foot out of plumb in the plane of the wall or perpendicular to the plane of the wall.

CHAPTER 3
ANCHORING

3.1 INTRODUCTION

A ground anchor normally consists of a high tensile steel cable or bar, called the tendon, one end of which is held securely in the soil by a mass of cement grout or grouted soil: the other end of the tendon is anchored against a bearing plate on the structural unit to be supported. The main application of ground anchors is in the construction of tie-backs for diaphragm or pile walls. Other applications are in the anchoring of structures subjected to overturning, sliding or buoyancy, in the provision of reaction for in-situ load tests and in pre-loading to reduce settlement. Ground anchors can be constructed in sands (including gravelly sands and silty sands) and stiff clays, and they can be used in situations where either temporary or permanent support is required (Craig, 2004).

Anchors transmit tensile forces into the rock mass. They are inserted into boreholes and bonded to the rock by grout or other chemicals. Their action is twofold. Firstly, on tensioning an anchor or rock bolt, the stress field is modified in the vicinity of the anchor. Secondly, where a tensioned anchor is holding a block of rock in its original position it also acts as a preventative measure against the further disintegration of the rock (Kıvanç, 2006).

A ground anchor functions as load carrying element, consisting essentially of a steel tendon inserted into suitable ground formations in almost any direction. Its loadcarrying capacity is generated as resisting reaction mobilized by stressing the ground along a specially formed anchorage zone. (Xanthakos, 1991)

3.2 GROUND ANCHORS

A prestressed grouted ground anchor is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground. Grouted ground anchors, referenced simply as ground anchors, are installed in grout filled drill holes (Sabatini et al. 1999). Grouted ground anchors are also referred to as "tiebacks". The basic components of a grouted ground anchor include the: (1) anchorage; (2) free stressing (unbonded) length; and (3) bond length. These and other components of a ground anchor are shown schematically in figure 3.1.

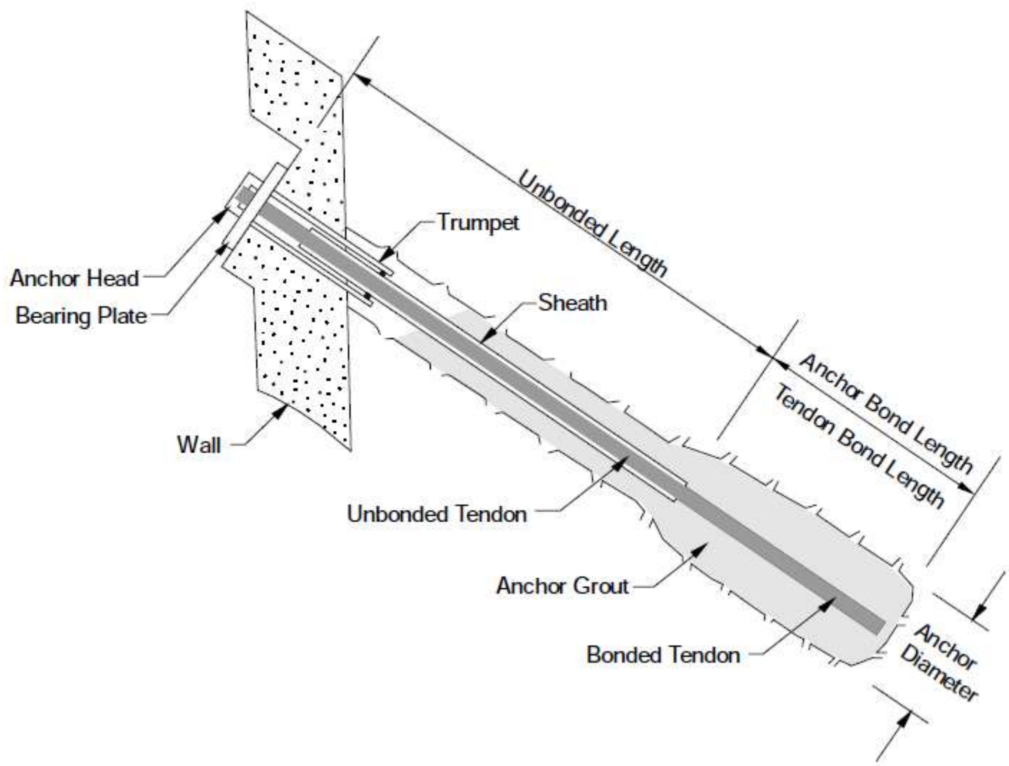


Figure 3.1 *Components of a ground anchor*

The anchorage is the combined system of anchor head, bearing plate, and trumpet that is capable of transmitting the prestressing force from the prestressing steel (bar or strand) to the ground surface or the supported structure. Anchorage components for a bar tendon and a strand tendon are shown in figure 3.2. The unbounded length is that portion of the prestressing steel that is free to elongate elastically and transfer the resisting force from the bond length to the structure. A bondbreaker is a smooth plastic sleeve that is placed over the tendon in the unbonded length to prevent the prestressing steel from bonding to the surrounding grout. It enables the prestressing steel in the unbonded length to elongate without obstruction during testing and stressing and leaves the prestressing steel unbonded after lock-off. The tendon bond length is that length of the prestressing steel that is bonded to the grout and is capable of transmitting the applied tensile load into the ground. The anchor bond length should be located behind the critical failure surface.

A portion of the complete ground anchor assembly is referred to as the tendon. The tendon includes the prestressing steel element (strands or bars), corrosion protection, sheaths (also referred to as sheathings), centralizers, and spacers, but

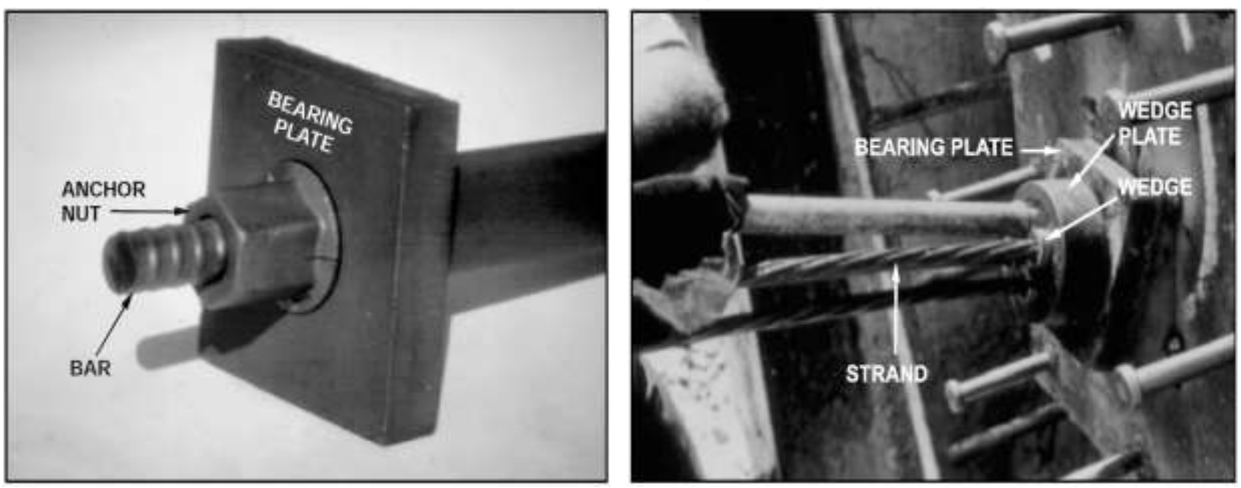


Figure 3.2 *Anchorage components for a bar tendon and strand tendon.*

specifically excludes the grout. The definition of a tendon, as described in PTI (1996), also includes the anchorage; however, it is assumed herein that the tendon does not include the anchorage. The sheath is a smooth or corrugated pipe or tube that protects the prestressing steel in the unbonded length from corrosion. Centralizers position the tendon in the drill hole such that the specified minimum grout cover is achieved around the tendon. For multiple element tendons, spacers are used to separate the strands or bars of the tendons so that each element is adequately bonded to the anchor grout. The grout is a Portland cement based mixture that provides load transfer from the tendon to the ground and provides corrosion protection for the tendon (Sabatini et al. 1999).

3.3 TYPES OF GROUND ANCHORS

There are three main ground anchor types that are currently used in practice:

(1) straight shaft gravity-grouted ground anchors (Type A);
(2) straight shaft pressure-grouted ground anchors (Type B);
(3) post-grouted ground anchors (Type C).

Although not commonly used today in practice, another type of anchor is the underreamed anchor (Type D). These ground anchor types are illustrated schematically in figure 3.3 and are briefly described in the following sections.

Drilling methods for each of the three main soil and rock ground anchors include rotary, percussion, rotary/percussive, or auger drilling. Detailed information on these drilling techniques may be found in Bruce (1989). The procedures and methods used to drill holes for ground anchors are usually selected by the contractor. The choice of a particular drilling method must also consider the overall site conditions and it is for this reason that the engineer may place limitations on the drilling method.

The drilling method must not adversely affect the integrity of structures near the ground anchor locations or on the ground surface. With respect to drilling, excessive ground loss into the drill hole and ground surface heave are the primary causes of damage to these structures. For example, the use of large diameter hollow stem augered anchors should be discouraged in sands and gravels since the auger will tend to remove larger quantities of soil from the drill hole as compared to the net volume of the auger. This may result in loss of support of the drill hole. In unstable soil or rock, drill casing is used. Water or air is used to flush the drill cuttings out of the cased hole. Caution should be exercised when using air flushing to clean the hole. Excess air pressures may result in unwanted removal of groundwater and fines from the drill hole leading to potential hole collapse or these excess pressures may result in ground heave (Sabatini et al. 1999).

3.3.1 Straight Shaft Gravity-Grouted Ground Anchors

Straight shaft gravity-grouted ground anchors are typically installed in rock and very stiff to hard cohesive soil deposits using either rotary drilling or hollow-stem auger methods. Tremie (gravity displacement) methods are used to grout the anchor in a straight shaft borehole. The borehole may be cased or uncased depending on the stability of the borehole. Anchor resistance to pullout of the grouted anchor depends on the shear resistance that is mobilized at the grout/ground interface.

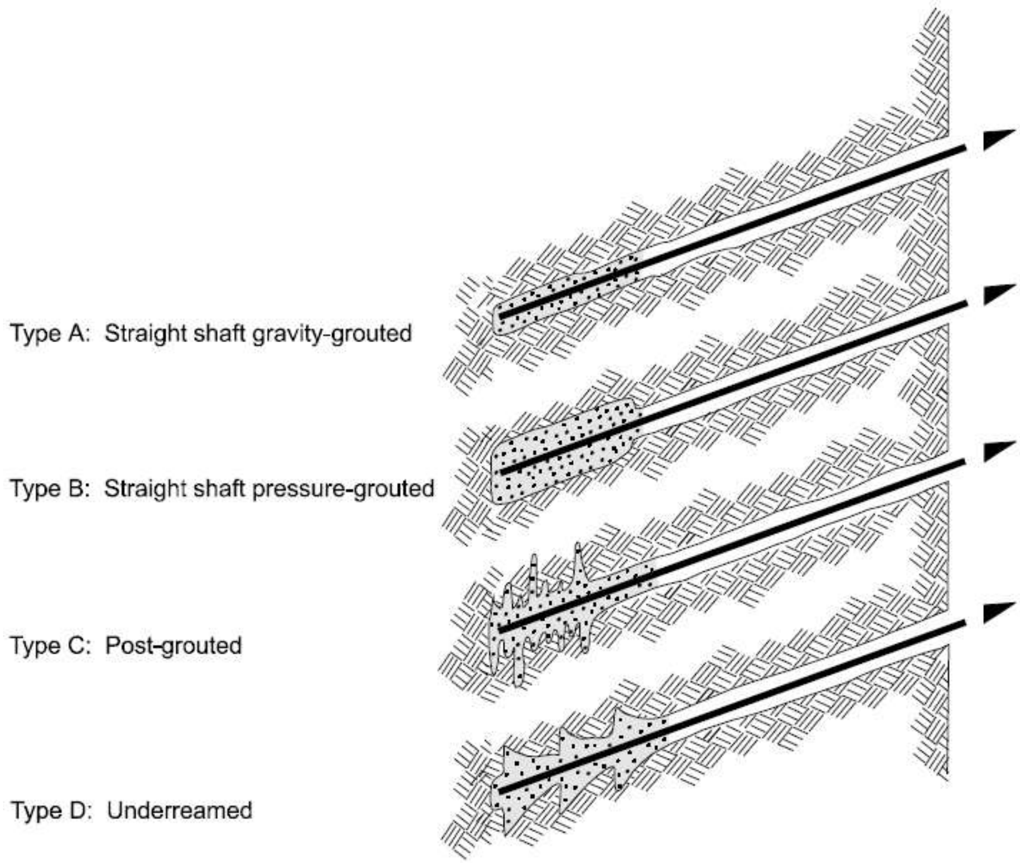


Figure 3.3 *Main types of grouted ground anchors (modified after Littlejohn, 1990)*

3.3.2 Straight Shaft Pressure-Grouted Ground Anchors

Straight shaft pressure-grouted ground anchors are most suitable for coarse granular soils and weak fissured rock. This anchor type is also used in fine grained cohesionless soils. With this type of anchor, grout is injected into the bond zone under pressures greater than 0.35 MPa. The borehole is typically drilled using a hollow stem auger or using rotary techniques with drill casings. As the auger or casing is withdrawn, the grout is injected into the hole under pressure until the entire anchor bond length is grouted. This grouting procedure increases resistance to pullout relative to tremie grouting methods by:

(1) increasing the normal stress (i.e., confining pressure) on the grout bulb resulting from compaction of the surrounding material locally around the grout bulb;
(2) increasing the effective diameter of the grout bulb.

3.3.3 Post-grouted Ground Anchors

Post-grouted ground anchors use delayed multiple grout injections to enlarge the grout body of straight shafted gravity grouted ground anchors. Each injection is separated by one or two days. Postgrouting is accomplished through a sealed grout tube installed with the tendon. The tube is equipped with check valves in the bond zone. The check valves allow additional grout to be injected under high pressure into the initial grout which has set. The high pressure grout fractures the initial grout and wedges it outward into the soil enlarging the grout body. Two fundamental types of postgrouted anchors are used. One system uses a packer to isolate each valve. The other system pumps the grout down the post-grout tube without controlling which valves are opened.

3.3.4 Underreamed Anchors

Underreamed anchors consist of tremie grouted boreholes that include a series of enlargement bells or underreams. This type of anchor may be used in firm to hard cohesive deposits. In addition to resistance through side shear, as is the principal load transfer mechanism for other anchors, resistance may also be mobilized through end bearing. Care must be taken to form and clean the underreams.

3.4 TENDON MATERIALS

3.4.1 Steel Bar and Strand Tendons

Both bar and strand tendons are commonly used for soil and rock anchors for highway applications in the U.S. Material specifications for bar and strand tendons are codified in American Society for Testing and Materials (ASTM) A722 and ASTM A416, respectively. Indented strand is codified in ASTM A886. Bar tendons are commonly available in 26 mm, 32 mm, 36 mm, 45 mm, and 64 mm diameters in uncoupled lengths up to approximately 18 m. Anchor design loads up to approximately 2,077 kN can be resisted by a single 64-mm diameter bar tendon. For lengths greater than 18 m and where space constraints limit bar tendon lengths, couplers may be used to extend the tendon length. As compared to strand tendons, bars are easier to stress and their load can be adjusted after lock-off.
Strand tendons comprise multiple seven-wire strands. The common strand in U.S. practice is 15 mm in diameter. Anchors using multiple strands have no practical

load or anchor length limitations. Tendon steels have sufficiently low relaxation properties to minimize long-term anchor load losses. Couplers are available for individual seven-wire strands but are rarely used since strand tendons can be manufactured in any length. Strand couplers are not recommended for routine anchor projects as the diameter of the coupler is much larger than the strand diameter, but strand couplers may be used to repair damaged tendons. Where couplers are used, corrosion protection of the tendon at the location of the coupler must be verified.

3.4.2 Spacers and Centralizers

Spacer/centralizer units are placed at regular intervals (e.g., typically 3 m) along the anchor bond zone. For strand tendons, spacers usually provide a minimum interstrand spacing of 6 to 13 mm and a minimum outer grout cover of 13 mm. Both spacers and centralizers should be made of noncorrosive materials and be designed to permit free flow of grout. Figure 3.4 shows a cut away section of a bar and a strand tendon, respectively.

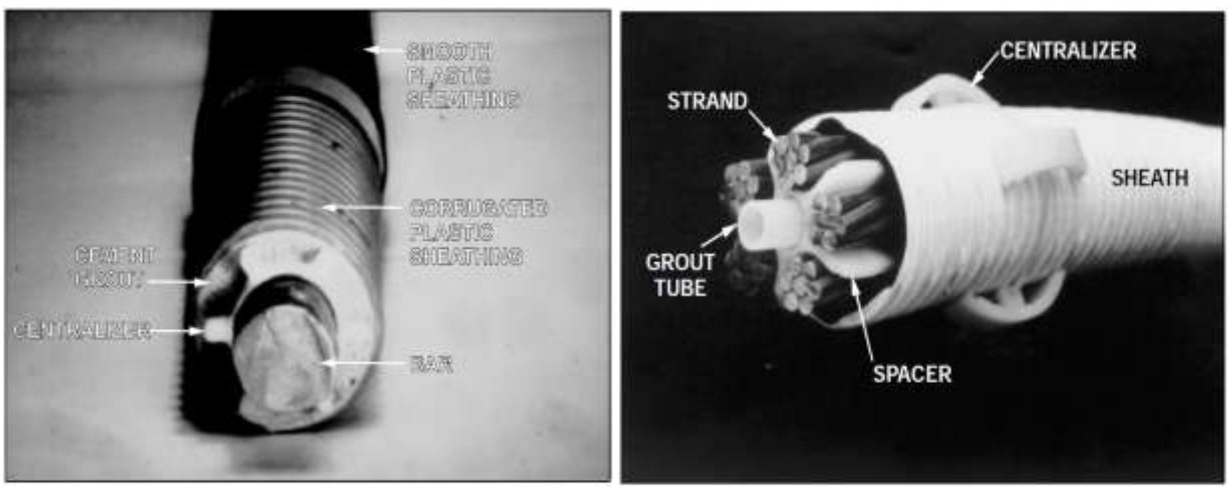


Figure 3.4 *Cut away view of bar tendon (left) and strand tendon (right)*

3.4.3 Epoxy-Coated Bar and Epoxy-Coated Filled Strand

Epoxy-coated bar (AASHTO M284) and epoxy-coated filled strand (supplement to ASTM A882), while not used extensively for highway applications, are becoming more widely used for dam tiedown projects. The epoxy coating provides an additional layer of corrosion protection in the unbonded and bond length as compared to bare prestressing steel.

For epoxy-coated filled strand, in addition to the epoxy around the outside of the strand, the center wire of the seven-wire strand is coated with epoxy. Unfilled epoxy-coated strand is not recommended because water may enter the gaps around the center wire and lead to corrosion. Unlike bare strand, creep deformations of epoxy-coated filled strands themselves are relatively significant during anchor testing. When evaluating anchor acceptance with respect to creep, the creep of the epoxy-coated filled strands themselves must be deducted from the total creep movements to obtain a reliable measurement of the movements in the bond zone.

3.4.4 Other Anchor Types and Tendon Materials

In addition to cement grouted anchors incorporating high strength prestressing steels, alternative anchor types and tendon materials are used in the U.S. Examples include Grade 60 and Grade 75 grouted steel bars, helical anchors, plate anchors, and mechanical rock anchors.

Research on the use of fiber reinforced plastic (FRP) prestressing tendons is currently being performed (e.g., Schmidt et al., 1994). FRP tendons have high tensile strength, are corrosion resistant, and are lightweight. These products, however, are not used in current U.S. construction practice. Other materials such as fiberglass and stainless steel have been used experimentally but cost and/or construction concerns have restricted widespread use.

3.4.5 Cement Grout

Anchor grout for soil and rock anchors is typically a neat cement grout (i.e., grout containing no aggregate) conforming to ASTM C150 although sand-cement grout may also be used for large diameter drill holes. Pea gravel-sand-cement grout may be used for anchor grout outside the tendon encapsulation. High speed cement grout mixers are commonly used which can reasonably ensure uniform mixing between grout and water. A water/cement (w/c) ratio of 0.4 to 0.55 by weight and Type I cement will normally provide a minimum compressive strength of 21 MPa at the time of anchor stressing. For some projects, special additives may be required to improve the fluid flow characteristics of the grout. Admixtures are not typically required for most applications, but plasticizers may be beneficial for applications in high temperature and for long grout pumping distances.

3.5 ANCHORED WALLS

A common application of ground anchors for highway projects is for the construction of anchored walls used to stabilize excavations and slopes. These anchored walls consist of nongravity cantilevered walls with one or more levels of ground anchors. Nongravity cantilevered walls employ either discrete (e.g., soldier beam) or continuous (e.g., sheet-pile) vertical elements that are either driven or drilled to depths below the finished excavation grade. For nongravity cantilevered walls, support is provided through the shear and bending stiffness of the vertical

wall elements and passive resistance from the soil below the finished excavation grade. Anchored wall support relies on these components as well as lateral resistance provided by the ground anchors to resist horizontal pressures (e.g., earth, water, seismic, etc.) acting on the wall.
Various construction materials and methods are used for the wall elements of an anchored wall. Discrete vertical wall elements often consist of steel piles or drilled shafts that are spanned by a structural facing. Permanent facings are usually cast-in-place (CIP) concrete although timber lagging or precast concrete panels have been used. Continuous wall elements do not require separate structural facing and include steel sheet-piles, CIP or precast concrete wall panels constructed in slurry trenches (i.e., slurry (diaphragm) walls), tangent/secant piles, soil-cement columns, and jet grouted columns.

3.5.1 Soldier Beam and Lagging Wall

Soldier beam and lagging walls are the most commonly used type of anchored wall system in the U.S. This wall system uses discrete vertical wall elements spanned by lagging which is typically timber, but which may also be reinforced shotcrete.

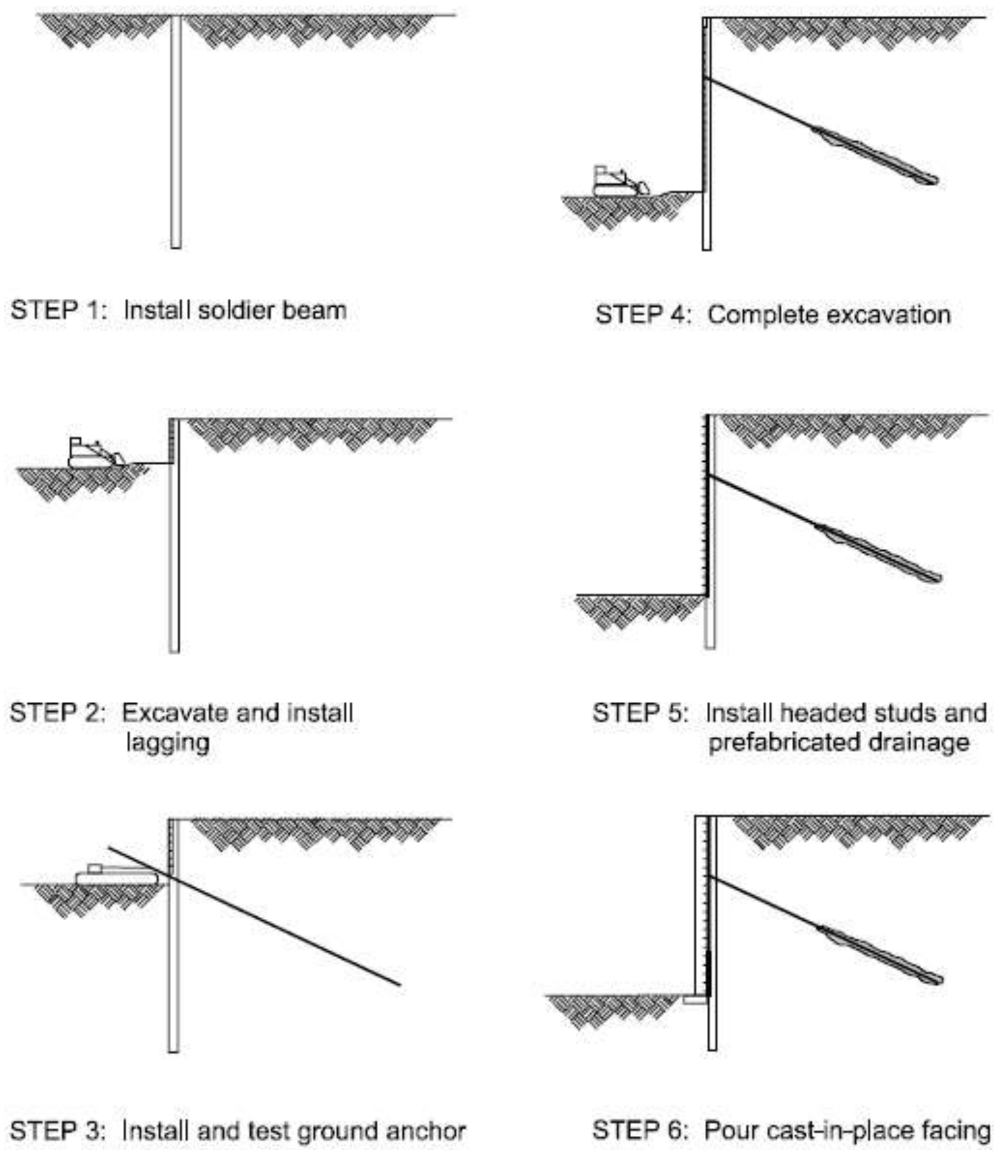


Figure 3.5 *Construction sequence for permanent soldier beam and lagging wall.*

These wall systems can be constructed in most ground types; however, care must be exercised in grounds such as cohesionless soils and soft clays that may have limited "stand-up" time for lagging installation. These wall systems are also highly pervious. The construction sequence for a permanent soldier beam and lagging wall is illustrated in figure 3.5 and is described below.

3.5.1.1 Soldier Beam

The initial step of construction for a soldier beam and lagging wall consists of installing the soldier beams from the ground surface to their final design elevation.

Horizontal spacing of the soldier beams typically varies from 1.5 to 3 m. The soldier beams may be steel beams or drilled shafts, although drilled shafts are seldom used in combination with timber lagging.

3.5.1.2 Drilled-in Soldier Beams

Steel beams such as wide flange (WF) sections or double channel sections may be placed in excavated holes that are subsequently backfilled with concrete. It is recommended that the excavated hole be backfilled with either structural or lean-mix concrete from the bottom of the hole to the level of the excavation subgrade. The selection of lean-mix or structural concrete is based on lateral and vertical capacity requirements of the embedded portion of the wall. From the excavation subgrade to the ground surface, the hole should be backfilled with lean-mix concrete that is subsequently scraped off during lagging and anchor installation. Structural concrete is not recommended to be placed in this zone because structural concrete is extremely difficult to scrape off for lagging installation. Lean-mix concrete typically consists of one 94 lb bag of Portland cement per cubic yard of concrete and has a compressive strength that does not typically exceed approximately 1 MPa. As an alternative to lean-mix concrete backfill, controlled low strength material (CLSM) or "flowable fill" may be used. This material, in addition to cement, contains fine aggregate and fly ash. When allowing lean-mix concrete or CLSM for backfilling soldier beam holes, contract specifications should require a minimum compressive strength of 0.35 MPa. Like lean-mix concrete, CLSM should be weak enough to enable it to be easily removed for lagging installation.

Ground anchors are installed between the structural steel sections and the distance between the sections depends upon the type of ground anchor used. Drill hole diameters for the soldier beams depend upon the structural shape and the diameter of the anchor. Replacement anchors can be installed between the structural sections at any location along the soldier beam. The ground anchor to soldier beam connection for drilled-in soldier beams can be installed on the front face of the structural sections or between the sections. For small diameter ground anchors, the connection may be prefabricated before the soldier beams are installed. The connections for large-diameter anchors are made after the anchors have been installed.

3.5.1.3 Driven Soldier Beams

Steel beams such as HP shapes or steel sheet piles are used for driven soldier beams. Driven soldier beams must penetrate to the desired final embedment depth without significant damage. Drive shoes or "points" may be used to improve the ability of the soldier beams to penetrate a hard stratum. High strength steels also improve the ability of the soldier beams to withstand hard driving. If the soldier beams cannot penetrate to the desired depth, then the beams should be drilled-in. Thru-beam connections or horizontal wales are used to connect ground anchors to driven soldier beams.

A thru-beam connection is a connection cut in the beam for a small diameter ground anchor. Thrubeam connections are usually fabricated before the beam is driven. This type of connection is designed so the ground anchor load is applied at the center of the soldier beam in line with the web of the soldier beam. Large-diameter (i.e., greater than approximately 150 mm) ground anchors cannot be used with thru-beam connections. Thru-beam connections are used when few ground anchor failures are anticipated because when a ground anchor fails, the failed anchor has to be removed from the connection or a new connection has to be fabricated. A "sidewinder connection" may be used with a replacement anchor for a temporary support of excavation wall, but it is not recommended for a permanent wall. A sidewinder connection is offset from the center of the soldier beam, and the ground anchor load is applied to the flange some distance from the web. Sidewinder connections subject the soldier beams to bending and torsion.

Horizontal wales may be used to connect the ground anchors to the driven soldier beams. Horizontal wales can be installed on the face of the soldier beams, or they can be recessed behind the front flange. When the wales are placed on the front flange, they can be exposed or embedded in the concrete facing. If the wales remain exposed, then the ground anchor tendon corrosion protection may be exposed to the atmosphere and it is therefore necessary that the corrosion protection for the anchorage be well designed and constructed. However, since exposed wales are unattractive and must be protected from corrosion, they are not recommended for permanent anchored walls. Wales placed on the front face of the soldier beams require a thick cast-in-place concrete facing. Wales can be recessed to allow a normal thickness concrete facing to be poured. Recessed wales must be individually fabricated and the welding required to install them is difficult and expensive. If a wale is added during construction, the horizontal clear distance to the travel lanes should be checked before approval of the change (Sabatini et al. 1999).

3.5.1.4 Lagging

After installation of the soldier beams, the soil in front of the wall is excavated in lifts, followed by installation of lagging. Excavation for lagging installation is commonly performed in 1.2 to 1.5 m lifts; however, smaller lift thicknesses may be required in ground that has limited "stand-up" time. Lagging should be placed

from the top-down as soon as possible after excavation to minimize erosion of materials into the excavation. Prior to lagging installation, the soil face should be excavated to create a reasonably smooth contact surface for the lagging. Lagging may be placed either behind the front flange of the soldier beam or on the soldier beam. Lagging placed behind soldier beam flanges is cut to approximate length, placed in-between the flanges of adjacent soldier beams, and secured against the soldier beam webs by driving wood wedges or shims. Lagging can also be attached to the front flange of soldier beams with clips or welded studs. In rare circumstances, lagging can be placed behind the back flange of the soldier beam. With either lagging installation method, gaps between the lagging and the retained ground must be backpacked to ensure good contact. Prior to placing subsequent lagging a spacer, termed a "louver", is nailed to the top of the lagging board at each end of the lagging. This louver creates a gap for drainage between vertically adjacent lagging boards. The size of the gap must be sufficiently wide to permit drainage, while at the same time disallowing the retained soil to fall out from behind the boards. Typically, placing vertically adjacent lagging boards in close contact is considered unacceptable; however, some waterproofing methods may require that the gap between the lagging boards be eliminated. In this case, the contractor must provide an alternate means to provide drainage.
Concrete lagging has been used, but its use may be problematic due to difficulties in handling and very tight tolerances on the horizontal and vertical positioning of the soldier beam to ensure easy installation of standard length concrete lagging. Trimming of concrete lagging is very difficult and field splicing is not possible. Also, the concrete lagging near the anchor location may crack during anchor testing or stressing.

3.5.1.5 Construction Sequence

Top-down installation of lagging continues until the excavation reaches a level of approximately 0.6 m below the design elevation of a ground anchor. At this point, the excavation is halted and the ground anchor is installed. Deeper excavation (i.e., greater than 0.6 m) below the level of a ground anchor may be required to allow the anchor connection to be fabricated or to provide equipment access. The wall must be designed to withstand stresses associated with a deeper excavation. The anchor is installed using appropriate drilling and grouting procedures, as previously described. When the grout has reached an appropriate minimum strength, the anchor is load tested and then locked-off at an appropriate load. Excavation and lagging installation then continues until the elevation of the next anchor is reached and the next anchor is installed. This cycle of excavation, lagging installation, and ground anchor installation is continued until the final excavation depth is reached.

When the excavation and lagging reach the final depth, prefabricated drainage elements may be placed at designed spacings and connected to a collector at the base of the wall. The use of shotcrete in lieu of timber lagging can be effective in certain situations. However, since the shotcrete is of low permeability, drainage must be installed behind the shotcrete.

3.5.2 Continuous Walls

Ground anchors are also used in continuous wall systems such as sheet-pile walls, tangent or secant pile walls, slurry walls, or soil mixed walls. Continuous walls are commonly used for temporary excavation support systems. Sheet-pile walls are constructed in one phase in which interlocking sheet-piles are driven to the final design elevation. Where difficult driving conditions are encountered, a template is often utilized to achieve proper alignment of the sheet-piles, however, it should be recognized that these wall systems may not be feasible for construction in hard ground conditions or where obstructions exist. Interlocking sheet-piles may be either steel or precast concrete, however, steel sheet-piles are normally used due to availability and higher strength than precast concrete sheet-piles.

Unlike soldier beam and lagging walls, continuous walls act as both vertical and horizontal wall elements. Cycles of excavation and anchor installation proceed from the top of the excavation and then between the level of each anchor. Because of the relative continuity of these wall systems, water pressure behind continuous walls must be considered in design. In cases where the continuous wall must resist permanent hydrostatic forces, a watertight connection must be provided at the ground anchor/wall connection.

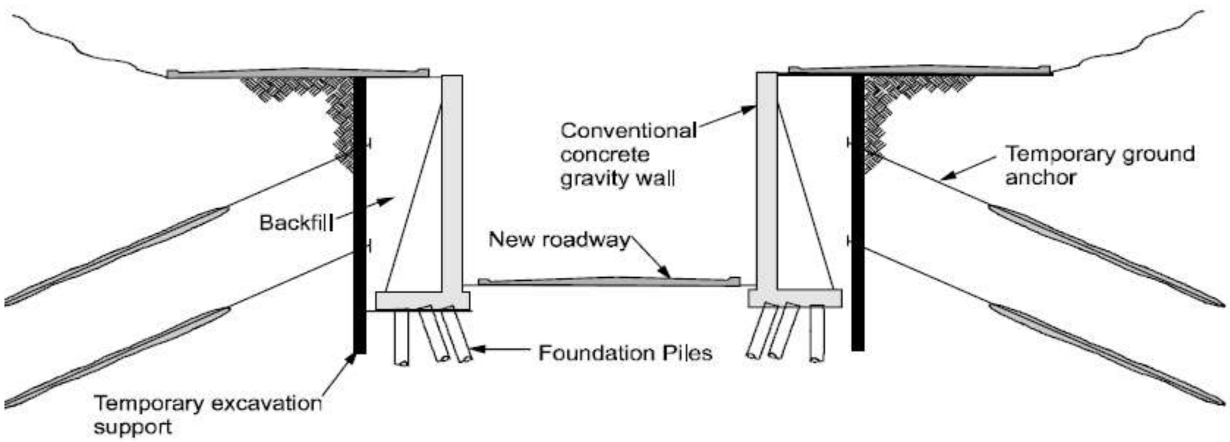
3.6 APPLICATIONS OF GROUND ANCHORS

3.6.1 Highway Retaining Walls

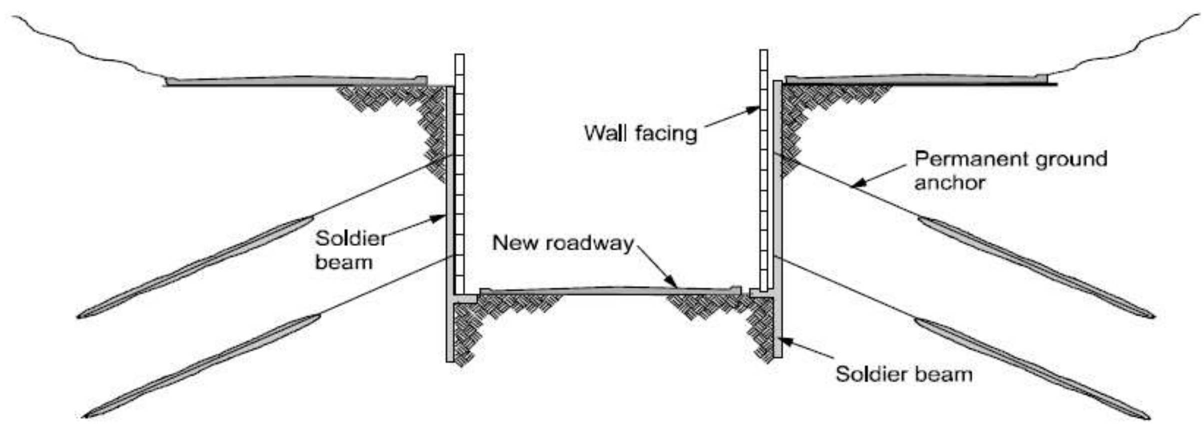
Anchored walls are commonly used for grade separations to construct depressed roadways, roadway widenings, and roadway realignments. Figure 3.6 provides a comparative illustration of a conventional concrete gravity wall and a permanent anchored wall for the construction of a depressed roadway. The conventional gravity wall is more expensive than a permanent anchored wall because it requires temporary excavation support, select backfill, and possibly deep foundation support. Anchored walls may also be used for new bridge abutment construction and end slope removal for existing bridge abutments (FHWA-RD-97-130, 1998).

3.6.2 Slope and Landslide Stabilization

Ground anchors are often used in combination with walls, horizontal beams, or concrete blocks to stabilize slopes and landslides. Soil and rock anchors permit relatively deep cuts to be made for the construction of new highways (figure 9a). Ground anchors can be used to provide a sufficiently large force to stabilize the mass of ground above the landslide or slip surface (figure 9b).



(a) Conventional Concrete Gravity Wall



(b) Permanent Anchored Soldier Beam and Lagging Wall

Figure 3.6 *Comparison of concrete gravity wall and anchored wall for a depressed roadway.*

This force may be considerably greater than that required to stabilize a vertical excavation for a typical highway retaining wall. Horizontal beams or concrete blocks may be used to transfer the ground anchor loads to the ground at the slope surface provided the ground does not "run" or compress and is able to resist the anchor reaction forces at the excavated face. Cost, aesthetics, and long-term maintenance of the exposed face will affect the selection of horizontal beams or blocks.

3.6.3 Tiedown Structures

Permanent ground anchors may be used to provide resistance to vertical uplift forces. Vertical uplift forces may be generated by hydrostatic or overturning forces. The method is used in underwater applications where the structure has insufficient dead weight to counteract the hydrostatic uplift forces. An example application of ground anchors to resist uplift forces is shown in figure 3.7 c. The advantage of ground anchors for tiedown structures include:

(1) the volume of concrete in the slab is reduced compared to a dead weight slab;
(2) excavation and/or dewatering is reduced.

Disadvantages of ground anchors for tiedowns include:

(1) potentially large variations in ground anchor load resulting from settlement and heave of the structure;
(2) difficulty in constructing watertight connections at the anchor-structural slab interface, which is particularly important for hydrostatic applications;
(3) variations in stresses in the slab.

A major uplift slab that incorporated tiedowns was constructed for the Central Artery Project in Boston, Massachusetts (Druss, 1994).

Although not a highway application, permanent rock anchor tiedowns may be used to stabilize concrete dams (figure 3.7 d). Existing dams may require additional stabilization to meet current safety standards with respect to maximum flood and earthquake requirements. Anchors provide additional resistance to overturning, sliding, and earthquake loadings.

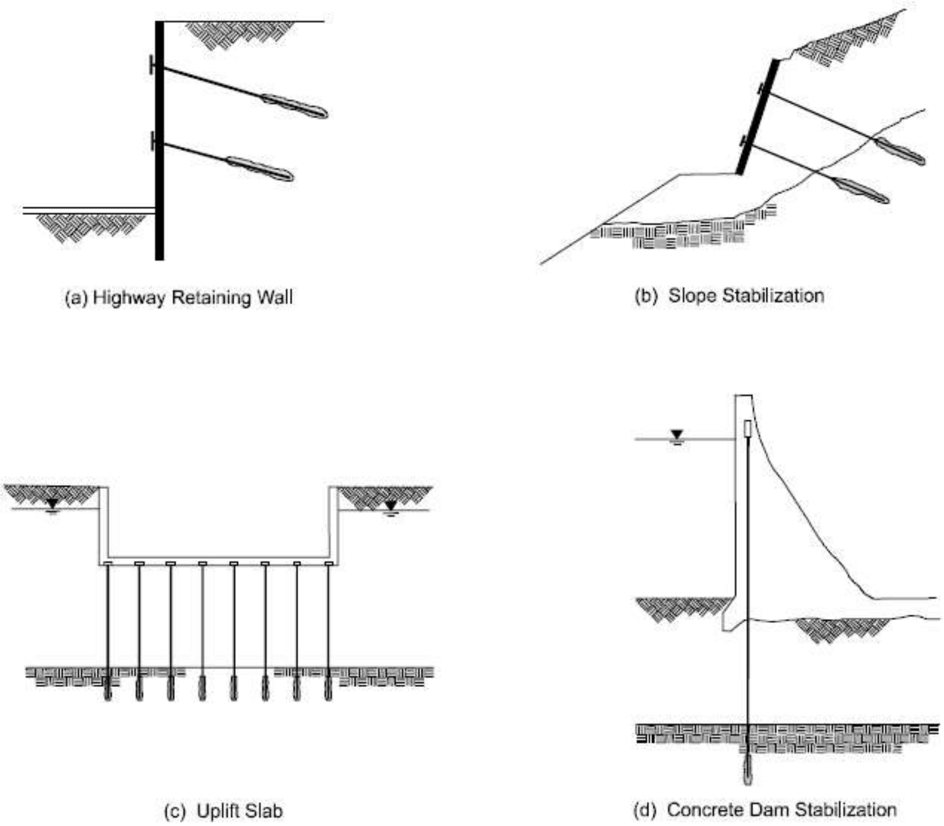


Figure 3.7 *Applications of ground anchors and anchored systems.*

3.7 GENERAL DESIGN CONCEPTS FOR ANCHORED WALLS

The concept of an anchored wall system is to create an internally stable mass of soil that will resist external failure modes at an adequate level of serviceability. The design of anchored walls concentrates on achieving a final constructed wall that is secure against a range of potential failure conditions. These conditions are illustrated in figure 3.8. The design should limit movements of the soil and the wall while providing a practical and economical basis for construction. The design should consider the mobilization of resistance by both anchors and wall elements in response to loads applied to the wall system.

The magnitude of the total anchor force required to maintain the wall in equilibrium is based on the forces caused by soil, water, and external loads. Anchors can provide the required stabilizing forces which, in turn, are transmitted back into the soil at a suitable distance behind the active soil zone loading the wall, as illustrated in figure 3.9a. This requirement that the anchor forces must be transmitted behind the active zone generally defines the minimum distance behind the wall at which the anchor bond length is formed.

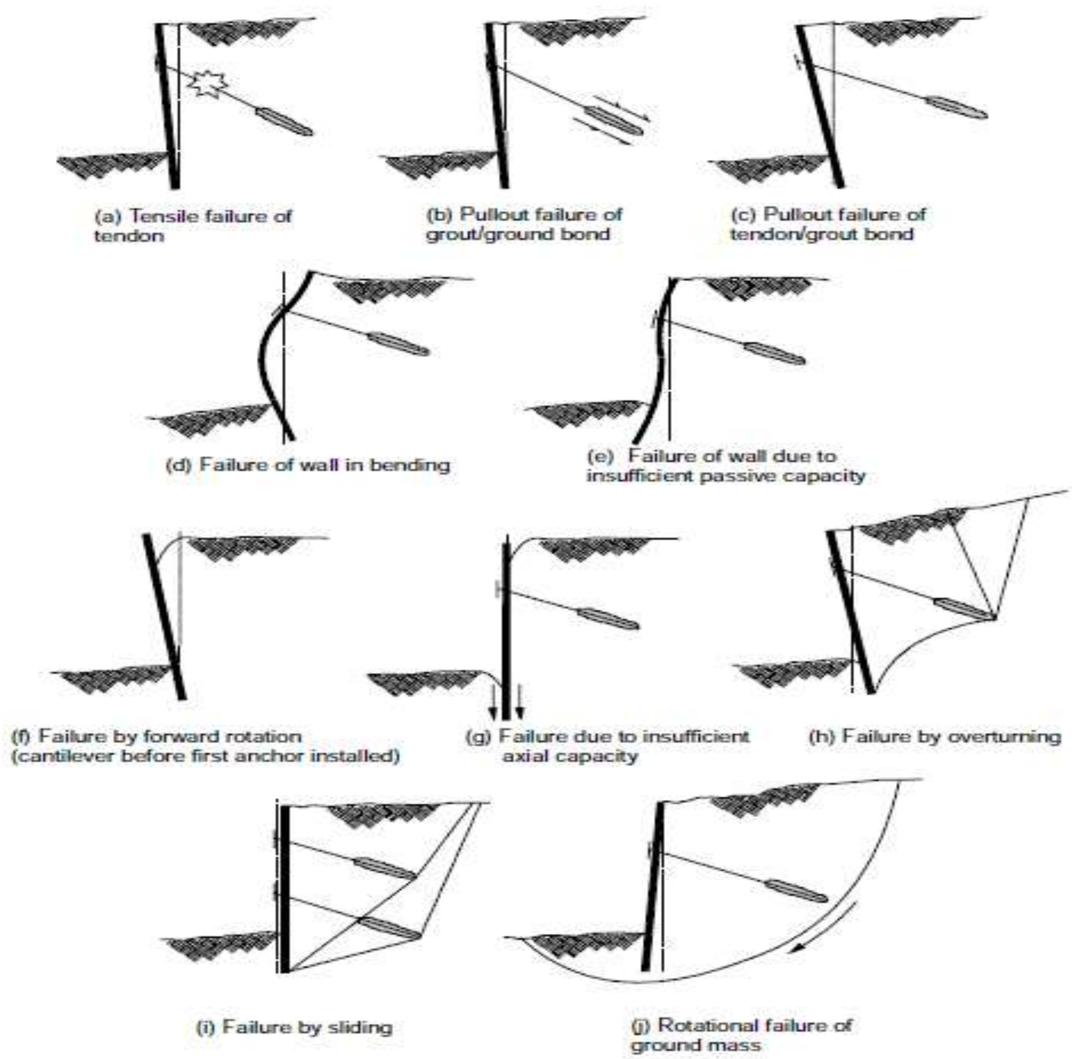


Figure 3.8 *Potential failure conditions to be considered in design of anchored walls.*

The anchor bond length must extend into the ground to intersect any potentially critical failure surfaces which might pass behind the anchors and below the base of the wall as illustrated in figure 3.9b. The required depth to which anchors must be installed in the soil should be determined based on the location of the deepest potential failure surfaces that have an insufficient factor of safety without any anchor force.

3.8 FAILURE MECHANISMS OF ANCHORED SYSTEMS

Many different types of anchored systems can usually fulfill the needs of a particular project. To achieve maximum economy, the objective of the designer is to specify only those parameters that are necessary for long-term stability of the anchored system and to leave the final selection of the anchor details to the contractor. Anchor system performance is evaluated by testing each installed anchor at loads that exceed the design load. To determine the parameters that should be specified, the designer must consider various possible failure mechanisms.

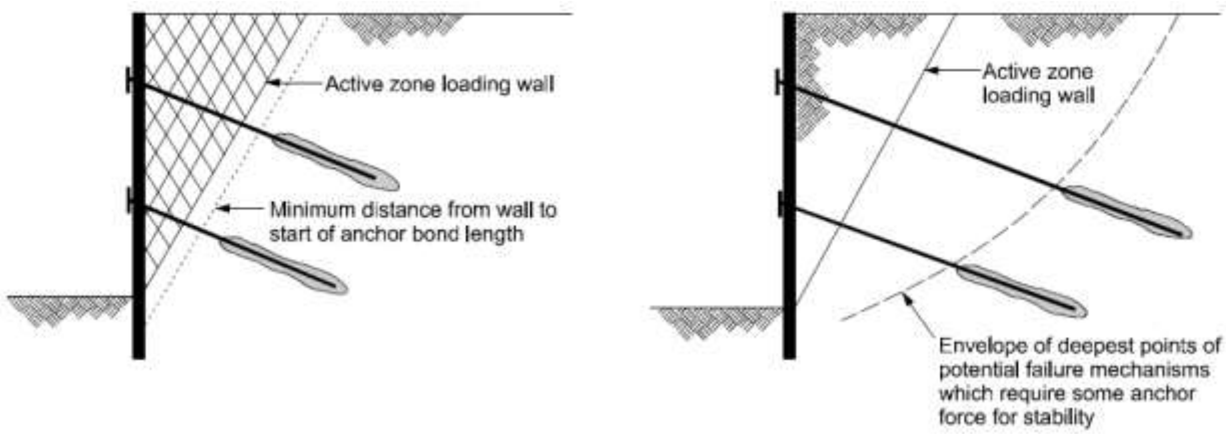


Figure 3.9 *Contribution of ground anchors to wall stability.*

3.8.1 Failure Mechanisms of the Ground Anchor

There are several possible failure mechanisms of ground anchors. These are usually caused by excessive static loading of an anchor. Excessive loads can be related to:

(1) tension placed in the anchor during load testing or at lock-off;
(2) excavation sequence;
(3) surcharge by construction materials or equipment;
(4) construction of adjacent structures; or
(5) a combination of these causes.

Ground anchor failure mechanisms may involve the steel tendon, the ground mass, the ground-grout zone, and the grout-tendon zone, as described subsequently.

Failure of the Steel Tendon

As the anchor is loaded, the steel tendon component of the anchor is stressed in tension. If the applied load is greater than the structural capacity of the tendon, failure is inevitable. Therefore, a factor of safety must be used with respect to structural failure of the steel. It is recommended that the tendon load not exceed 60 percent of the specified minimum tensile strength (SMTS) for final design and 80 percent of SMTS for temporary loading conditions (e.g., loading during testing).

Failure of the Ground Mass

Failure of the soil mass, as referred to herein, involves failure resulting from anchor loads, not external forces such as landslides which potentially introduce excessive static loading to the anchor. For shallow soil anchors, failure of the ground mass is characterized by uplift of a mass of soil in front of the anchor bond zone followed by pullout of the bond zone. A shear surface develops in the soil mass ahead of the anchor as increasing stresses cause complete mobilization of resistance in the anchor bond zone. The failure surface simulates a passive earth pressure failure. Practically, failure of the soil mass is not a factor for anchors embedded more than 4.5 m below the ground surface.

For rock anchors, the likely plane of failure for shallow installations in sound bedrock is along a cone generated at approximately a 45 degree angle from the anchorage. In fractured or bedded rock, the cone shape and size varies with the distribution of bedding and cleavage planes and the grout take in fissures. Even in fractured rock, rock mass failure seldom occurs in anchors embedded more than 4.5 m below ground because the bond strength between the rock and grout or the grout and tendon is much less than the rock strength.

Failure of the Ground-Grout Bond

Ground anchors mobilize skin friction between the anchor bond zone and the ground. In general, this bond is dependent on the normal stress acting on the bond zone grout and the adhesion and friction mobilized between the ground and the grout. Anchors which are underreamed may also develop the base resistance of the increased annular area.

In general, the ground-grout bond is mobilized progressively in uniform soil or rock as the stress is transferred along the bond length. Initially, as the anchor is stressed, the portion of the bond length nearest the load application elongates and transfers load to the ground. As the resistance in this portion of the bond length is mobilized, stress is transferred farther down. During this process, the anchor continues to elongate to mobilize deeper bond zones. Once the stress is transferred to the end of the bond zone and the ultimate ground-grout bond is exceeded, anchor failure by pullout occurs. Anchors which have been improperly grouted such that a column of grout exists between the bearing plate or wall and the top of the bond zone will show no load transfer into the bond length when the load is increased. Factors influencing stress transfer for small diameter ground anchors with bond lengths in a uniform soil are summarized in table 3.1.

Experience has shown that increasing the bond length for typical soil anchors beyond 9 to 12 m does not result in significant increases in resistance. A possible reason for this observation is that after the load has been transferred that distance down the bond zone, sufficient movement at the ground-grout interface has occurred in the upper bond length to decrease the upper ground-grout interface resistance to residual strength levels. Bond lengths greater than 12 m may be used effectively provided special procedures are used to bond the tendon to the grout such that capacity can be mobilized along the longer length.

Table 3.1 *Typical factors influencing bond stress transfer for small diameter ground anchors*

	Soil Type	
Factor	Cohesionless	Cohesive
Soil Properties	Friction angle and grain size distribution.	Adhesion and plasticity index.
Drilling Method	Driven casing increases normal stress and friction.	Drilling without casing or with fluids decreases capacity.
Bond Length	Steady increase in anchor capacity to 6 m with moderating increases to 12 m.	Steady increase in anchor capacity for soils with undrained strength less than 96 kPa.
Hole Diameter	Slight increase in anchor capacity to 100 mm.	Anchor capacity increases to 300 mm.
Grout Pressure	Anchor capacity increases with increasing pressure.	Anchor capacity increases only with stage grouting. High initial pressures should be avoided.

Note: To ensure ground-grout bond, the drill hole should be cleaned and the grout should be placed as quickly as possible after the hole has been drilled.

Failure at the ground-grout interface may also be characterized by excessive deformations under sustained loading (i.e., creep). Soil deposits that are potentially susceptible to excessive creep deformations include:

(1) organic soils;
(2) clay soils with an average liquidity index (LI) greater than 0.2;
(3) clay soils with an average liquid limit (LL) greater than 50;
(4) clay soils with an average plasticity index (PI) greater than 20.

Conservative anchor design loads and working bond stress values are recommended for design involving permanent anchor installations in such soils, unless based on results from a predesign or preproduction test program.

The LL, plastic limit (PL) and moisture content (wn) of a clay soil are commonly measured clayey soil index properties. The LI indicates where the moisture content of the clay falls within the range between the plastic and liquid limits. Liquidity index for a soil is defined as:

$$LI = \frac{W_n - PL}{PI} \tag{3.1}$$

A low LI indicates that the moisture content is relatively close to the PL of the soil, indicating a potentially overconsolidated or stiff soil. A LI close to 1.0 indicates that the moisture content is relatively close to the LL for the soil, indicating a potentially normally consolidated or soft soil.

Failure of Grout-Tendon Bond

The bond between the grout and steel tendon must not be exceeded if the full strength of the supporting ground is to be mobilized. The failure mechanism of the grout-tendon bond involves three components:

(1) adhesion;
(2) friction; and
(3) mechanical interlock.

Adhesion is the physical coalescence of the microscopically rough steel and the surrounding grout. This initial bond is replaced by friction after movement occurs. The friction depends on the roughness of the steel surface, the normal stress, and the magnitude of the slip. Mechanical interlock consists of the grout mobilizing its shear strength against major tendon irregularities such as ribs or twists. This interlock is the dominant bond mechanism for threadbars where the ultimate strength of the bar may be developed in a short embedment in the grout. The grout-tendon bond on smooth steel tendons is mobilized progressively in a fashion similar to the ground-grout bond. "Slip" occurs only after the maximum intensity of grout-tendon bond resistance has been mobilized over nearly the total bond length. After this slip, the tendon will only offer frictional resistance (amounting to about half the maximum total resistance obtained) to further elongation. Experience has shown that:

- Bond resistance of the grout to the tendon is not linearly proportional to the compressive strength of the grout. Although the bond strength usually increases as the compressive strength of the grout increases, the ratio of bond to ultimate strength decreases with increasing grout strengths. For example, a 17.2 MPa bond strength for 27.6 MPa grout may only increase by 12 percent to 19.3 MPa when the grout strength is increased by 25 percent to 34.5 MPa.
- Bond resistance developed by added embedment increases as the tendon length increases, but at reduced unit values.
- Flaky rust on bars lowers the bond, but wiping off the loosest rust produces a rougher surface which develops a bond equal to or greater than an unrusted

bar. Obviously pitted bars cannot be accepted even though the grout tendon bond may be adequate.

- The loose powdery rust appearing on bars after short exposures does not have a significant effect on grout-tendon bond.

Mill test reports should be requested by the owner for each lot used to fabricate the tendons. Test reports should include the results of bond capacity tests performed in accordance with the prestressing strand bond capacity test described in ASTM A981. ASTM A981 provides a standard test method to evaluate the bond strength between prestressing strand and cement grout. This specification was developed in 1997 in response to an industry initiative concerning the effects of certain residues from the manufacturing process that appeared to reduce the bond between the strand and the cement grout.

3.8.2 Failure of Soldier Beams

Soldier beams are subject to both lateral and vertical loads from the retained soil mass and the forces imparted from prestressing the anchors. The lateral resistance of the soldier beam is most critical during stressing and testing of the first anchor level, and for the final excavation condition when all wall loads have been applied. In the former case, stressing of the upper anchor to the test load is often done at shallow depths where the available passive resistance behind the soldier beam is low. Soldier beam deflections can be minimized in design by applying a safety factor of 1.5 to the passive resistance and in construction by ensuring that the upper lagging is tight against the soil and that the soil behind the soldier beam has not been removed. For the final excavation condition, the passive resistance in front of the wall must be adequate to restrain the toe of the soldier beam for long term wall loadings and for any future undercuts of the area in front of the wall.

Load transfer of the vertical loads on the soldier beams is more complex than for simple deep foundation elements. As the excavation for the wall deepens, vertical load is transferred above grade to the soil behind the back face of the soldier beam, but the magnitude of the load that is transferred is difficult to estimate. Theoretically, if adequate downward movement of the soldier beam (relative to the soil) occurs, load will be transferred to the soil mass behind the wall. However, this load transfer also results in the development of a negative interface wall friction angle for the active block of soil behind the wall resulting in an increase in the earth pressures behind the wall.

Vertical load capacity below the excavation base is calculated using common procedures for deep foundations (i.e., driven piles or drilled shafts). Two issues, however, are unique to evaluating axial capacity for soldier beam walls and must be considered. These issues are described below.

- Stress relief in front of the wall caused by excavation will reduce the effective stresses acting on the embedded portion of the soldier beam. This reduction in stress may vary with depth based on the width of the excavation. Common practice is to assume the effective stress is equal to the average of the effective stress imparted by the retained soil height behind the wall and by the depth of the soil in front of the wall.
- Structural sections are commonly placed in predrilled holes which are filled with concrete. In the case of a structural concrete filling, it is usually assumed that axial and lateral load are shared by the steel and the concrete and lateral capacity computations may be performed on the basis of the hole diameter. However, in the case of nonstructural (i.e., "lean-mix") concrete, the shear capacity between the structural section and the lean-mix concrete fill may not be adequate to provide load sharing between the steel and the concrete. This shear capacity should therefore be checked as part of the determination of axial and lateral soldier beam capacity (Sabatini et al. 1999).

3.8.3 Failure of Lagging

In general, the timber lagging is only used for support of temporary loads applied during excavation; however, pressure-treated timber lagging has been used to support permanent loads. The contribution of the temporary lagging is not included in the structural design of the final wall face. Temporary timber lagging is not designed by traditional methods; rather lagging is sized from charts developed based on previous project experience which accounts for soil arching between adjacent soldier beams (FHWA-RD-75-130, 1976).

3.9 EARTH PRESSURES

A wall system is designed to resist the lateral earth pressures and water pressures that develop behind the wall. Earth pressures develop primarily as a result of loads induced by weight of the retained soil, earthquake ground motions, and various surcharge loads. For purposes of anchored wall system design, three different lateral earth pressure conditions are considered:

(1) active earth pressure;
(2) passive earth pressure;
(3) at-rest earth pressure.

The distinction between actual ground behavior and conventional design assumptions is particularly important when considering earth pressures. The simple linear assumptions about active and passive pressures based on theoretical analyses are a considerable simplification of some very complex processes which depend on the following factors:

(1) the mode of wall movement (rotation, translation);
(2) wall flexibility;
(3) soil stiffness and strength properties;
(4) horizontal prestress in the ground;
(5) wall/soil interface friction.

For anchored wall systems with flexible wall elements, semi-empirical “apparent earth pressure envelopes” are commonly used.

3.9.1 Active and Passive Earth Pressure

Active and passive horizontal earth pressures may be considered in terms of limiting horizontal stresses within the soil mass, and, for purposes of this discussion, a smooth (i.e., zero wall friction) wall retaining ground with a horizontal backslope is considered (figure 3.10); this case defines Rankine conditions.

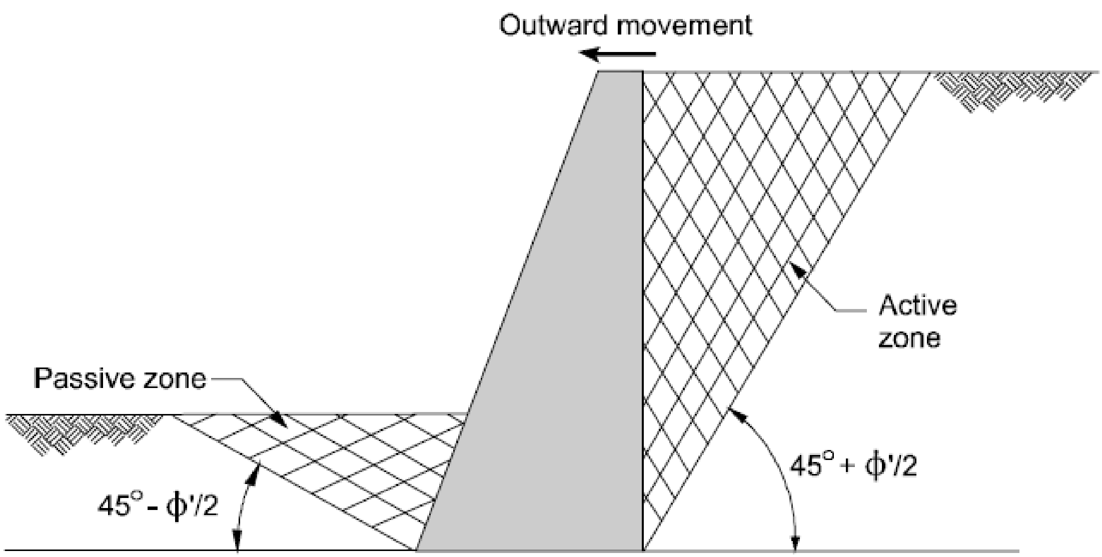


Figure 3.10 *Mobilization of Rankine active and passive horizontal pressures for a smooth retaining wall.*

Consider an element of soil in the ground under a vertical effective stress, σ'_v (figure 3.11). In considering the potential movements of a retaining wall, the element may be brought to failure in two distinct ways that are fundamentally important in the context of retaining wall design. The horizontal soil stress may be increased until the soil element fails at B, when the stress reaches its maximum value $\sigma'_{h(max)}$. This scenario will occur when significant outward movement of the wall increases the lateral earth pressure in the soil at the base of the wall (see figure

3.10). Similarly, the horizontal stress may be reduced until failure at A, when the stress reaches its minimum value $\sigma'_{h(min)}$. This scenario models the outward movement which reduces the lateral earth pressures behind the wall (see figure 3.10).

The geometry of figure 3.11 gives the following two relationships:

$$\frac{\sigma'_{h(min)}}{\sigma'_v} = K_A = \frac{1-\sin\varphi'}{1+\sin\varphi'} = \tan^2(45 - \frac{\varphi'}{2}) \quad (3.2)$$

$$\frac{\sigma'_{h(max)}}{\sigma'_v} = K_P = \frac{1+\sin\varphi'}{1-\sin\varphi'} = \tan^2(45 + \frac{\varphi'}{2}) \quad (3.3)$$

where K_A is the active earth pressure coefficient and K_P is the passive earth pressure coefficient. The definitions of K_A and K_P, based on equations 3.2 and 3.3, are consistent with a Rankine analysis for a cohesionless (i.e., c=0) retained soil.

For a cohesive soil defined by effective stress strength parameters φ′ and c′, the active and passive earth pressure coefficients are:

$$K_A = \tan^2\left(45 - \frac{\varphi'}{2}\right) - \frac{2c'}{\sigma'_v}\tan(45 - \frac{\varphi'}{2}) \quad (3.4)$$

$$K_P = \tan^2\left(45 + \frac{\varphi'}{2}\right) + \frac{2c'}{\sigma'_v}\tan(45 + \frac{\varphi'}{2}) \quad (3.5)$$

For the undrained case with φ = 0 and c = S_u, the total stress active and passive earth pressure coefficients are:

$$K_{AT} = 1 - \frac{2S_u}{\sigma'_v} \quad (3.6)$$

$$K_{PT} = 1 + \frac{2S_u}{\sigma'_v} \quad (3.7)$$

where σ_v is the total vertical stress.

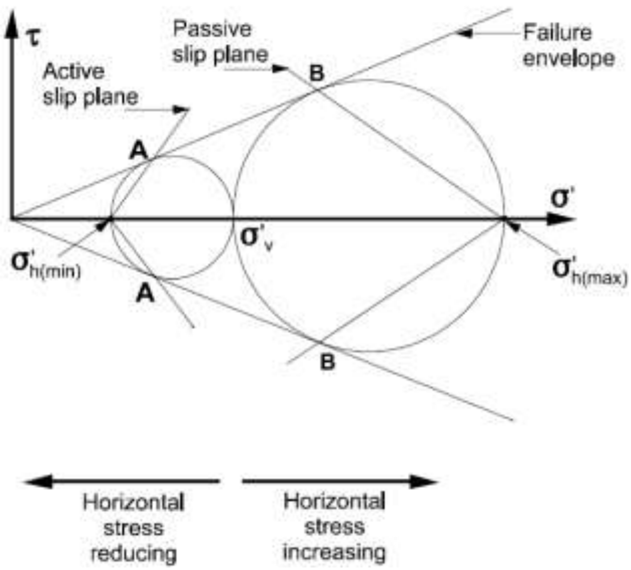


Figure 3.11 *Limiting active and passive horizontal pressures.*

For most anchored wall applications, the effect of wall friction on active earth pressures is relatively small and is often ignored. The active earth pressure coefficient, K_A, may be evaluated using the appropriate equations from above or, for more general cases, from the lower part of figure 3.12 or figure 3.13. The earth pressure coefficients depicted in figure 3.12 and figure 3.13 are based on the assumption of log-spiral shaped failure surfaces for the active and passive sides of the wall.

To evaluate the passive earth pressure coefficient, K_P, the upper part of figure 3.12 or 3.13 should be used.

It is acknowledged that in addition to the Rankine equations and the log-spiral method, a third closed-form technique, herein referred to as the Coulomb method, is often used to calculate lateral earth pressures. For this method, equations are available to calculate K_A and K_P (NAVFAC, 1982). While calculations of K_A are considered to be reasonable, the Coulomb method is unreliable for evaluating passive earth pressures since the planar shape of the assumed Coulomb failure surface is in error compared to the more accurate log-spiral shaped surfaces. Passive pressures calculated using the Coulomb theory are always higher than those based on log-spiral shaped surfaces.

The magnitude of wall friction (δ) typically used in evaluating design passive pressures in front of an excavation ranges from δ=0.5φ’ to 1.0φ’. The value used for design depends on the wall material (e.g. steel or concrete), soil type, method of wall construction, and axial load transfer. For the analysis of continuous sheet-pile walls, a value of δ=0.5φ’ is recommended.

3.9.2 Earth Pressure at Rest

Sand or clay, normally consolidated in the ground under the natural condition of no lateral deformation (i.e., vertical compression only) and under an incremental application of vertical load, experience a condition referenced as the earth pressure at rest. The value of the coefficient of the earth pressure at rest, K_0, is found to be in close agreement with the empirical equation:

$$K_0 = \frac{\sigma'_h}{\sigma'_v} = 1 - sin\varphi' \tag{3.8}$$

For normally consolidated clay, K_0 is typically in the range of 0.55 to 0.65; for sands, the typical range is 0.4 to 0.5. For lightly overconsolidated clays ($OCR \leq 4$), K_0 may reach a value up to 1; for heavily overconsolidated clays ($OCR > 4$), K_0 values may range up to or greater than 2.

In the context of anchored wall design using steel soldier beams or sheet-pile wall elements, design earth pressures based on at-rest conditions are not typically used. Using at-rest earth pressures implicitly assumes that the wall system undergoes no lateral deformation. This condition may be appropriate for use in designing heavily preloaded, stiff wall systems, but designing to this stringent (i.e., zero wall movement) requirement for flexible anchored wall systems for highway applications is not practical (Sabatina et al. 1999).

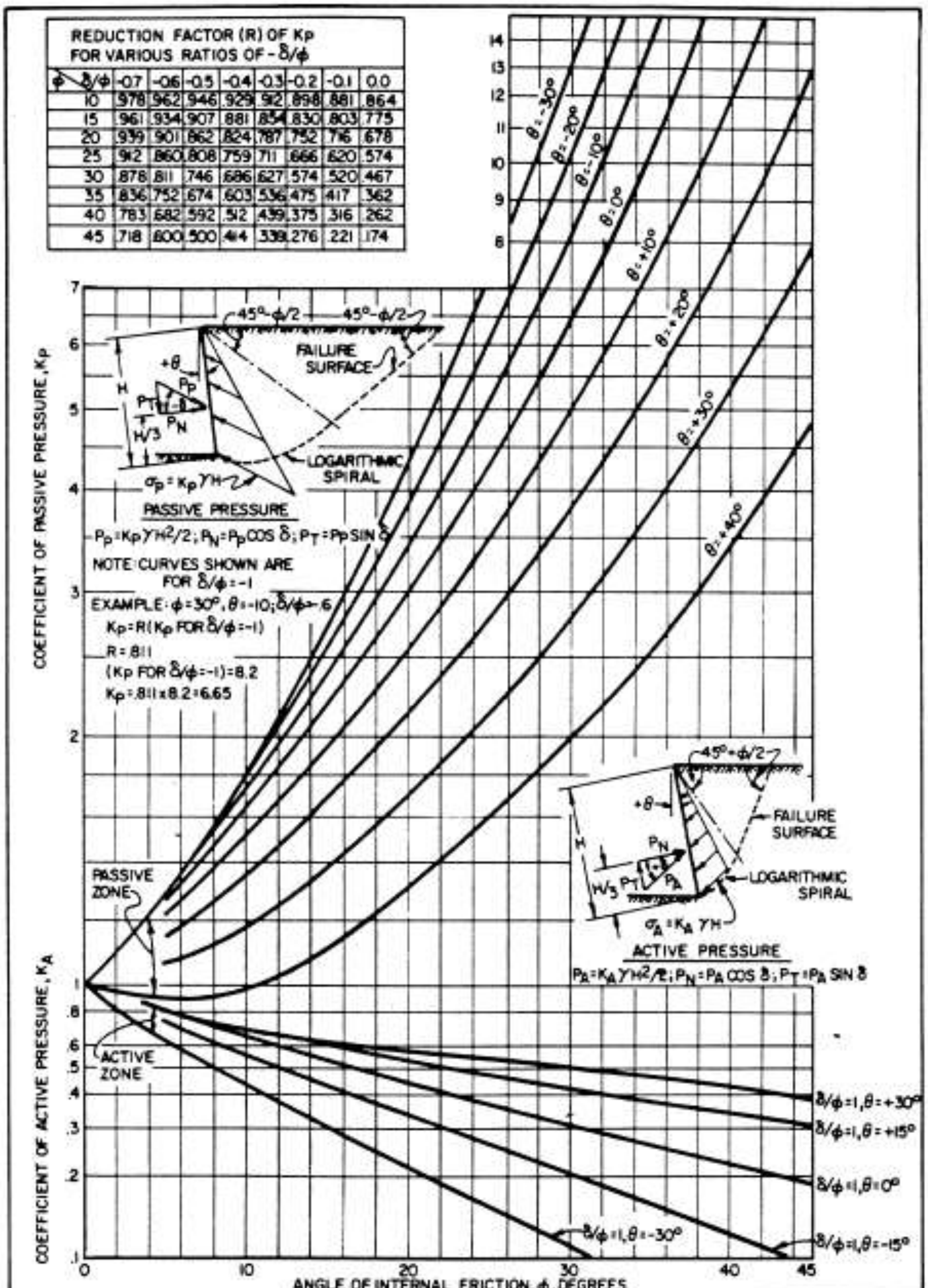


Figure 3.12 *Active and passive earth pressure coefficients (effect of wall inclination).*

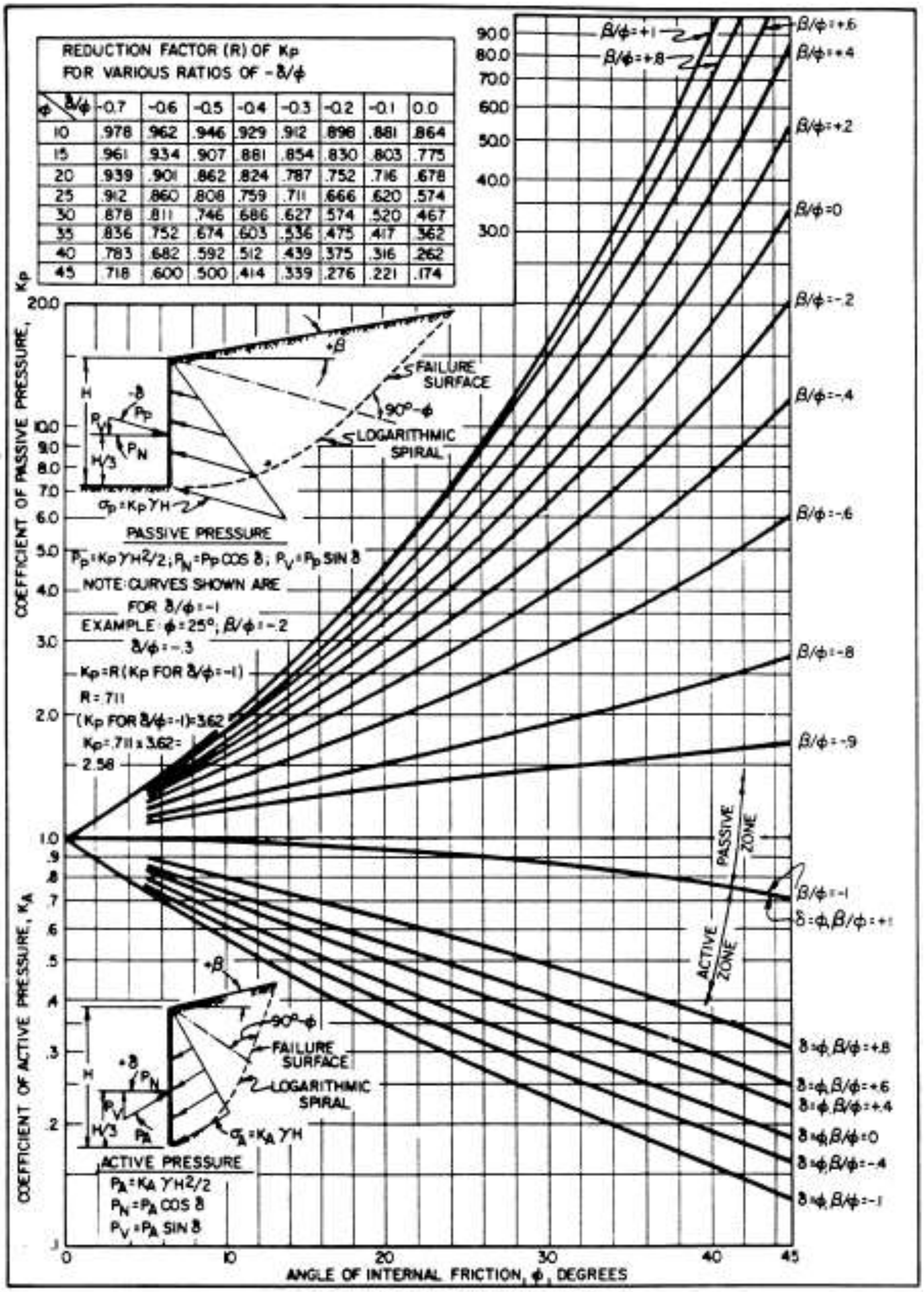


Figure 3.13 *Active and passive earth pressure coefficients (effect of backslope inclination).*

CHAPTER 4
A PARAMETRIC STUDY

4.1 INTRODUCTION

Retaining structures are used in excavations and enables vertical slopes. Sheet pile walls are one of the oldest earth retaining structures constructed to retain earth, water or any other fill material in civil engineering projects. They have been commonly used in the construction industry since early last century. They are used in a wide variety of both temporary and permanent building applications, including retaining walls, slope stabilization, seepage control barriers, river and canal frontages, quays, wharves, sea walls, dock and harbor works, permanent foundations and ground reclamation works. Sheet pile walls are widely used in the construction of container and dry-bulk terminals, as well as for sea walls and reclamation projects where a fill is needed seaward of the existing shore and for marinas and other structures where deep water is needed directly at the shore (El-Naggar, 2010). The commonest types of sheet piles walls are steel (Frodingham or Larssen sections), timber and reinforced concrete (Magbo et al. 2012). Sheet pile walls used to provide lateral earth support can be either cantilever or anchored depending on the wall height. While relatively shorter sheet pile walls can be cantilever, higher walls require anchors (Bilgin, 2010). The selection of wall type, either cantilever or anchored, is based on the function of wall, the characteristics of foundation soils, and the proximity of wall to existing structures (ASCE 1996).

The movement of in-situ walls is an important issue because of the development of new technologies in this field and increase in litigation associated with damages caused by the movements of these walls to adjacent structures (Bilgin & Erten, 2009). Stability of sheet pile walls depend on pressures excreted on its faces. They include the overturning, that results from active earth pressure; unbalanced water, acting upon the inner face of the wall and the passive pressure, acting on the wall's front embedment depth below the dredge line (El-Naggar, 2010). So the stability of anchored sheet pile wall is a function of its depth of embedment and the area of tie rod used as anchor. The wall, as well as its supporting footing must therefore, be suitably designed for stability to be achieved under the effects of lateral earth pressure and also to fulfill the usual requirements of safety and serviceability (Magbo et al. 2012). In order to ensure a successful excavation, the behaviors of the wall and the adjacent ground must be considered during the design phase. However, soil movements due to excavations are not entirely predictable because they are related to a number of factors including soil type, base stability, compression and rebound of the soils, consolidation of soils, wall system stiffness, construction procedures, and workmanship. Any of these factors can contribute to the overall movement of a supported excavation. It is difficult to make a direct and quantitative analysis of the deformation of the ground and support system because the interaction of the factors is complex. So, the estimation of ground movements during excavations is generally a combination of analytical and empirical methods, judgment, and experience (Ma et al., 2010). Many authors and researchers in the literature have attempted to analyze and predict load carrying capacity of these types of walls. Peck (1969); Mana and Clough (1981); Clough and O'Rourke (1990); and Ou et al. (1993) provided well-accepted empirical analysis diagrams on this subject. Some authors used finite element technique or gave examples for numerical analysis to investigate behavior and failure mechanisms of the structure (Magbo et al. (2012); El-Naggar (2010); Bilgin (2010); Ma et al. (2010); Bilgin and Erten (2009); Don and Warrington (2007); Krabbenhoft et al. (2005); Finno and Calvello (2005); Hashash and Whittle (1996, 2002); Briaud and Lim (1999); and Bjerrum et al. (1972)). Full-scaled field tests have been also used to investigate the behavior of these structures (Briaud et al., 2000). More recently, Ghare and saidi (2011) and Shao and Macari (2008) reviewed the application of in situ instrumentation and numerical feedback analysis on deep excavations.

Generally, quay walls play a crucial role in the operational capacity of ports, marinas, shipyards and other waterside facilities. Sheet pile walls are one of the most common types of quay walls used in port construction (El-Naggar, 2010). In a port environment, deepening of the harbour is increasingly becoming a priority requirement, necessary for guaranteeing the ongoing operability of the wharves in the face of a continual increase in the traffic and in the tonnages of merchant and cruise ships (Sciacca et al. 2012). Under these new serviceability conditions, the sheet pile quay walls should be strengthened to ensure an adequate factor of the wall’s safety against soil collapse and/or structural overstressing. Several methods can be used to increase the load-carrying capacity of sheet pile walls. The use of additional anchored tie rods grouting to the backfill soil and arranged along the exposed wall height as an alternative solution to the installation of new sheet piles is one of the most practical and appropriate solutions adopted for rehabilitation and upgrading of the existing quay wall. The Ravenna Port Authority initiated a project to deepen the harbor bottom at selected wharves to a depth of 12.00 m and 14.00 m. Within this context SAPIR Engineering commissioned by the Ravenna Port Authority, has drawn up the project of the wharves of the SAPIR Terminal and of the neighboring docks.

The objective of the present study is to investigate the behavior of anchored sheet pile walls constructed in cohesionless soils. For this purpose, Ravenna Port in Italy, stabilized using sheet pile walls and submerged grouted anchors, was chosen as a case study. The objective was achieved by a series of comprehensive analyses through the finite element program, PLAXIS 2D, version 2012. An extensive parametric study was carried out to investigate the enhancement of using submerged grouted anchors technique on the load response of sheet pile quay wall for the final navigation depth (14.00 m) by varying different parameters such as grout-ties area, length of grouted body, anchor inclination and anchor location. These parameters, which affect the performance of sheet pile quay wall, were considered and evaluated through prediction of horizontal displacements and bending moments induced along the sheet pile wall as well as ground surface settlements and the original anchored force due to the effect of different system parameters.

4.2 NUMERICAL MODELING PROCEDURE

The previous studies showed that the most parameters of sheet pile wall have a significant effect on the behavior of structure. The influence of sheet pile wall geometry, grout-ties area, inclination and location, length of grout, dredging depth and backfill soil angle of repose are the most effective parameters in enhancing this type of walls (El-Naggar, 2010). In the present study, the influence of the anchor such as grout-ties area, length of grouted body, inclination and location are analyzed and the results are presented. A schematic of a typical wall and soil section analyzed is shown in Fig. 4.1.

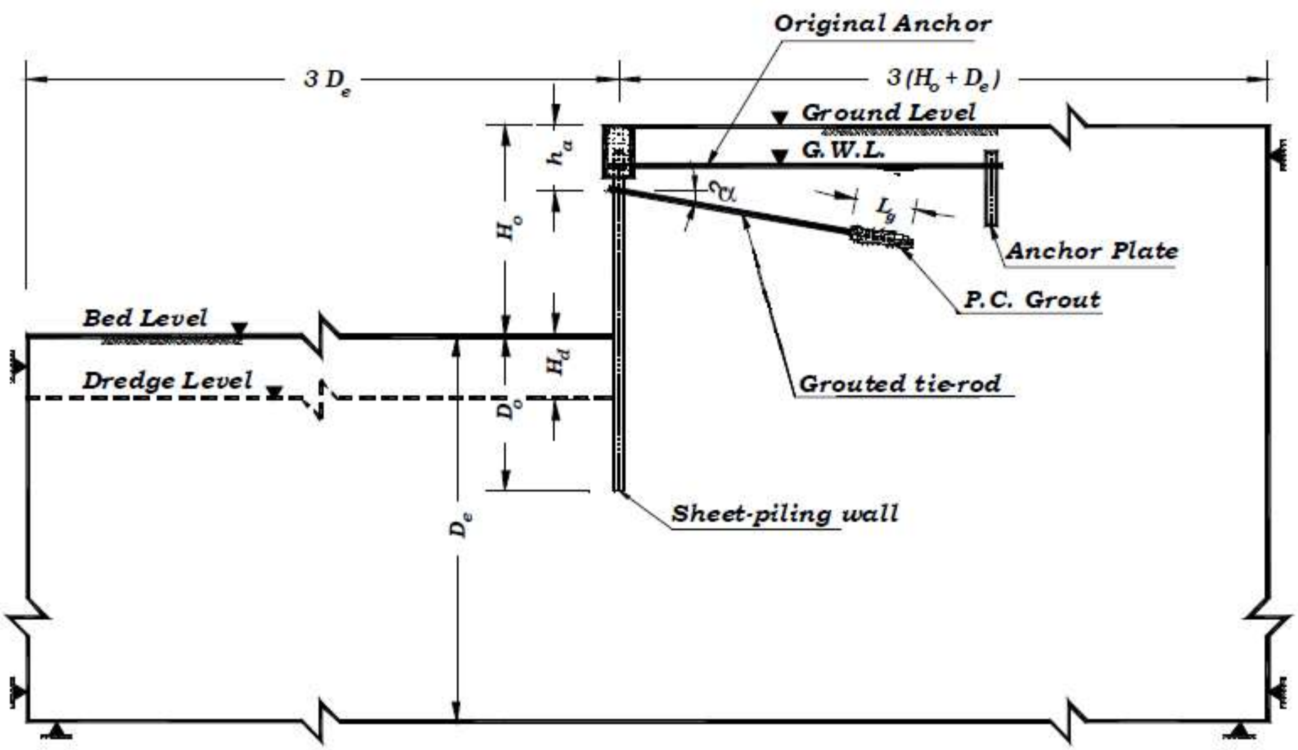


Figure 4.1 *Typical soil and wall profile used in parametric study*

Anchor tie rods were placed at 1 m interval, their diameter is 50 mm. In the finite element model, the additional tie rods are pre-stressed by a force of 100 kN. The average diameter of grouted concrete body varied from 100 mm to 250 mm, whereas its lengths varied from 2 m to 10 m. The choice of the above parameter was based on the practical properties of the material required for marine structures (El-Naggar, 2010).

4.2.1 Finite element analysis

With the advances in computing technology, the use of continuum mechanics numerical methods in the analysis and design of sheet piles has been increasing in recent years. The finite element method has been utilized by researchers to study and understand the behavior of cantilever, braced, and anchored sheet pile walls under static and dynamic loading conditions (Bilgin, 2010). The finite element modeling comprised two-dimensional plane strain analysis and analyses were carried out using PLAXIS.

4.2.2 Boundaries and fixity conditions

One of the first steps in any numerical simulation is to determine where to place the boundaries so that their influence on the results will be minimized. Briaud & Lim (1997) suggested that bottom of the mesh is best placed at a depth where soil becomes notably harder (say at a depth D below the bottom of the excavation). Based on their studies, if D is not exactly known, D can be taken as two to three times the vertical depth of excavation He. Further, for known values of D and He, width of excavation We can be taken equal to three to four times D and the horizontal distance from wall face to the end of mesh boundary Be can be chosen equal to three to four times (He + D).

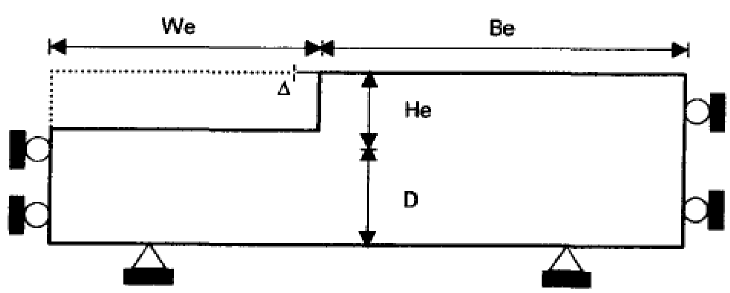


Figure 4.2 *Mesh boundaries and fixity conditions (Briaud and Lim 1997)*

All nodes on the vertical sides of the model are restrained from moving in the horizontal direction to represent the rigid-smooth lateral boundaries and to represent the rough-rigid surface boundaries. All nodes on the bottom surface are restrained in both horizontal and vertical directions. Figure 4.2 shows the mesh boundaries and fixity conditions.

Soil layers were modeled using 15-node triangular elements. The 15-node elements provide a fourth order interpolation for displacements and the numerical integration involves 12 stress points. The sheet pile wall was modeled by using five-node elastic plate elements. Interface elements had 10 nodes, five on the soil elements and five on the wall elements. The original and grouted anchors are modeled by the node-to-node anchor element, while the grout body is simulated by the geo-grid element. Due to a stress concentration in and around the wall, a finer finite element mesh was used in these areas and mesh became coarser in the zones away from the wall. The finite element model used for the present work is shown in Figure 4.3.

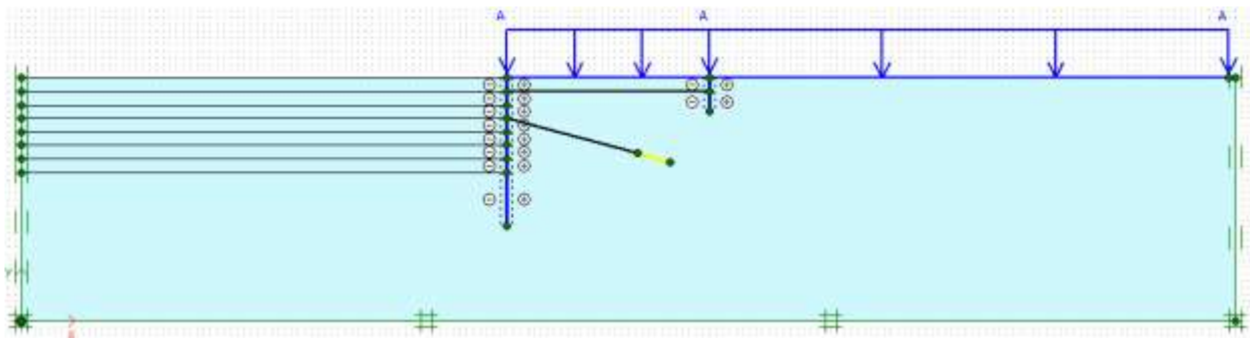


Figure 4.3 *Finite Element Model as Generated by PLAXIS.*

The Mohr–Coulomb constitutive model for soils has been commonly used in finite element modeling of retaining wall behavior (Potts & Fourie, 1985; Day & Potts, 1993; Grande et al., 2002; Krabbenhoft et al., 2005; Bilgin & Erten, 2009; Tan & Lu, 2009; Fan & Luo, 2008; Neher & Lachler, 2006). There are more sophisticated constitutive models available in PLAXIS and used by researchers to model complex soil behavior, i.e. soft-soil-creep model for peat (Tan & Paikowsky, 2009) and hardening-soil model for clay (Rechea et al., 2008; Bilgin, 2006). The Mohr-Coulomb failure criterion is currently the most widely used method for soil in practical applications (El-Naggar, 2010) and has been successfully used for granular soils and therefore was also employed in this study to model the stress–strain behavior of sands. The Mohr–Coulomb model is a linearly elastic and perfectly plastic constitutive model. The parameters needed for the Mohr–Coulomb model are the Young's modulus, E, and Poisson's ratio, ν, for the elastic strain component of the soil behavior. The effective strength parameters cohesion, c', and friction angle, φ', as well as the dilatancy angle, ψ, are needed for the plastic strain component of the soil behavior. Based on geotechnical reports of Ravenna Port the backfill soil is homogenous sand with the average properties given in Table 4.1. The position of the original anchor is considered to be coincident with the ground water level. According to Ravenna port quay walls, a constant distance of 2.0 m is taken into account between the ground surface and horizontal levels of the original anchor.

Table 4.1 *Soil properties*

Property	Sand
Saturated unit weight	21 (kN/m^3)
Unsaturated unit weight	19 (kN/m^3)
Friction angle	30 (°)
Dilatancy angle	2 (°)
Modulus of elasticity	30 (MPa)
Poisson's ratio	0.3
Interface strength, R_{int}	0.67

4.3 RESULTS AND ANALYSIS

It is concluded from parametric studies that all parameters pertaining to sheet pile quay walls system in this study have a significant effect on the behavior of the structure. In the following subsections the results of these analyses in terms of wall deformations, surface settlements, wall bending moments, and anchor forces obtained from different parameters is presented.

4.3.1 Effect of grouted anchor inclination (α)

To represent the effect of the grouted anchor inclination on the internal forces of this type of marine structure, five values of α were studied, (α = 0°, 10°, 20°, 30° and 40°). The other parameters of the model were kept constant as, H_0 = 10 m, H_d = 2 m, h_a = 6 m, φ = 30°, D_{DH} = 150 mm, L_g = 6 m and P_v = 40 kN/m^2.

Figures 4.4, 4.5, 4.6 and 4.7 present the effect of grouted anchor inclination on the internal forces of the structure. The results show that an increase of α up to 20° leads to a decrease in the maximum bending moment, horizontal displacement of wall face and ground surface settlement, but an increase in the original anchor force. After that by increasing the anchor inclination α, the maximum bending moment, horizontal displacement and ground surface settlement increase and original anchor force decreases. These observations indicate that, for this case study, increasing the angle of grouted anchor inclination α up to 20° enhance the performance of the structure and generally improve the efficiency of the wall.

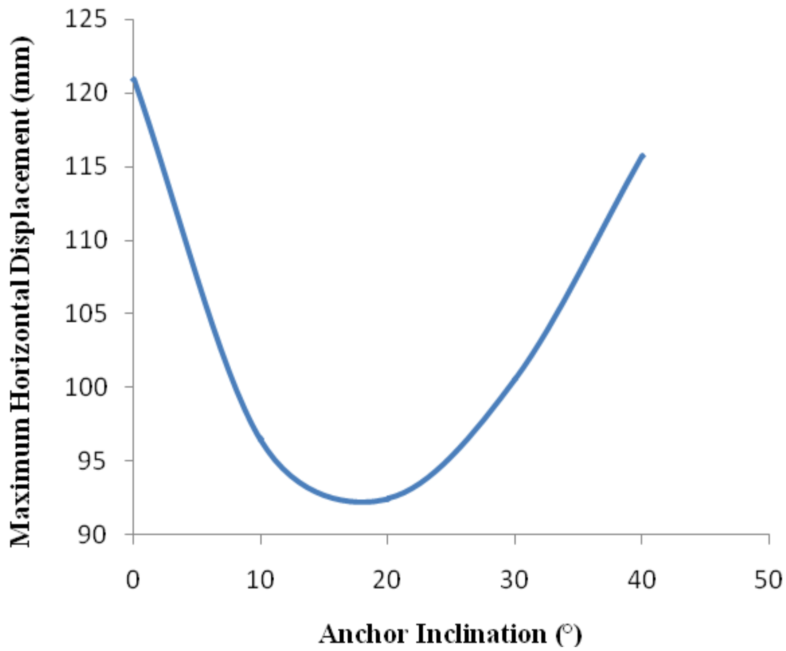


Figure 4.4 *Maximum horizontal displacements versus anchor inclination degree*

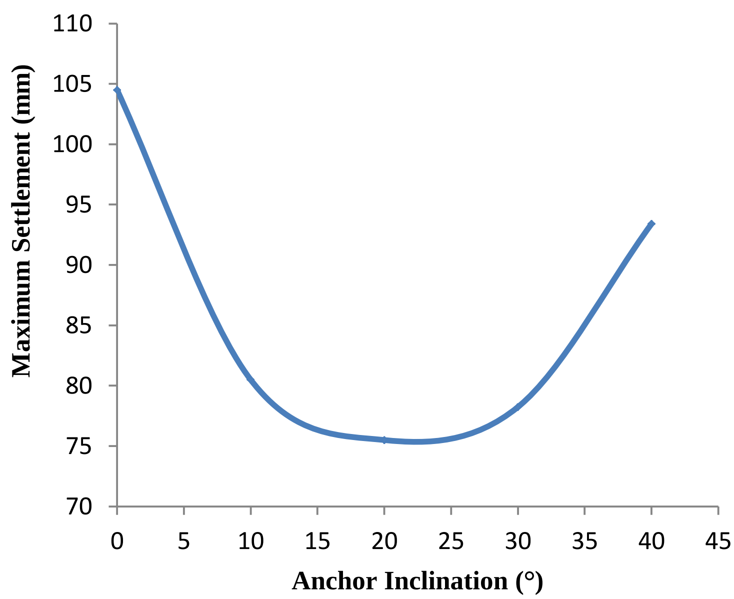


Figure 4.5 *Maximum ground surface settlement versus anchor inclination degree*

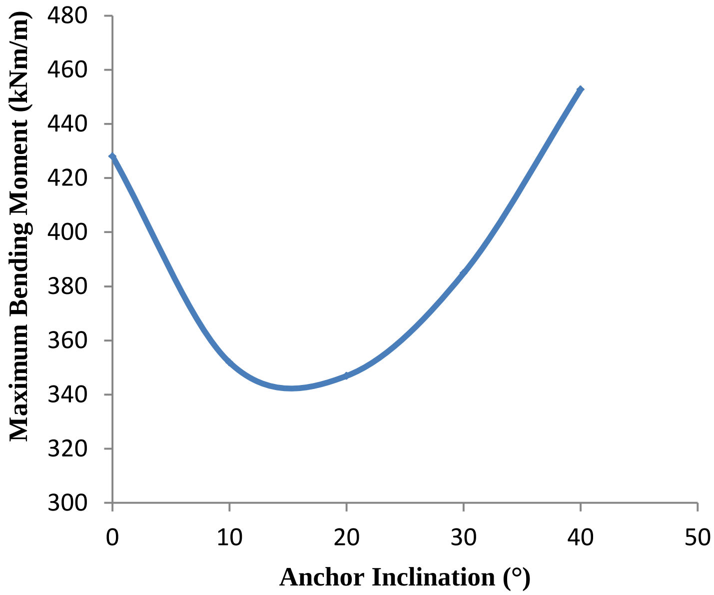


Figure 4.6 *Maximum bending moment in sheet pile versus anchor inclination degree*

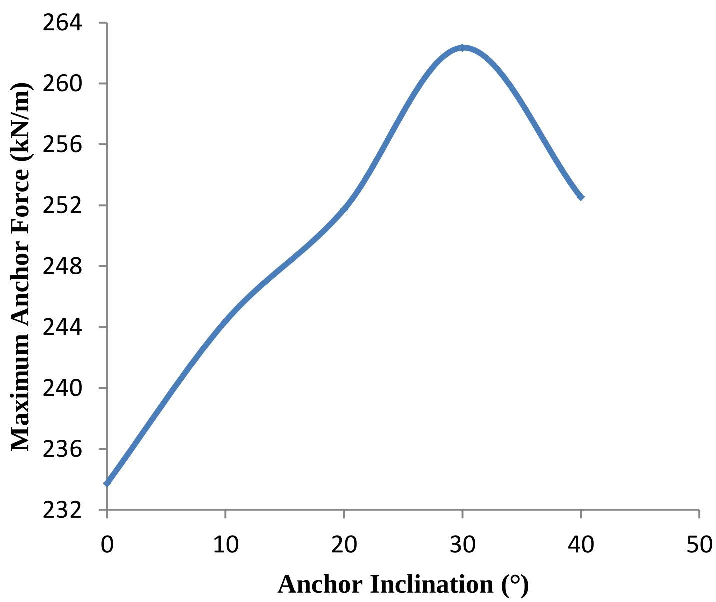


Figure 4.7 *Maximum anchor forces versus anchor inclination degree*

4.3.2 Effect of grouted body length (Lg)

The location of the critical potential failure surface must be evaluated since the anchor bond zone must be located sufficiently behind the critical potential failure surface so that load is not transferred from the anchor bond zone into the "no-load" zone. For walls constructed in cohesionless soils, the critical potential failure surface can be assumed to extend up from the corner of the excavation at an angle of 45° + φ/2 from the horizontal (i.e., the active wedge) (Sabatini et al., 1999). To study the effect of grouted body length on the behavior of the structure, five different values of L_g were considered (L_g = 2, 4, 6, 8, and 10 m). The other parameters of the model were kept constant as, H_0 = 10 m, H_d = 2 m, h_a =6 m, α = 20° m, φ = 30°, D_{DH} = 150 mm and P_v = 40 kN/m^2.

Table 4.2 presents the results of different values of grout length Lg. From these results, it can be noticed that the increase of L_g enhances the performance of this type of marine walls. Also, it reduces the maximum bending moment and horizontal displacement of the sheet pile wall as well as it reduces the maximum ground surface settlement. These observations indicate that increasing the grout length, L_g leads to increase in fixity of the structure and then enhances the performance of the wall. The results also show that the reductions in the internal forces are not significant when the grout length, L_g is greater than 6 m. In this case study, it is recommended that the value of L_g should fall between 4 m and 6 m.

Table 4.2 *Summary of the effect of grouted body length on structure behavior*

L_g (m)	Max. U_x (mm)	Max. U_y (mm)	Max. M (kNm/m)	Max. F (kN/m)
2	109.30	88.31	398	234.6
4	97.62	79.91	363	249.0
6	92.44	75.50	347	251.7
8	90.93	74.56	343	260.1
10	89.48	73.14	343	264.1

4.3.3 Effect of grouted anchor depth (ha)

Location of grouted anchor from ground surface is one of the most dominant factors affecting the behavior of the anchored sheet pile quay walls.

To investigate the role of grouted anchor depth on the enhancement of sheet pile quay walls, four values of ha have been adopted, (h_a = 4, 6, 8 and 10 m). The other variables of the system were considered constant as, H_0 = 10 m, H_d = 2 m, α = 20° m, φ = 30°, D_{DH} = 150 mm, L_g = 6 m and P_v = 40 kN/m^2.

The results showed a considerable reduction in the internal forces of the structure due to an increase in grouted anchor depth; see (Figures 4.8 – 4.11). The results present that an increase in grouted anchor depth reduces significantly the maximum bending moment, horizontal displacement of wall face, and ground surface settlement. Unfortunately, this leads to an increase of the maximum original anchor force as shown in figure 4.11.

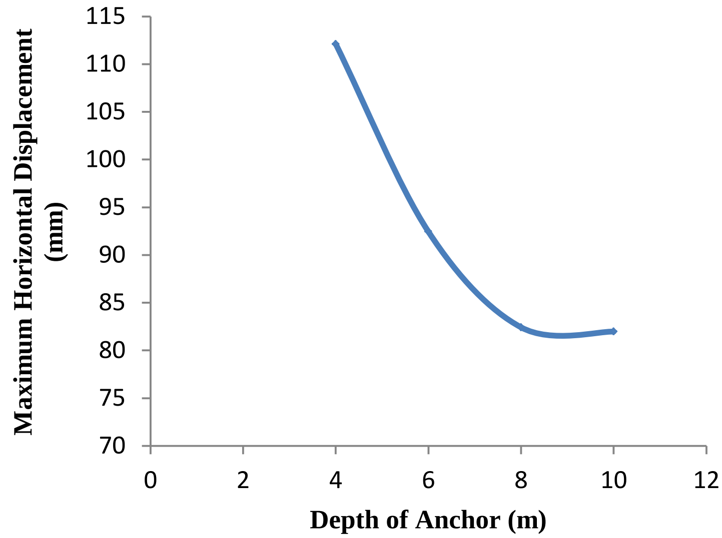


Figure 4.8 *Maximum horizontal displacements versus anchor inclination degree*

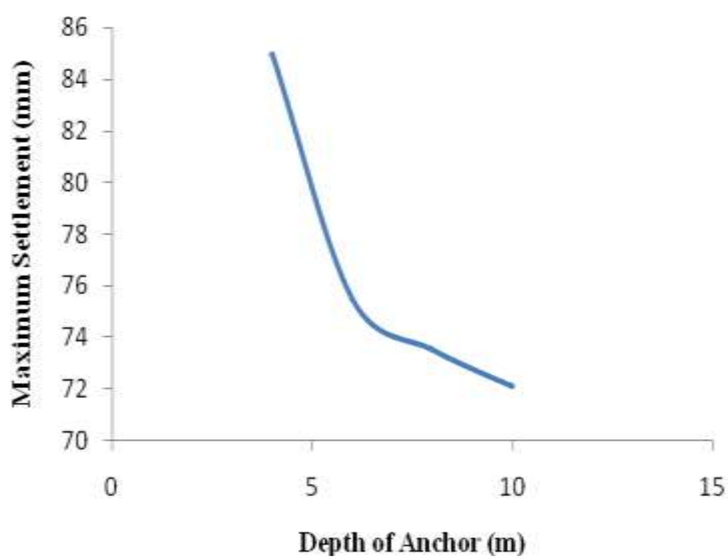


Figure 4.9 *Maximum ground surface settlement versus anchor inclination degree*

The former observations indicate that, increasing the grouted anchor depth generally enhances the performance of the structure, but a trade-off is required to balance the increase of original anchor force.

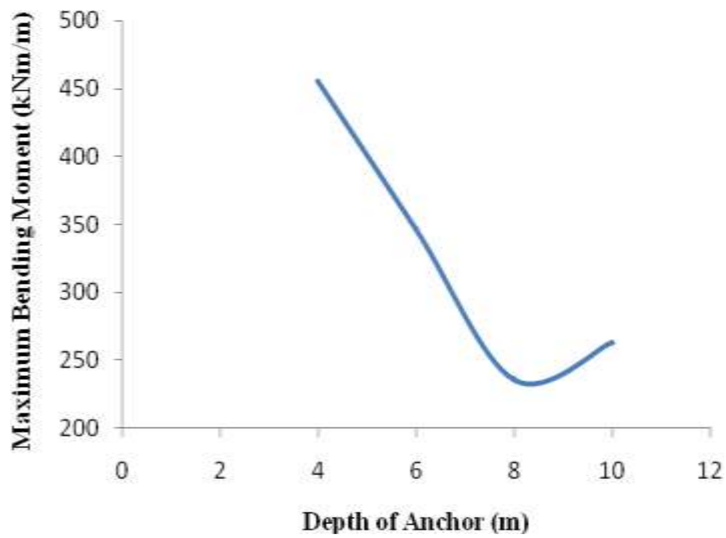


Figure 4.10 *Maximum bending moment in sheet pile versus anchor inclination degree*

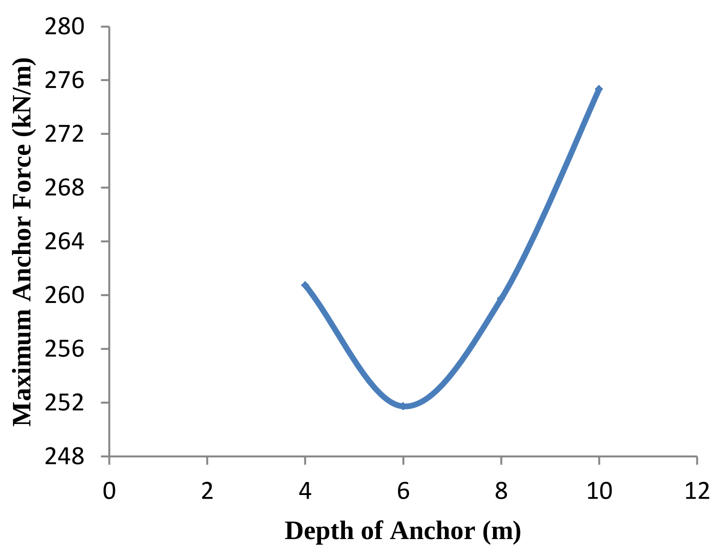


Figure 4.11 *Maximum anchor forces versus anchor inclination degree*

4.3.4 Effect of grouted body area (Ag)

Using grouted anchors in the enhancement of sheet pile quay walls leads to a considerable reduction in the maximum bending moment exerted in the quay wall as well as the corresponding and maximum horizontal displacement of the sheet pile wall, ground surface settlement and existing anchor force. To study the effect of grouted body area on the enhancement of the structure, four different values of A_g were examined (D_{DH} = 100, 150, 200 and 250 mm). The other parameters of the model were kept constant as, H_0 = 10 m, H_d = 2 m, h_a = 6 m, α = 20°, φ = 30°, L_g = 6 m and P_v = 40 kN/m^2.

Table 4.3 summarizes the results of different values of grouted body area, A_g. The results show that, as the grouting area, A_g increased, the maximum bending moment decreased. Similarly, the maximum horizontal displacement and ground surface settlement will decrease with the increase of A_g. This enhancement could be attributed to the contribution of grouting ties area, which helps to increase the overall stiffness of the structure and thus decrease the internal forces induced in the sheet pile quay wall system.

Table 4.3 *Summary of the effect of grouted body area on structure behavior*

D_{DH} *(mm)*	Max. U_x *(mm)*	Max. U_y *(mm)*	Max. M *(kNm/m)*	Max. F *(kN/m)*
100	94.98	77.24	357	249.7
150	92.44	75.50	347	251.7
200	92.06	75.33	345	254.0
250	91.74	75.13	344	254.9

4.4 CONCLUSION

A numerical study using finite element analysis was conducted to investigate the effect of grouted body area, length of grouted body, inclination and location of the anchor on the behavior of anchored sheet pile walls. Based on the results of presented parametric study, several conclusions can be drawn, which are to be included in guideline recommendations for enhancement of the sheet pile quay walls in cohesionless soils. The rehabilitation of sheet pile quay walls using additional grouted tie-rods has a significant role on the performance of deepened quay wall systems. The anchored wall system and surrounding soil show more stabilized behavior when the grouted anchors are used. The maximum bending moment and horizontal displacement occurring along the sheet pile wall and also the maximum ground surface settlement have been considerably reduced by increasing the pertaining parameters of the system but obviously, this leads to an increase of the maximum original anchor force. Therefore, a trade-off is required to balance the increase of original anchor force and the reduction of maximum bending moment and displacements. Results also show that the grouted anchor inclination has a great effect on the system's performance and an inclination up to 20° increases the performance enhancement of the system. Furthermore, the optimal length of the grouted anchor, in this case study, is in the range between 4 m and 6 m.

CHAPTER 5
COMPARISON OF A 2D AND 3D MODEL

5.1 INTRODUCTION

The major challenge involved for the geotechnical engineers is the task of accurately predicting the performance of geotechnical project such as excavations. Numerical analysis such as finite element method is almost always used to make predictions of ground behavior (Ehsan, 2013). The development of two-dimensional (2D) and three-dimensional (3D) finite element analysis has greatly enhanced our ability to model complex geotechnical problems. These advanced techniques, however, offer a number of challenges including: evaluating the effects of numerical error and instability; the time for setup, verification, and execution; and, computer memory requirements. Traditional models, such as 1D consolidation theory are severely limited in their ability to model complex situations, although they offer the advantage of simplicity (Klettke & Edgers, 2011).

This paper describes a comparison of results obtained from a 2D and a 3D model, focusing on the differences between the two methods. The San Vitale quay wall in Ravenna as shown in figure 5.1 was selected as the case study for this comparison.
The purpose of this paper is to compare the behavior of sheet pile quay wall predicted by Plaxis 2D and 3D programmes in terms of horizontal displacements induced along the sheet pile wall as well as ground surface settlements and the original anchored force and calculated factor of safety.

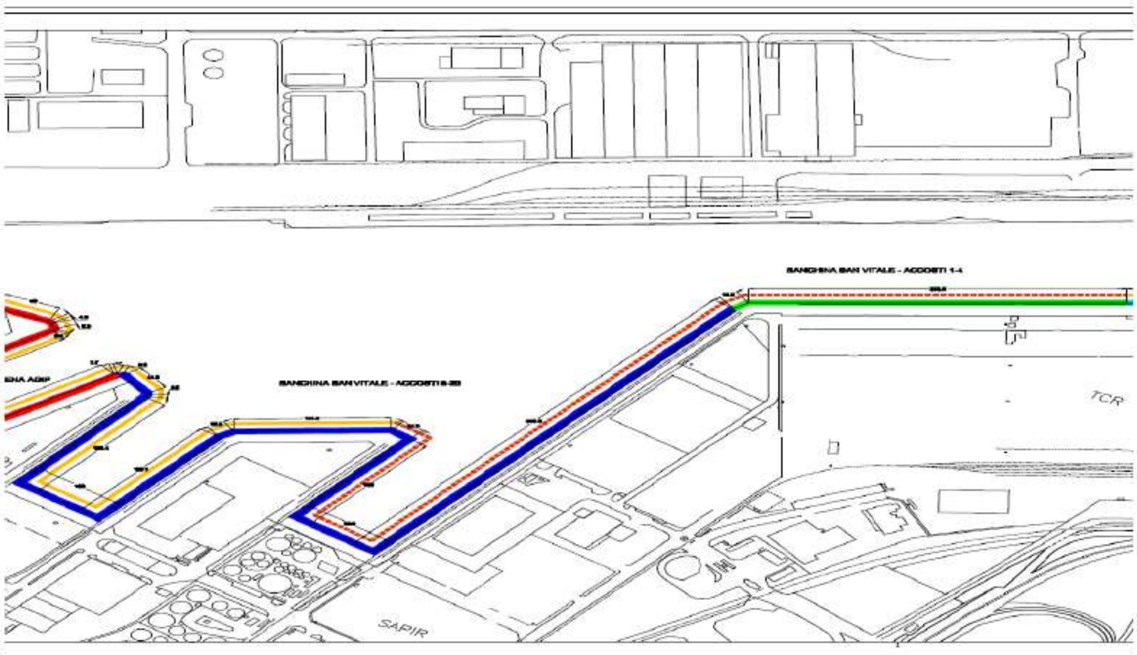


Figure 5.1 *San Vitale quay wall plan*

5.2 NUMERICAL MODELING

This project consists of 2 main phases. In the first phase after installing the sheet pile, the upper horizontal and vertical plate and inclined and vertical piles the harbor is going to be dredged until -10.60 m under the mean sea level. In the second phase after installing the anchor the bottom of the sea is going be deepened until -12.0 m below the mean sea level. A schematic of a typical wall, piles, anchor and soil section analyzed is shown in Fig. 5.2.

5.2.1 Soil properties

One of the most important parameters in the numerical modeling is properties of soil profile. To determine the parameters of soil profile, it is required to conduct geotechnical studies. Therefore, in the current research, the results of both field and

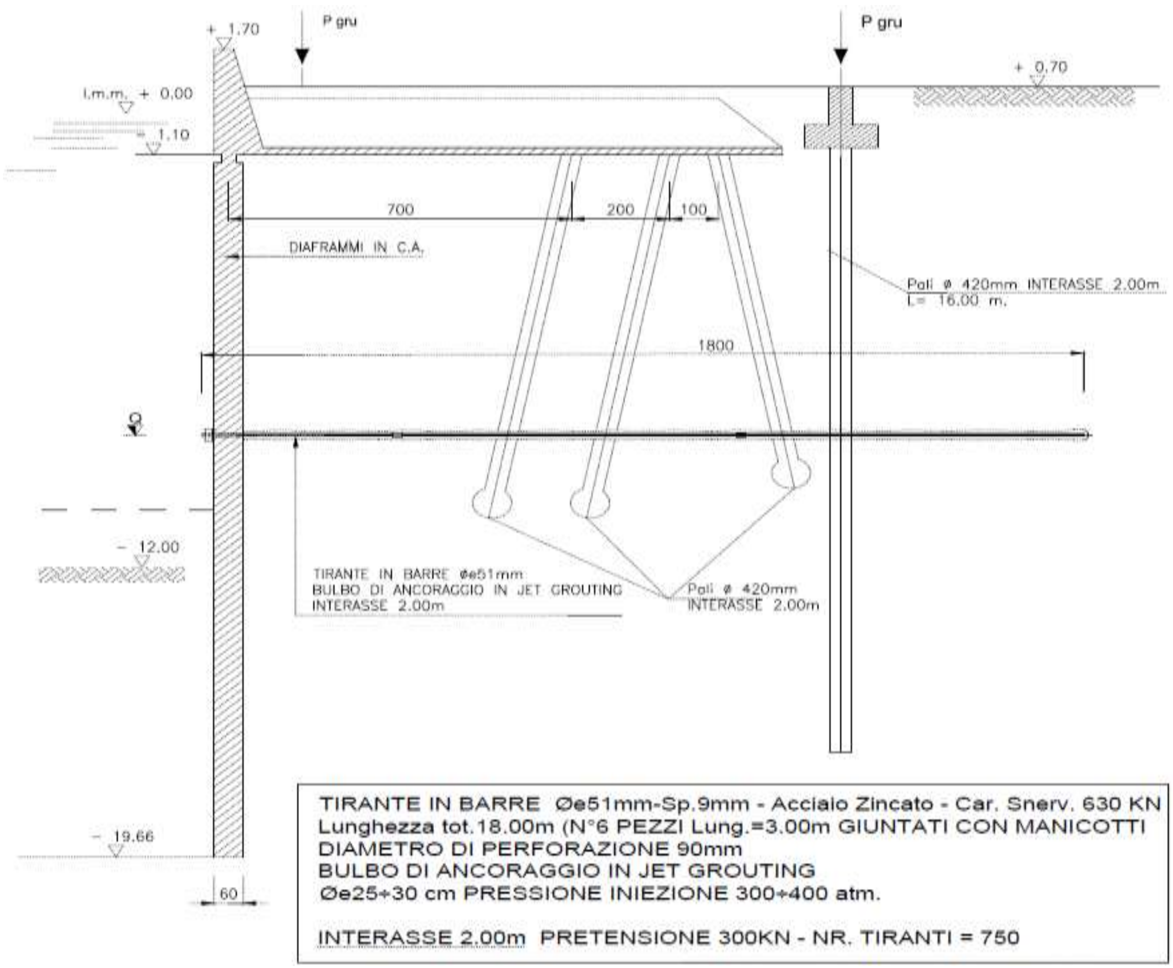


Figure 5.2 *A schematic view of the project.*

laboratory studies have been employed. After examining the field and laboratory studies, soil profile was classified into two layers according to Table 5.1.

Table 5.1 *Soil properties*

Property	Sand	Silty Clay
Saturated unit weight	20 (kN/m^3)	20 (kN/m^3)
Unsaturated unit weight	19 (kN/m^3)	18 (kN/m^3)
Friction angle	33 (°)	26 (°)
Dilatancy angle	2 (°)	-
Modulus of elasticity	20 (MPa)	8 (MPa)
Poisson’s ratio	0.3	0.35
Interface strength, R_{int}	0.67	0.5

The main important design parameters and characteristics of the bulkhead are summarized in the table 5.2.

Table 5.2 *Main Project Parameters*

Design elevation of the plaza	+0.7 m
Current position of the bottom	-10.50 m
Future position of the bottom	-12.00 m
Position of the toe of the bulkhead	-19.70 m
Position of existing anchor	-1.10 m
Length of the existing anchor	11.40 m
Distance between existing bolts	2.00 m
Position of the new anchor	-8.50 m
Distance between new anchors	2.00 m
Inclination of the new anchors	0°
Prestressing of the new anchors	300 KN
Overload operating	40 KPa

5.2.2 Finite element analysis

PLAXIS 2D and PLAXIS 3D software which are based on the finite element method, have wide applications in solving numerical geotechnical problems such as determining the excavation wall behavior (Niroumand and Kassim, 2010). In the finite element method, differential equations are solved by interpolation functions and the governing equation of each element is derived. Through the integration of equations of each element, the governing equations on the numerical model can be obtained, and finally, these equations are substituted by a system of linear or non-linear equations (Ghareh and Saidi, 2011).

2D Analysis

PLAXIS 2D is a robust simulator of geotechnical problems which works on the basis of finite element method. This software is capable of solving a wide range of problems, from simple linear analysis to highly complex nonlinear simulation, particularly, through considering the effect of soil-structure interaction (Babu and Singh, 2009). The software enables modeling different types of soil model in a simple graphical environment with the possibility of stage construction so as to reflect more realistic conditions of the problem. Finite element method is a powerful tool for the analysis of interaction of anchorage, soil, and structure. The advantage of this method lies in its capability to analyze elements of anchored wall, soil, and their interaction considering the effects of parameters. Moreover, using finite element method, it is possible to study any two-dimensional soil profile under plane strain theory. It should be mentioned that one of the reasons for the success of finite element method in the analysis of problems is the use of different behavioral models of soil considering a wide range of strains, effects of loading speed, effects of stiffness decrease, etc. (PLAXIS Ver. 8).

Finite element analyses were performed with PLAXIS 2D and calculations were carried out by plane-strain analysis. Mohr-Coulomb failure criterion is currently the most widely used method for soil in practical applications (El-naggar, 2010). As a result, the soil was presented as an elastic-perfectly plastic material based on Mohr- Coulomb failure criterion. 15-noded triangle elements were used for modeling the soils. The sheet-pile wall and horizontal and vertical slabs have been modeled by 5-noded beam elements. Between the structure and soil elements, 5-noded interface elements of zero thickness were used. The original and grouted anchors are modeled by the node-to-node anchor element, while the grout body is simulated by the geo-grid element. As stated earlier, that anchored sheet pile walls are modeled as plane strain problem in PLAXIS 2D, plate (or geogrid) structural elements can used to simulate as grouted body of anchors. The most important input material parameters for plate elements are the flexural rigidity (bending stiffness) EI and the axial stiffness EA (for geogrid structural element only the axial stiffness EA is required) (Babu and Singh, 2009). Both plate and geogrid structural elements are rectangular in shape with width equal to 1 m in out-of-plane direction.

Since, the anchors and the grouted body parts are circular in cross-section and placed at designed horizontal spacing, it is necessary to determine equivalent axial and bending stiffnesses for the correct simulation of circular anchors and the grouted body parts as rectangular anchor, plate or geogrid elements.

For the grouted body, equivalent modulus of elasticity E_{eq} shall be determined accounting for the contribution of elastic stiffnesses of both grout cover as well as original anchor-tie rods. From the fundamentals of strength of materials, E_{eq} can be determines as:

$$E_{eq} = E_a\left(\frac{A_a}{A}\right) + E_g\left(\frac{A_g}{A}\right) \tag{5-1}$$

Where: E_g is the modulus of elasticity of grout material; E_a is the modulus of elasticity of original anchor-tie rods; E_{eq} is the equivalent modulus of elasticity of grouted anchor; $A = 0.25\pi D_{DH}{}^2$ is the total cross-sectional area of grouted Anchor; $A_g = A - A_a$ is the cross-sectional area of grout cover; $A_a = 0.25\pi d^2$ is the cross-sectional area of original anchor-tie rods and D_{DH} is the diameter of drill hole. If, S_h is horizontal spacing of anchors, knowing the equivalent modulus of elasticity for the grouted anchor, the axial and bending stiffnesses can be determined using the following equations respectively.

$$\text{Axial stiffness EA[kN/m]} = \frac{E_{eq}}{S_h}\left(\frac{\pi D_{DH}{}^2}{4}\right) \tag{5-2}$$

$$\text{Bending stiffness EI[kNm}^2\text{/m]} = \frac{E_{eq}}{S_h}\left(\frac{\pi D_{DH}{}^4}{64}\right) \tag{5-3}$$

Substituting, EA and EI values in the material properties menu for Plate elements, PLAXIS automatically determines the equivalent plate thickness in meter d_{eq} using the following equation.

$$d_{eq} = \sqrt{12\left(\frac{EI}{EA}\right)} \tag{5-4}$$

All nodes on the vertical sides of the model are restrained from moving in the horizontal direction to represent the rigid-smooth lateral boundaries and to represent the rough-rigid surface boundaries. All nodes on the bottom surface are restrained in both horizontal and normal directions. The finite element mesh used for the present work is shown in Figure 5.3.

5.3 RESULTS AND ANALYSIS

Horizontal displacements induced along the sheet pile wall, ground surface settlements and the original anchored force and calculated factor of safety were investigated for the comparison of the 2D and 3D model.

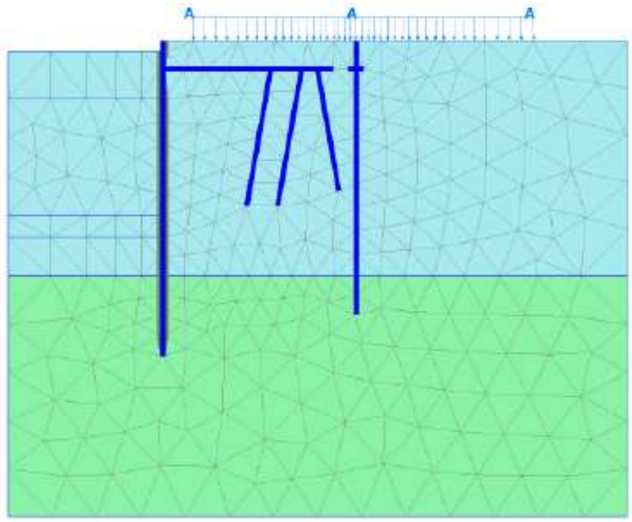


Figure 5.3 *Finite Element Model generated by PLAXIS 2D.*

For the first part, the model has been developed with the full length of the piles to investigate the results. The following calculation steps have been performed, but only results for the final stages are presented in the following (Figures 5.4 and 5.5).

- Step 0: Initial stress state ($\sigma'_v = \gamma.h$, $\sigma'_h = K_0\ \sigma'_v$, $K_0 = 1 - \sin\phi'$)
- Step 1: Activate sheet pile wall
- Step 2: Excavation to level -3.0 m
- Step 3: Activate horizontal slab and piles at level -1.1 m
- Step 4: Excavation to level -10.50 m
- Step 5: Apply surcharge load (permanent load of 40 kPa)

The summary of the results for this part is shown in Table 5.3.

Table 5.3 *Summary of the results for phase 1*

Phase	Max. Horizontal Displacement (cm)	Max. Vertical Displacement (cm)	F.S
Phase 1 (Full Length Piles)	9.93	13.47	1.33

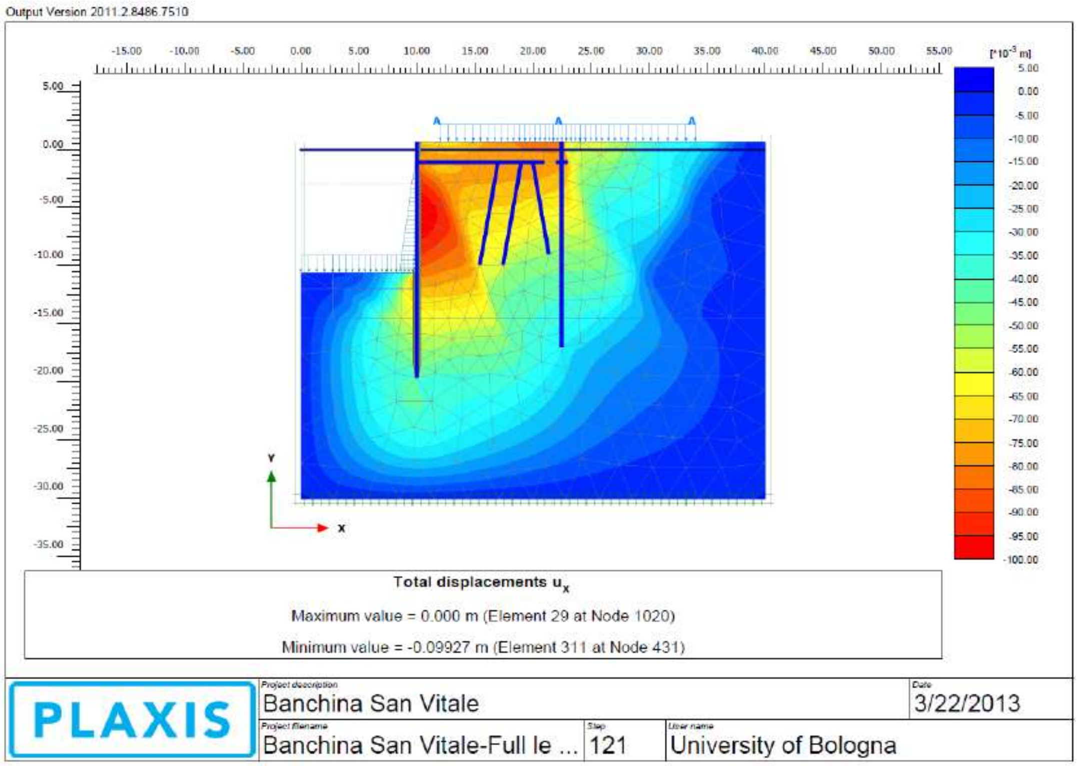


Figure 5.4 *Horizontal Displacements for the full length piles.*

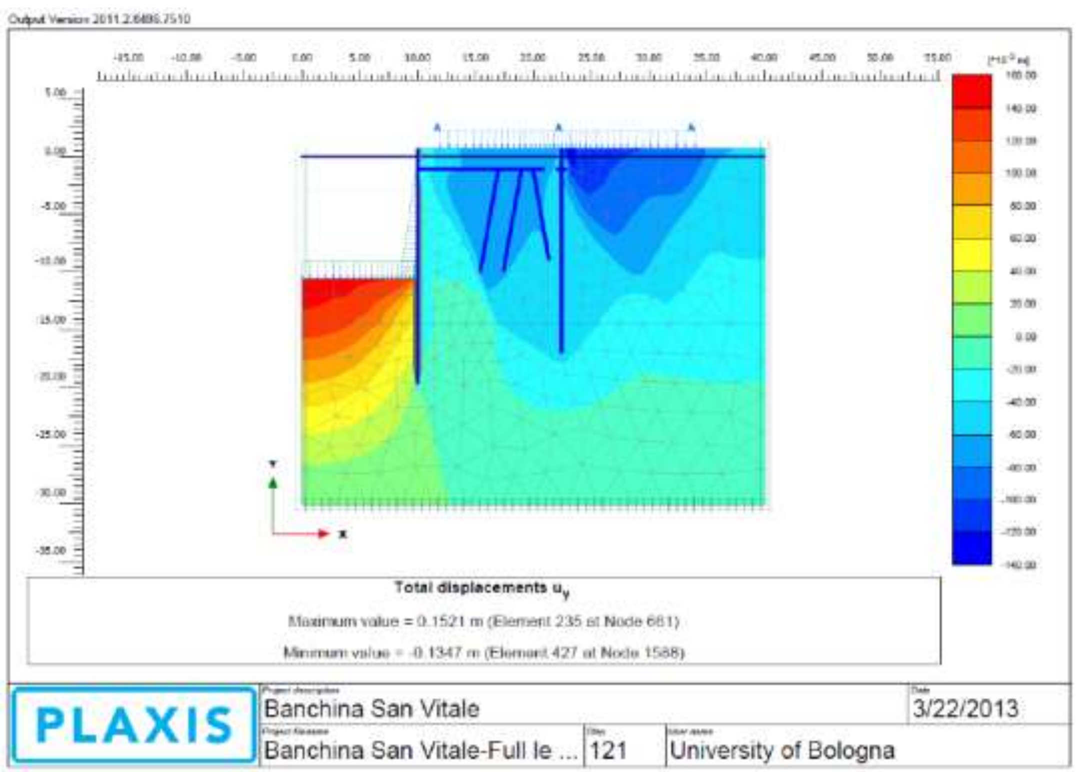


Figure 5.5 *Vertical Displacements for the full length piles.*

The next step in this phase is installation the anchor at the depth of -8.50 m. As we are working with the 2D version, by installing the anchor we have an intersection between piles and the anchor. To avoid this intersection, the lengths of the piles have been decreased but the capacities of the piles were remained constant and did not change (Figure 5.6). To control this procedure the results at the end of this phase were compared to the previous case (Figures 5.7 and 5.8). Table 5.4. presents the results of this case and the previous one for the comparison.

Table 5.4 *Summary of the results for phase 1 in 2 conditions*

Phase	Max. Horizontal Displacement (cm)	Max. Vertical Displacement (cm)	F.S
Phase 1 (Full Length Piles)	9.93	13.47	1.33
Phase 1 (Shortened Piles)	10.57	13.04	1.32

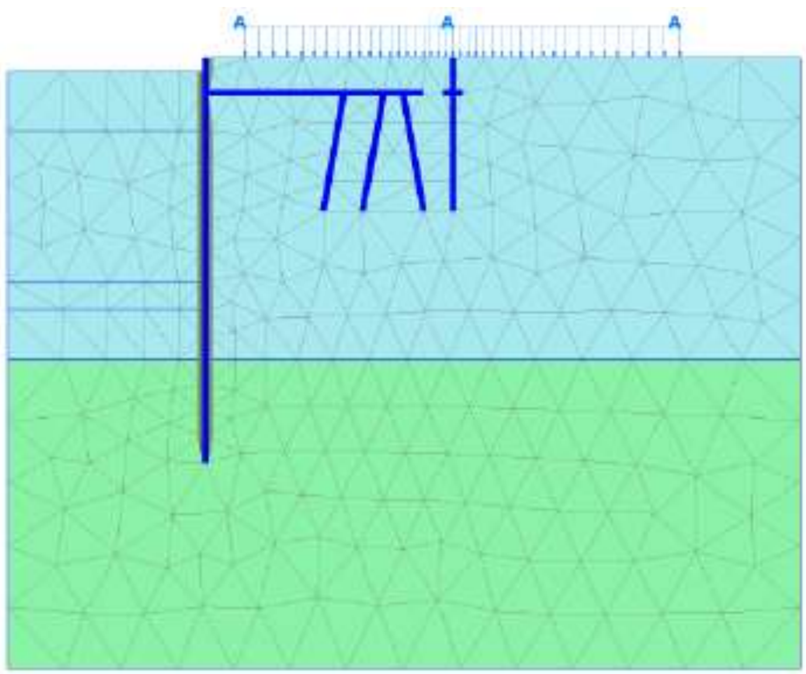


Figure 5.6 *Finite Element Model generated by PLAXIS 2D (Shortened Piles).*

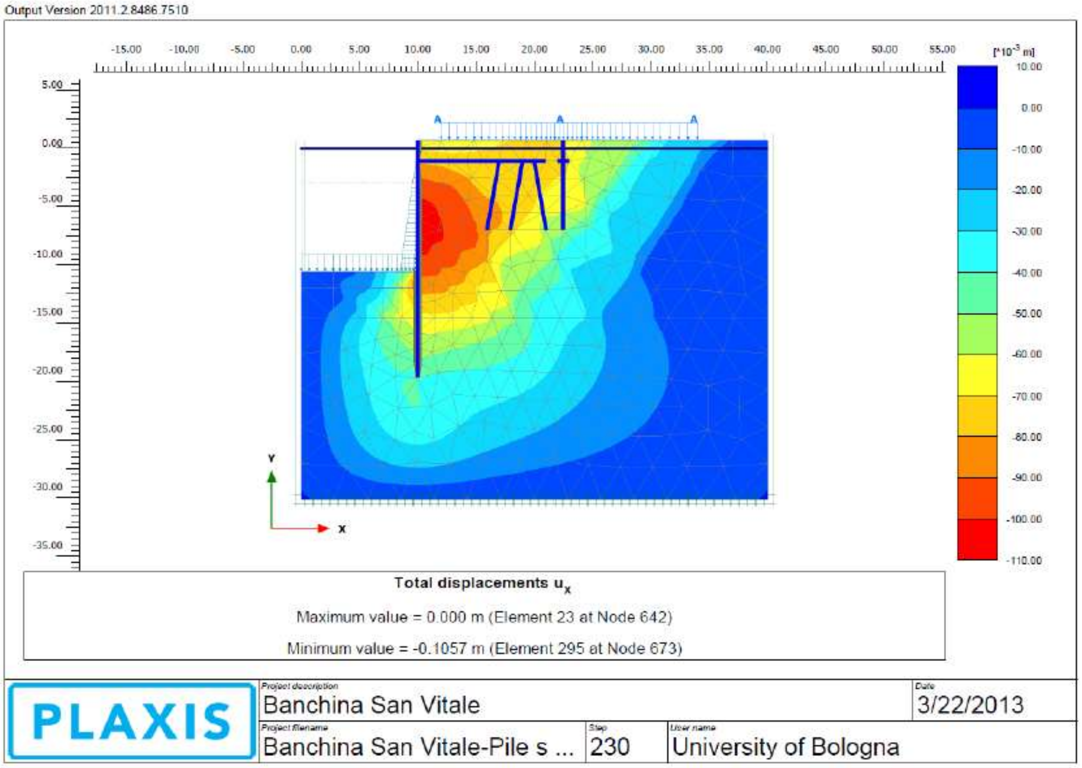


Figure 5.7 *Horizontal Displacements for the Shortened piles.*

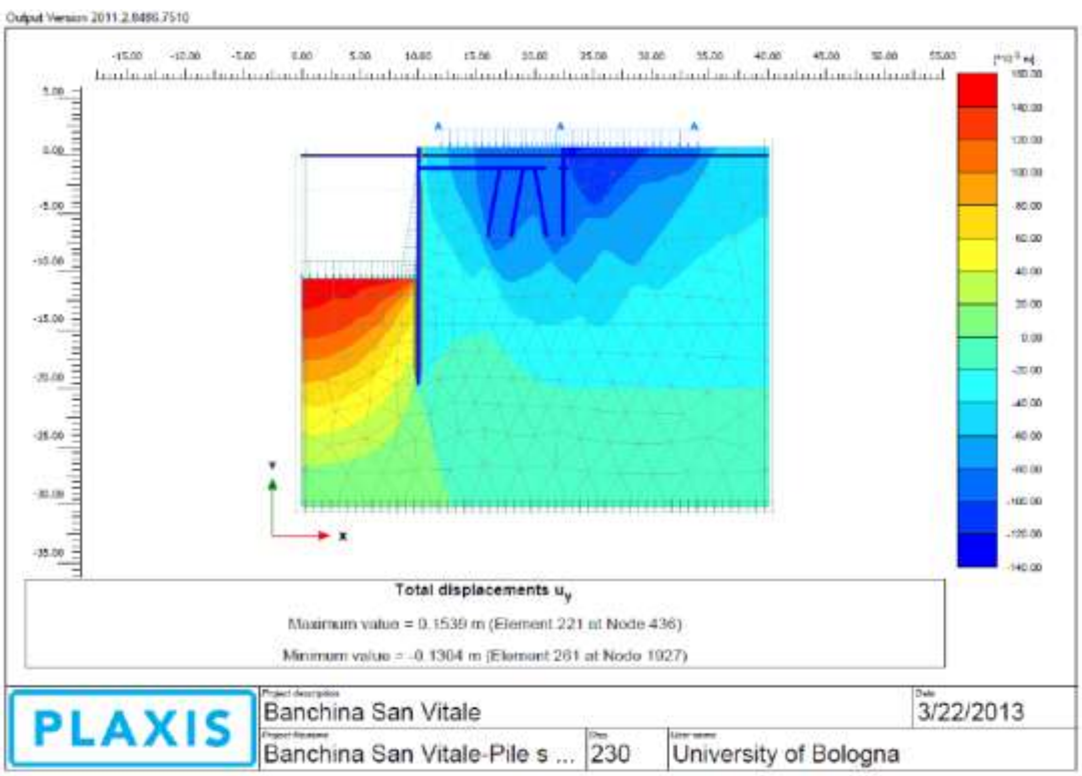


Figure 5.8 *Vertical Displacements for the shortened piles.*

For the second part, the model has been developed with the shortened piles and the anchor has been installed at the depth of -8.50 m. After that the bottom of the excavation has been deepened to -12.00 m. The geometry of the model is shown in Figure 5.9. The following calculation steps have been performed, but only results for the final stages are presented in the following (Figures 5.10 and 5.11).

- Step 6: Activate anchor at level -8.50 m and prestressed by 300 kN/m
- Step 7: Excavation to level -12.00 m
- Step 8: Apply surcharge load (permanent load of 40 kPa)

The summary of the results for this part is shown in Table 5.5.

Table 5.5 *Summary of the results for final phase*

Phase	Max. Horizontal Displacement (cm)	Max. Vertical Displacement (cm)	Max. Force in Anchor (kN/m)	F.S
Phase 1 (Full Length Piles)	9.93	13.47	357	1.33

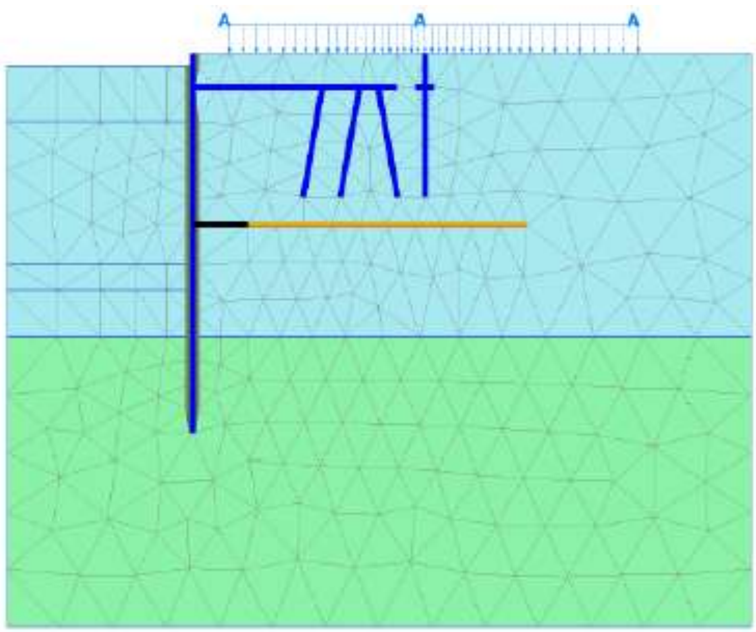


Figure 5.9 *Final Finite Element Model generated by PLAXIS 2D.*

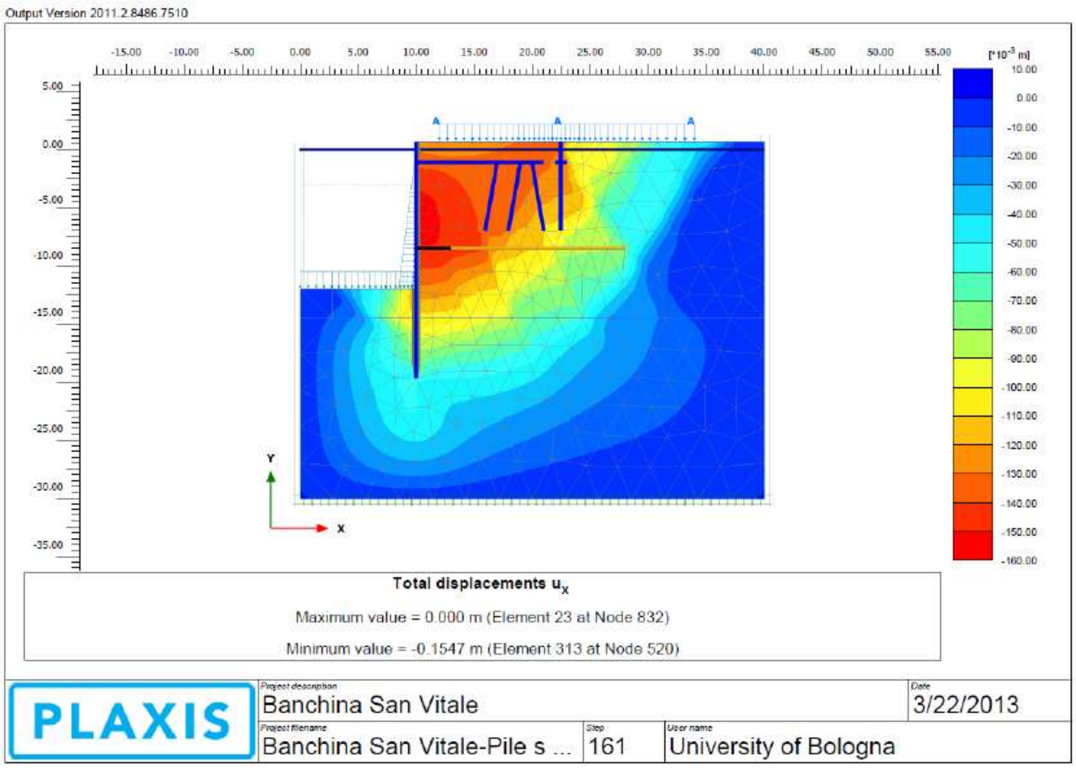


Figure 5.10 *Horizontal Displacements for the final model.*

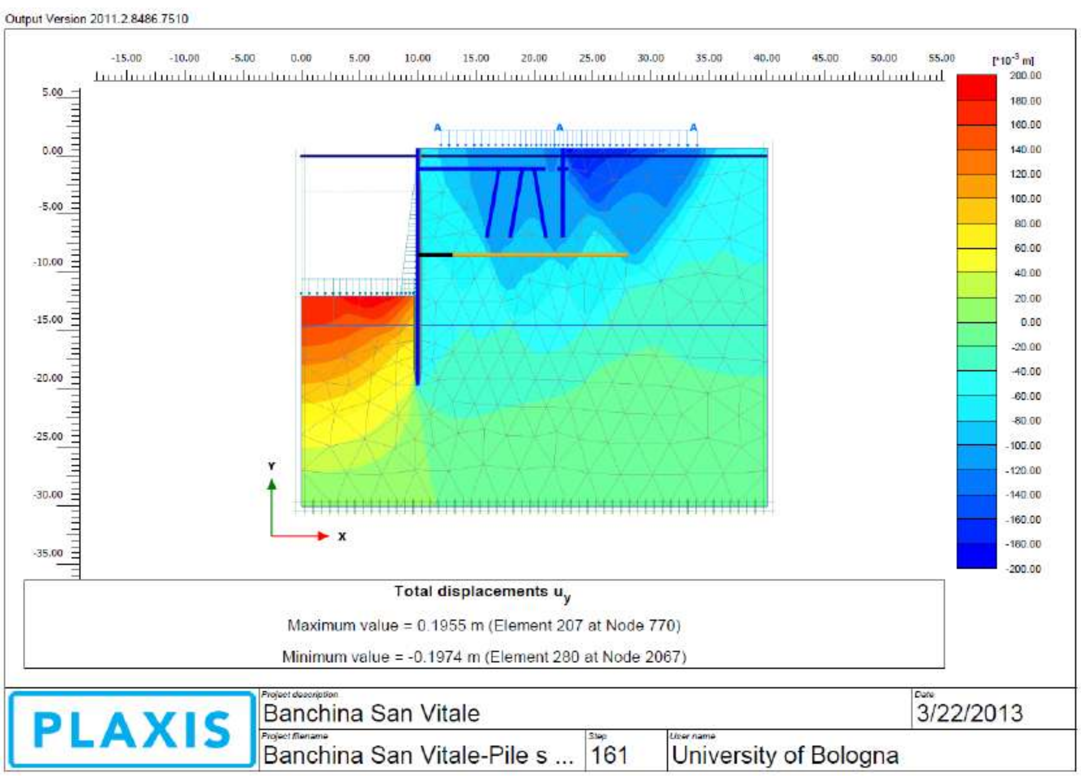


Figure 5.11 *Vertical Displacements for the final model.*

3D Analysis

The finite element method is well established in the current geotechnical engineering practice. Although most calculations are still 2D, there is a tendency to model complicated situations in more detail using 3D models. 3D numerical analysis becomes an increasingly affordable tool for predicting deformations and stress distribution in special geotechnical projects. As excavation is clearly a three dimensional problem, considering the third dimension should intuitively lead to more accurate predictions. However, simplified procedures that allow us to consider 3D effects within a simplified 2D plane strain analysis are still popular in geotechnical design (Mair, 2008).

For the 3D modeling, the model has been developed with the all details such as full length of the piles, horizontal and vertical slabs, 3D node to node anchors and embedded vertical and inclined piles. Also it should be mentioned that there was not any intersection between the piles and anchors. Since the horizontal distance between piles were 2.50 m and the horizontal distance between anchors were 2 m, to achieve a constant module the model has been developed by a length of 10 m in horizontal direction. The geometry of the model and structural elements of the model has been shown in Figures 5.12 and 5.13 respectively.

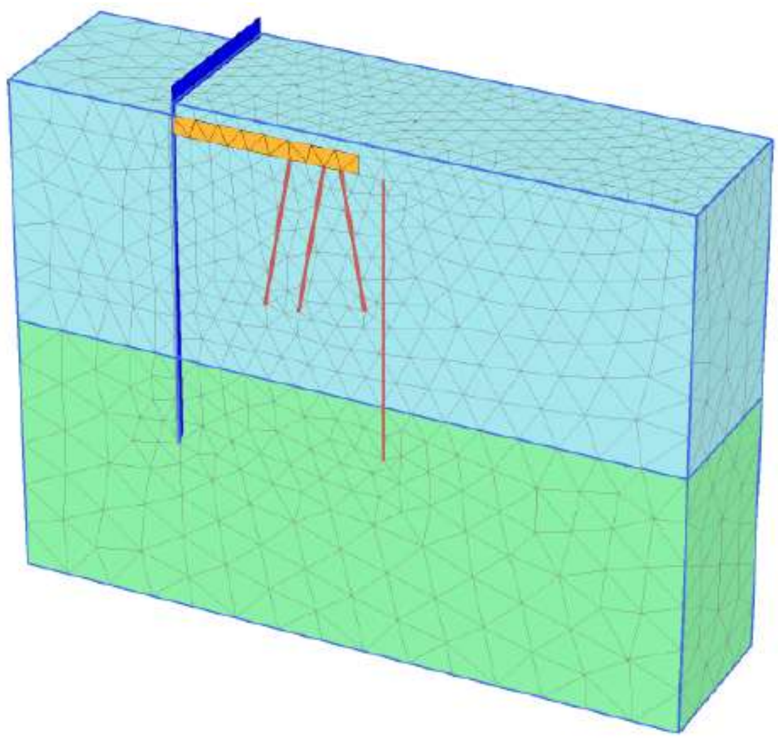


Figure 5.12 *Geometry of the model in PLAXIS 3D.*

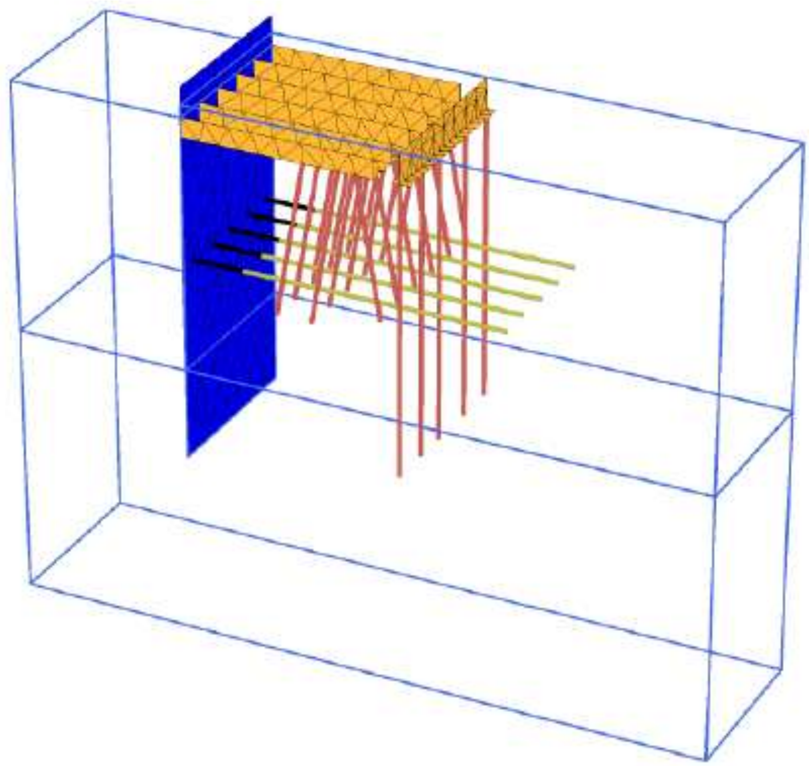


Figure 5.13 *Structural elements of the model in PLAXIS 3D.*

The results have been investigated through the predictions of horizontal displacements induced along the sheet pile wall as well as ground surface settlements and the original anchored force and calculated factor of safety. The following calculation steps have been performed, but only results for the final stages are presented in the following (Figures 5.14 and 5.15).

- Step 0: Initial stress state ($\sigma'_v = \gamma.h$, $\sigma'_h = K_0\ \sigma'_v$, $K_0 = 1 - \sin\phi'$)
- Step 1: Activate sheet pile wall
- Step 2: Excavation to level -3.0 m
- Step 3: Activate horizontal and vertical slabs and piles at level -1.1 m
- Step 4: Excavation to level -10.50 m
- Step 5: Apply surcharge load (permanent load of 40 kPa)
- Step 6: Activate anchors at level -8.50 m and prestressed by 300 kN/m
- Step 7: Excavation to level -12.00 m
- Step 8: Apply surcharge load (permanent load of 40 kPa)

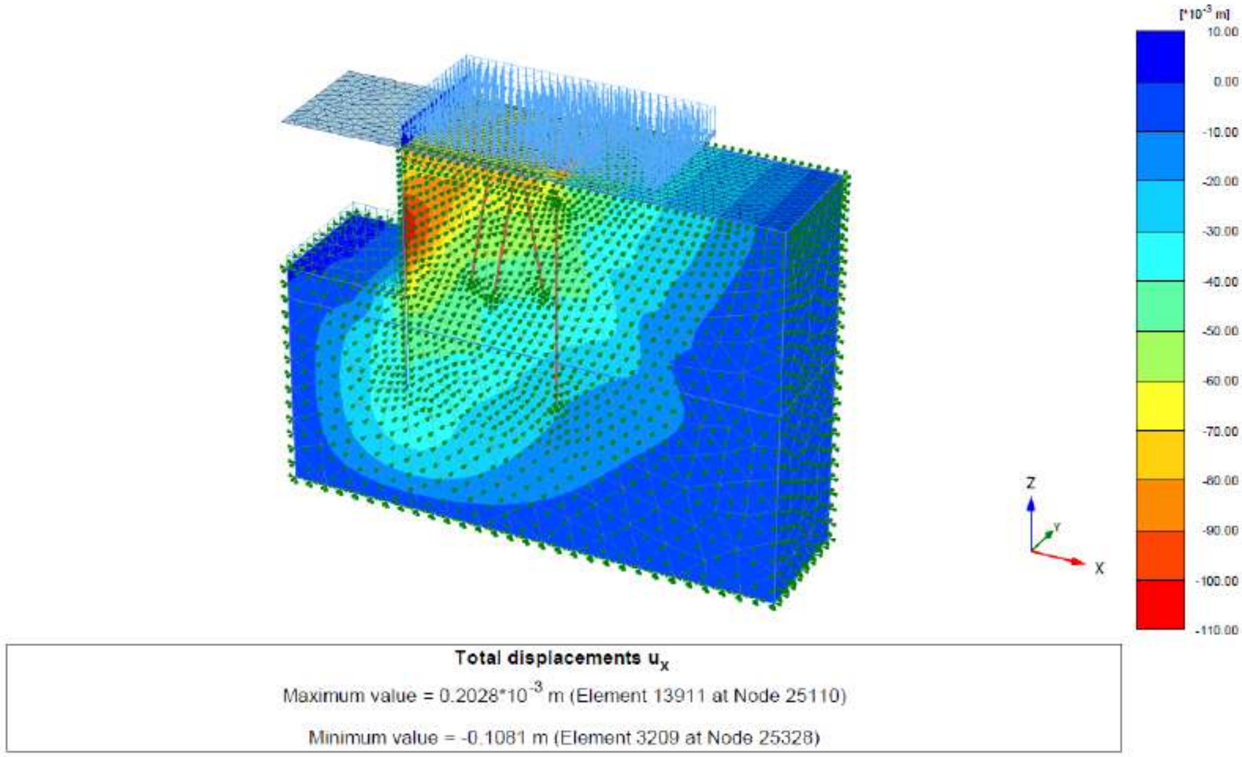


Figure 5.14 *Horizontal displacements of the model in PLAXIS 3D.*

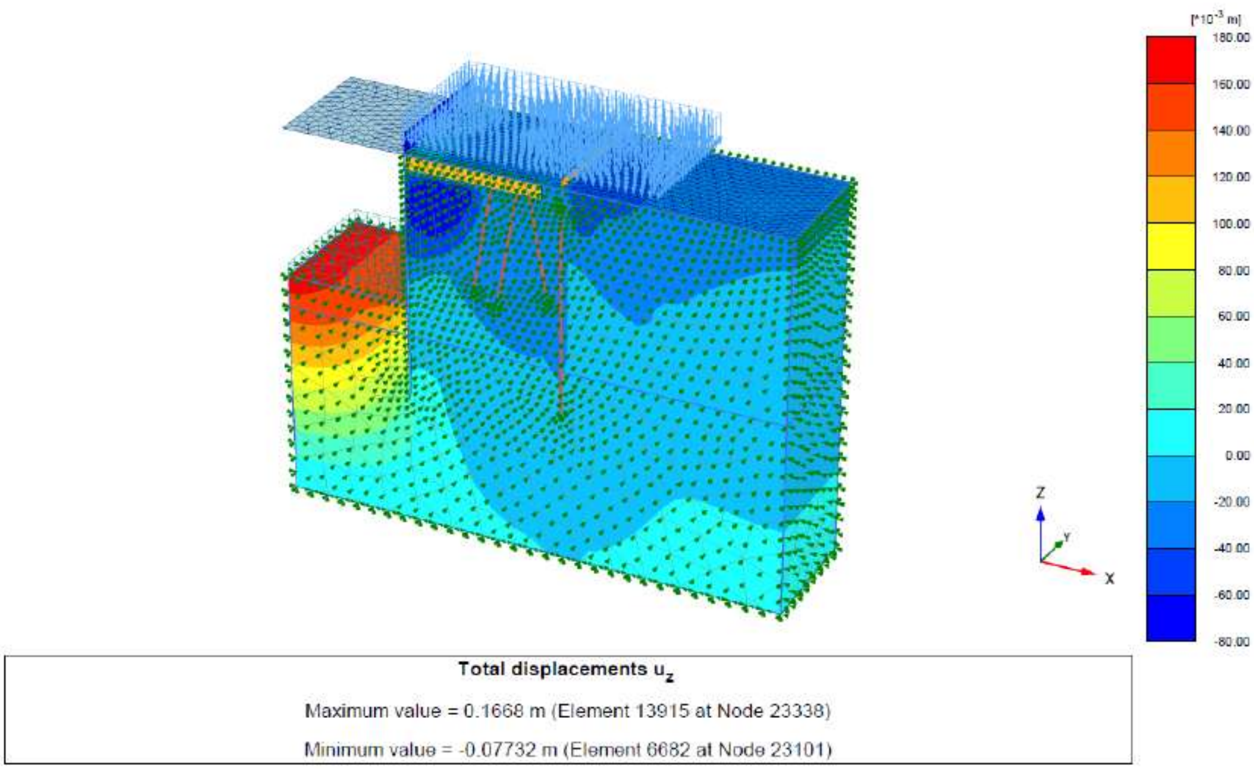


Figure 5.15 *Vertical Displacements of the model in PLAXIS 3D.*

The results and comparison of the 2D model and 3D model have been summarized in Table 5.6.

Table 5.6 *Summary of the results and comparison of 2D and 3D model*

Model	Max. Horizontal Displacement (cm)	Max. Vertical Displacement (cm)	Max. Force in Anchor (kN/m)	F.S
2D Model	15.47	19.74	357	1.26
3D Model	10.81	7.73	347	1.56

5.4 CONCLUSION

Taking all the results into account, comparing 2D and 3D modelings, it could be concluded that:

- The maximum horizontal displacements have been decreased 30% in 3D modeling.
- The maximum vertical displacements have been decreased 60% in 3D modeling in comparison with 2D model.
- The factor of safety has been increased 24% and the maximum force in anchors has not been changed significantly.

It is worth mentioning that according to the results it seems that with the 3D modeling we obtain more realistic results that it could be verified with further investigations. These particular results may be due to the 3D modeling, which is based on a more precise representation of the structural elements, including particulars that could not be represented in the 2D case, such as vertical slabs that make the subsurface slab more rigid, and also using the full length piles in all phases.

CHAPTER 6
DESIGN OF EXCAVATION WITH FEM - INFLUENCE OF CONSTITUTIVE MODEL

6.1 INTRODUCTION

The capability of giving a reliable prediction on wall deflections prior to excavation is very import, because it can help designers and practitioners evaluate the effects on the adjacent structures and facilities and then adopt appropriate countermeasures. Traditionally, there are two types of methodologies for estimating wall deflections due to excavation:

(1) Numerical analysis such as FE modeling;

(2) Empirical or semi-empirical methods.

Compared to numerical analysis, empirical and semi-empirical methods (e.g., Peck 1969; Clough and O'Rourke 1990) provide probably less accurate but more straightforward estimations, and thus are favored by designers and practitioners. As empirical and semi-empirical methods are based on local field data and did not account for the effects of some important factors (e.g., soil properties, types of struts, duration time of excavation, and structures or facilities nearby), it is difficult for them to give a reliable prediction under complex construction conditions (Lu et al., 2012). The acceptance of numerical analyses in geotechnical problems is growing and finite element calculations are more and more used in the design of foundations (Wehnert and Vermeer, 2004). Numerical analyses are widely used in practical geotechnical engineering to assess the deformation behavior of excavations, in particular when the influence on existing infrastructure has to be evaluated. In addition it becomes increasingly common to use results from numerical analysis as basis for the design. When doing so, compatibility of the design with relevant standards and codes of practice, valid in the respective country, has to be assured. In general this is a well established procedure when employing conventional design calculations based e.g. on limit equilibrium methods, but there are no clear guidelines how this can be achieved when numerical methods are used (Schweiger, 2010). Thus not much literature is available on this issue although some attempts have been made (e.g. Bauduin et al. (2000), Schweiger (2005, 2009), Simpson (2000, 2007)). An additional difficulty arises, namely the appropriate choice of the constitutive model for the soil, which has a direct consequence for the design because different constitutive models will lead to different design forces. Until now, numerous constitutive models (e.g., Mohr-Coulomb (MC) model, Duncan-Chang (DC) model (Duncan and Chang 1970), Drucker-Prager (DP) model (Drucker and Prager 1952), Modified Cam-Clay (MCC) model (Roscoe and Burland 1968), Hardening-Soil (HS) model and Soft-Soil-Creep (SSC) model of PLAXIS (Brinkgreve et al. 2002) have been developed to characterize soil behaviors during construction. Input parameters for constitutive models commonly derive from laboratory tests (e.g., oedometer tests and triaxial tests), in which soil behavior is just controlled by stress levels. In reality, soil strength and stiffness are affected by both stress and strain levels. Degradation of soil stiffness is not only a function of stress levels, but also strain levels. In addition, some levels of disturbance on soil strength and stiffness will be introduced by construction activities. Therefore, the actual soil behaviors during construction are different from the stress-strain data obtained from laboratory tests. Considering that soils might exhibit non-linear stress-strain responses even at very small strains, some researchers developed more advanced and complex small-strain constitutive models (Finno and Tu 2006). However, these small-strain models require amount of input parameters, some of which have to be determined from special laboratory tests. This limits their application to practice use.

The objective of the present study is to investigate the behavior of anchored sheet pile walls constructed in cohesionless soils by different constitutive models. The objective was achieved by a series of analyses through the finite element program, PLAXIS 2D, version 2012. A comprehensive study was carried out by using different constitutive models to simulate the mechanical behavior of the soil to investigate the enhancement of using submerged grouted anchors technique on the load response of sheet pile quay wall by varying anchor location. This parameter, which affect the performance of sheet pile quay wall, were considered and evaluated through prediction of horizontal displacements and bending moments induced along the sheet pile wall as well as the original anchored force and calculated factor of safety.

6.2 NUMERICAL APPROACH

Numerical analyses are widely used in practical geotechnical engineering to assess the deformation behavior of excavations. In addition it becomes increasingly common to use results from numerical analysis as basis for the design. With the advances in computing technology, the use of continuum mechanics numerical methods in the analysis and design of sheet piles has been increasing in recent years. The finite element method has been utilized by researchers to study and understand the behavior of cantilever, braced, and anchored sheet pile walls under static and dynamic loading conditions (Bilgin, 2010). The finite element modeling comprised two-dimensional plane strain analysis and analyses were carried out using PLAXIS.

In this chapter the constitutive models used in the analyses are briefly introduced. The sheet pile wall was simulated by a plate-element model and grouted anchors are modeled by the node-to-node anchor element, while the grout body is simulated by the geo-grid element. The soil behavior is described by the Mohr-Coulomb (MC) and by the Hardening-Soil (HS) model. Due to a stress concentration in and around the wall, a finer finite element mesh was used in these areas and mesh became coarser in the zones away from the wall. The finite element model used for the present work is shown in Figure 6.1.

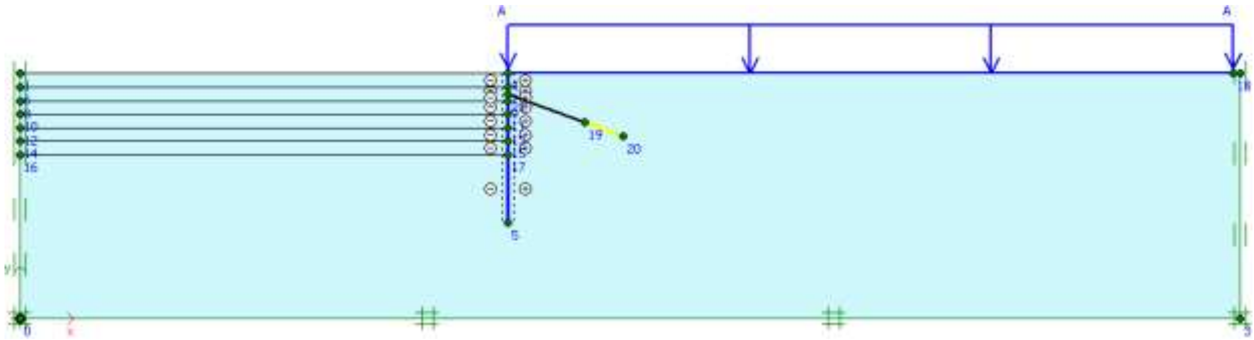


Figure 6.1 *Finite Element Model as Generated by PLAXIS.*

6.2.1 Constitutive theory

The constitutive relationship for a material depends on the homogeneity, isotropy and continuity of the body material, as well as its response to cyclic loading, and the rate and magnitude of the applied load (Verghese et al., 2013). General techniques have been developed to characterize soil as elastic, plastic, or viscous in nature to consequently conduct constitutive analyses.

Theory of Plasticity

Classical theory developed by Hill 1950 seeks to explain the stress and strain behavior of plastically deformed solids and is fundamentally analogous to Hooke's law which stipulates the relationship between stress and strain governed by the material's modulus. It is important to note that a material's total strain rate is controlled by an elastic and plastic rate component.

$$\dot{\sigma} = \underline{M}(\dot{\varepsilon^e} + \dot{\varepsilon^p}) \qquad (6\text{-}1)$$

The yield limit of an elastic soil material is defined by a yield function, denoted f, and is a function of the stress components, friction angle, φ and cohesion, c. The failure limit under all deviatoric loading combinations for a perfectly plastic material remains fixed and does not move in principal stress space. Hill 1950 states that plastic strain rates are proportional to the derivative of the yield function with respect to the stresses. This notion provides the basis upon which plastic deformation can be determined once the stress point (p-q) reaches the yield surface.

$$\dot{\varepsilon^p} = \lambda \frac{\partial f}{\partial \dot{\sigma}} \qquad (6\text{-}2)$$

Experimental data indicates that plastic strain rates are not always orthogonal to the yield surface and hence cannot be accommodated under the coaxial assumption. Therefore, the plastic potential function has been derived to model this type of plastic strain rate behavior and considers the dilatancy angle, ψ, where φ ≠ ψ, to avoid the previous overestimation of dilatancy under the normality rule. The yield condition for both MC model and HS model is an extension of Coulomb's friction law to general states of stress (Wehnert and Vermeer, 2004).

Mohr-Coulomb Soil Model

The MC model in Plaxis captures the linear elastic perfectly plastic stress-strain behavior of a soil element when considered in its general stress state, and all deformations are fully recoverable upon unloading. Once the stress point (p-q) is loaded past the model's elastic limits, φ and cohesion, c, define a fixed shear failure surface upon which the stress point (p-q) is assumed to follow.

$$\tau = \sigma_m{}'\tan(\varphi') + c' \tag{6-3}$$

The above failure criterion produces a linear failure envelope for a two-dimensional analysis. As the development of plastic strains occurs, the yield surface does not admit changes of expansion or contraction and hence is considered a fixed yield surface (perfectly plastic). The basic ideas of the MC model is shown in Figure 6.2.

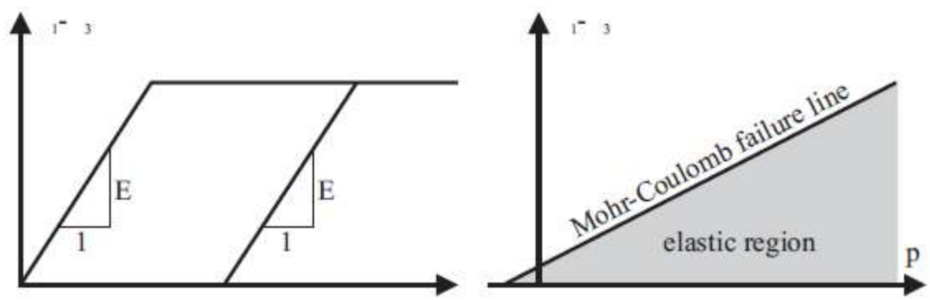


Figure 6.2 *Basic ideas of the MC model*

Hardening Soil Model

The HS model in Plaxis is a second order constitutive relationship that seeks to describe the non-linear behavior of soil upon yielding and is derived from the hyperbolic model of Duncan and Chang (Verghese et al., 2013). The term hardening within this context defines the various changes, in size, location or shape, of the yield surface and is directly related to the loading history measured by a form of plastic deformation. Note, the Plaxis HS model considers isotropic hardening only.

Isotropic hardening is governed by the assumption that expansion or contraction about the centre of the yield surface is uniform, whilst the shape, centre and orientation of the yield surface remain unchanged. Fundamentally this type of hardening behavior can be characterized into two components: shear hardening and compression hardening. When a soil body is subjected to primary deviatoric loading, plastic axial strains develop as observed in a triaxial test. These irreversible strains are consequently accounted for by the shear hardening component of the HS model. Furthermore, compression hardening is used to model the irreversible plastic volumetric strains due to primary compression in oedometer loading and isotropic loading. Careful consideration to stress dependent stiffness is a fundamental element of the HS model and is associated with a power law value, m. This power value equates to 1 when a linear analysis is assumed and is typically taken as 0.5 (recommended for hard soils) when attempting to model hyperbolic stress-strain behavior.
This model requires three input stiffness parameters, namely: E_{50}, E_{oed}, and E_{ur} (Figure 6.3).

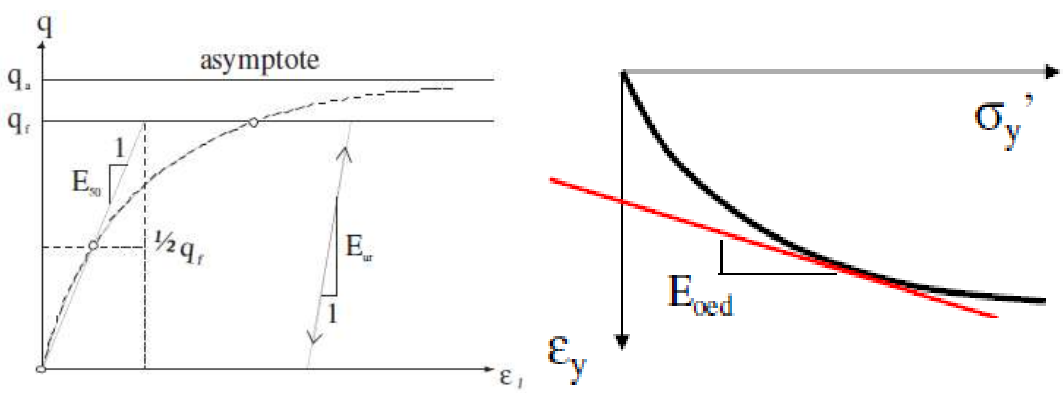


Figure 6.3 *Definitions of E50, Eur and Eoed*

E_{50} is the confining stress dependent stiffness modulus for primary loading and is described by Eq. (6-4).

$$E_{50} = E_{50}^{ref} \left(\frac{c'.cot\varphi' + \sigma_3{}'}{c'.cot\varphi' + P^{ref}}\right)^m \qquad (6\text{-}4)$$

For unloading and reloading elastic stiffness, E_{ur} is defined as follows:

$$E_{ur} = E_{ur}^{ref} \left(\frac{c'.cot\varphi' + \sigma_3{}'}{c'.cot\varphi' + P^{ref}}\right)^m \qquad (6\text{-}5)$$

The oedometer stiffness modulus, E_{oed} for primary compression is:

$$E_{oed} = E_{oed}^{ref}\left(\frac{c'.cot\varphi'+\sigma_1{}'}{c'.cot\varphi'+p^{ref}}\right)^m \quad (6\text{-}6)$$

Note, p^{ref} is defined as the reference pressure and is taken as 100 kN/m^2 within the Plaxis FEM software (PLAXIS 2012). E_{50}^{ref} , E_{ur}^{ref} , E_{oed}^{ref} and are the respective stiffness moduli corresponding to p^{ref}.

The first type of hardening, shear hardening has a linear flow relationship and is characterized by the development of plastic strains when mobilizing the soil's material strength, or increasing the soil's preconsolidation stress, commonly referred to as compaction hardening. The fundamental shear hardening yield function is given by (PLAXIS 2012):

$$f = \bar{f} - \gamma^p \quad (6\text{-}7)$$

where $\bar{f}$ is a function of stress and γ^p is the strain hardening parameter and is expressed as a function of plastic strains (PLAXIS 2012):

$$\bar{f} = \frac{2}{2E_{50}}\frac{q}{1-\frac{q}{q_a}} - \frac{2q}{E_{ur}} \quad (6\text{-}8)$$

$$\gamma^p = -(2\varepsilon_1^p - \varepsilon_v^p) \quad (6\text{-}9)$$

For hard soils, plastic volume changes tend to be relatively small and hence ε_v^p can be assumed to equal 0 (hard soils only). Hence, the combination of Eq. 6-8 and Eq. 6-9 produces multiple yield surfaces with increasing values of γ^p as seen in Fig 6.4.

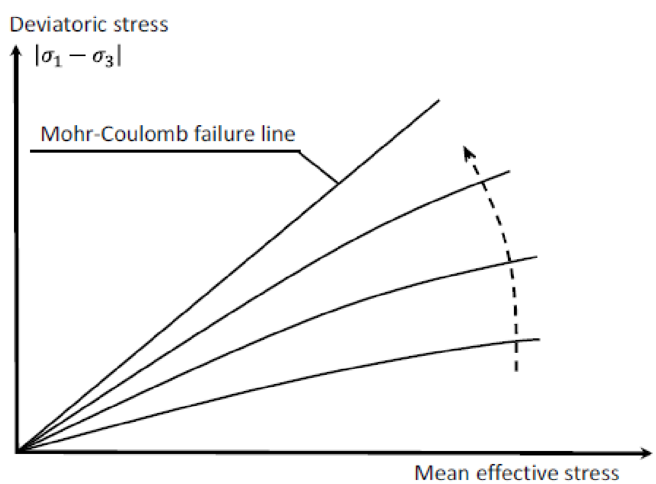


Figure 6.4 *Yield loci for varying constant values of the* $\boldsymbol{\gamma^p}$ *parameter*

The second type of hardening mechanism is plastic volumetric strain. As an element of soil undergoes compressive loading, a stress point asymptotically follows the yield locus respective to its strain hardening parameter, thus the induction of a yield surface limits the elastic region upon which the stress point is asymptotically moving, based on a direct relationship between E_{50}^{ref} and E_{oed}^{ref} . Hence, the shear yield surface is controlled by the triaxial modulus and the cap yield surface by the oedometer modulus respectively (PLAXIS 2012).

6.2.2 Soil properties

One of the most important parameters in the numerical modeling is properties of soil profile. As mentioned before for the excavation in the sand layer two different constitutive models have been employed, namely the simple Mohr-Coulomb failure criterion (MC), the standard Plaxis Hardening Soil model (HS). The parameters are listed in Table 6.1. Strength parameters are the same for all models but stiffness parameters are different. They are stress dependent in the HS model (values in Table 1 are reference values) but constant in the Mohr-Coulomb model.

Table 6.1 *Required parameters for calculation*

Parameter	Meaning	Value
γ_{sat} [kN/m^3]	Saturated unit weight	21
γ [kN/m^3]	Unsaturated unit weight	19
ϕ' [°]	Friction angle	30
ψ [°]	Dilatancy angle	2
v [-]	Poisson’s ratio	0.3
E [MPa]	Modulus of elasticity	30
E_{50}^{ref} [MPa]	Secant modulus for primary triaxial loading	30
E_{oed}^{ref} [MPa]	Tangent modulus for oedometric loading	30
E_{ur}^{ef} [MPa]	Secant modulus for un- and reloading	90
m [-]	Exponent of the Ohde/Janbu law	0.50
P_{ref} [kPa]	Reference stress for the stiffness parameters	100
K_0^{nc} [-]	Coefficient of earth pressure at rest (NC)	0.50
R_{inter} [-]	Interface strength, R_{int}	0.67

6.3 RESULTS AND DISCUTION

It is concluded from this numerical modeling that the location of grouted anchor from ground surface is one of the most dominant factors affecting the behavior of the anchored sheet pile quay walls. In the following the results of these analyses in terms of wall deformations, wall bending moments, anchor forces and calculated factor of safety obtained from different location of the anchor is presented.

To investigate the role of grouted anchor depth on the enhancement of sheet pile quay walls, four values of h_a have been adopted, (h_a = 1, 3, 5 and 7 m). The following calculation steps have been performed for the analyses.

- Step 0: Initial stress state ($\sigma'_v = \gamma.h$, $\sigma'_h = K_0\ \sigma'_v$, $K_0 = 1 - \sin\phi'$)
- Step 1: Activate sheet pile wall
- Step 2: Excavation to level -2.0 m (-4.0, -6.0 and -8.0 m for different models)
- Step 3: Activate grouted anchor at level -1.0 m (-3.0, -5.0 and -7.0 m for different models) and prestressed by 100 kN/m
- Step 4: Excavation to level -12.00 m
- Step 5: Apply surcharge load (permanent load of 40 kPa)

The results showed a considerable enhancement in the performance of the structure due to an increase in grouted anchor depth. Since in this study just a single row of anchor has been used, the results present that an increase in grouted anchor depth increases significantly the maximum horizontal displacement of wall face.

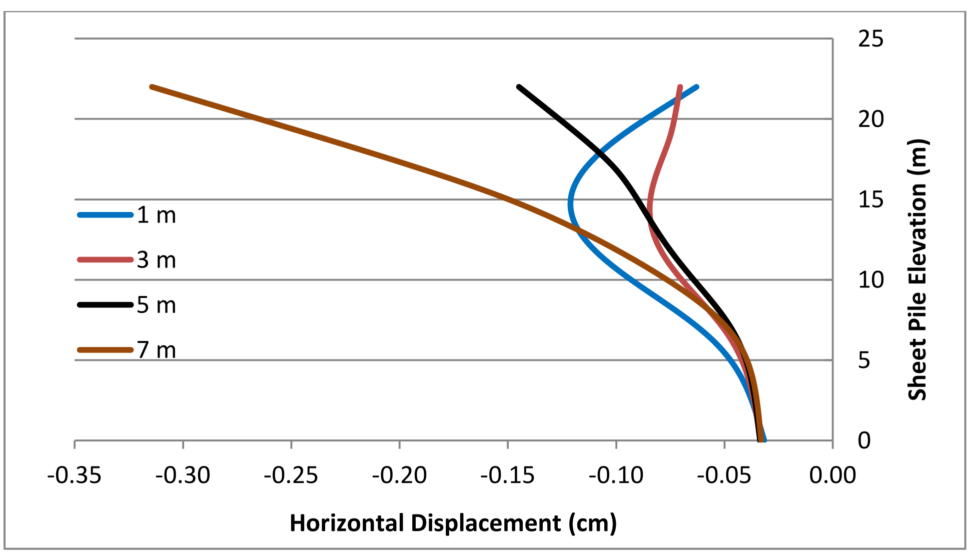


***Figure 6.5** Horizontal displacement of the wall versus different location of anchor*

According to figure 6.5 as the depth of the anchor increases, obviously the location of the maximum horizontal displacement occurs at the top of the wall.

Figure 6.6 presents the results of different values of grouted anchor location versus maximum anchor force. From these results, it can be noticed that the increase of the depth of the anchor increases the maximum force in anchor.

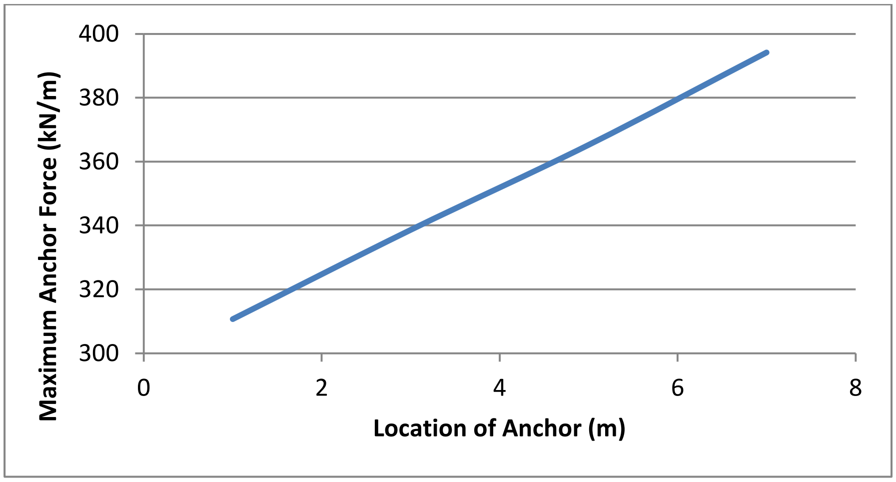


Figure 6.6 *Maximum anchor forces versus different location of anchor*

Figure 6.7 presents the effect of grouted anchor location on the bending moment along the sheet pile wall. The results show that an increase of the depth of the anchor decrease in the positive maximum bending moment, but an increase in the negative maximum bending moment.

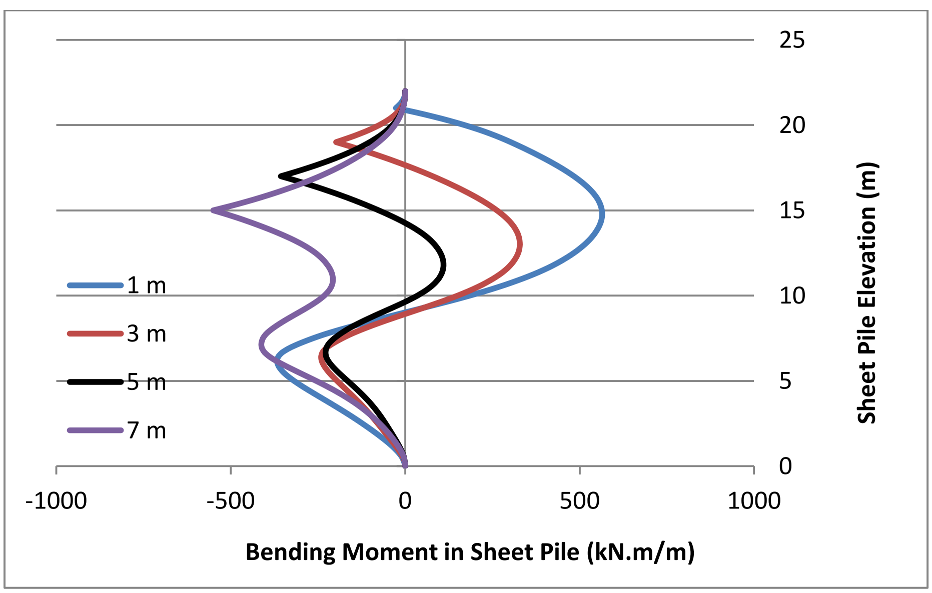


Figure 6.7 *Bending moment along the wall versus different location of anchor*

Taking all the absolute amount of maximum bending moment in to account, it can be concluded that the best location for the anchor is between -3.0 m and -5.0 m.

Figure 6.8 illustrates the results of different values of grouted anchor location versus calculated factor of safety. The results show that, as the depth of the anchor increased, the calculated factor of safety increased considerably. The former observations indicate that, increasing the grouted anchor depth generally enhances the performance of the structure, but a trade-off is required to balance the increase of original anchor force. This enhancement could be attributed to the contribution of location of the anchor, which helps to increase the overall stiffness of the structure and thus decrease the internal forces induced in the sheet pile quay wall system.

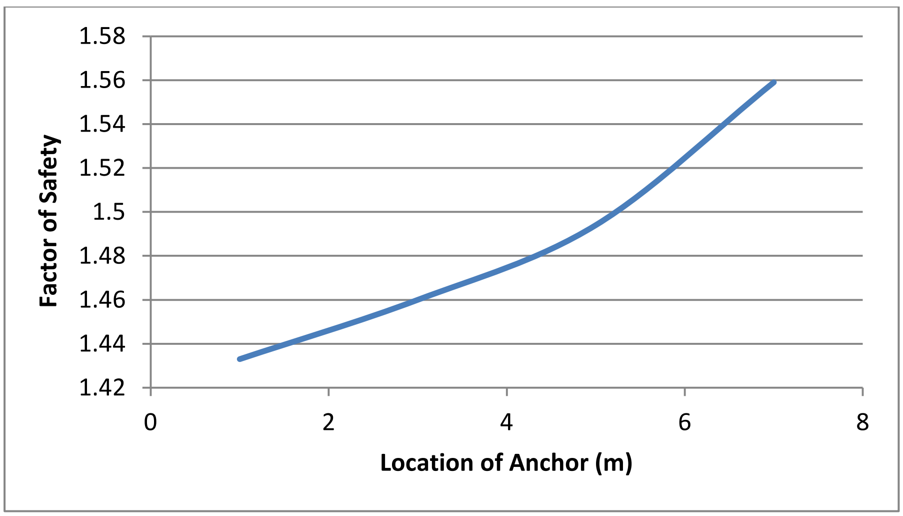


Figure 6.8 *Calculated factor of safety versus different location of anchor*

Comparison between MC model and HS model

For the comparison between the Mohr-Coulomb (MC) and Hardening-Soil (HS) model the model with the anchor depth of -3.0 m was chosen. Figure 6.9 shows the lateral displacement of the sheet pile wall for the final excavation stage. It is observed that the MC model predicts the larger maximum displacement but of course this strongly depends on the chosen elasticity modulus. The maximum deflection value predicted by MC model was approximately 8.44 cm in comparison to the maximum horizontal displacement by the HS model that was 7.60 cm (Figure 6.10). Firstly, the MC model provides a first order prediction of the excavation procedure and consequently assumes linear elastic behavior during the excavation process.

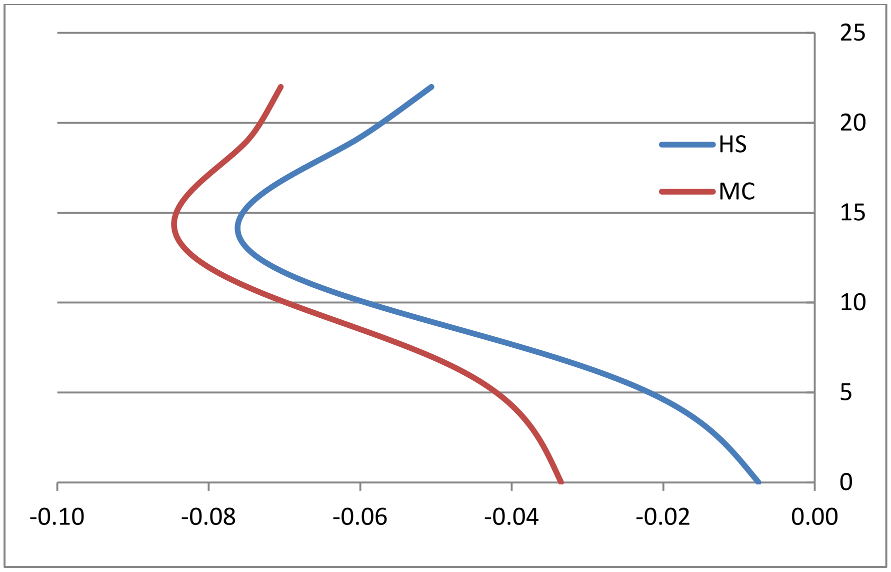


Figure 6.9 *Comparison of wall horizontal displacements with MC and HS soil models.*

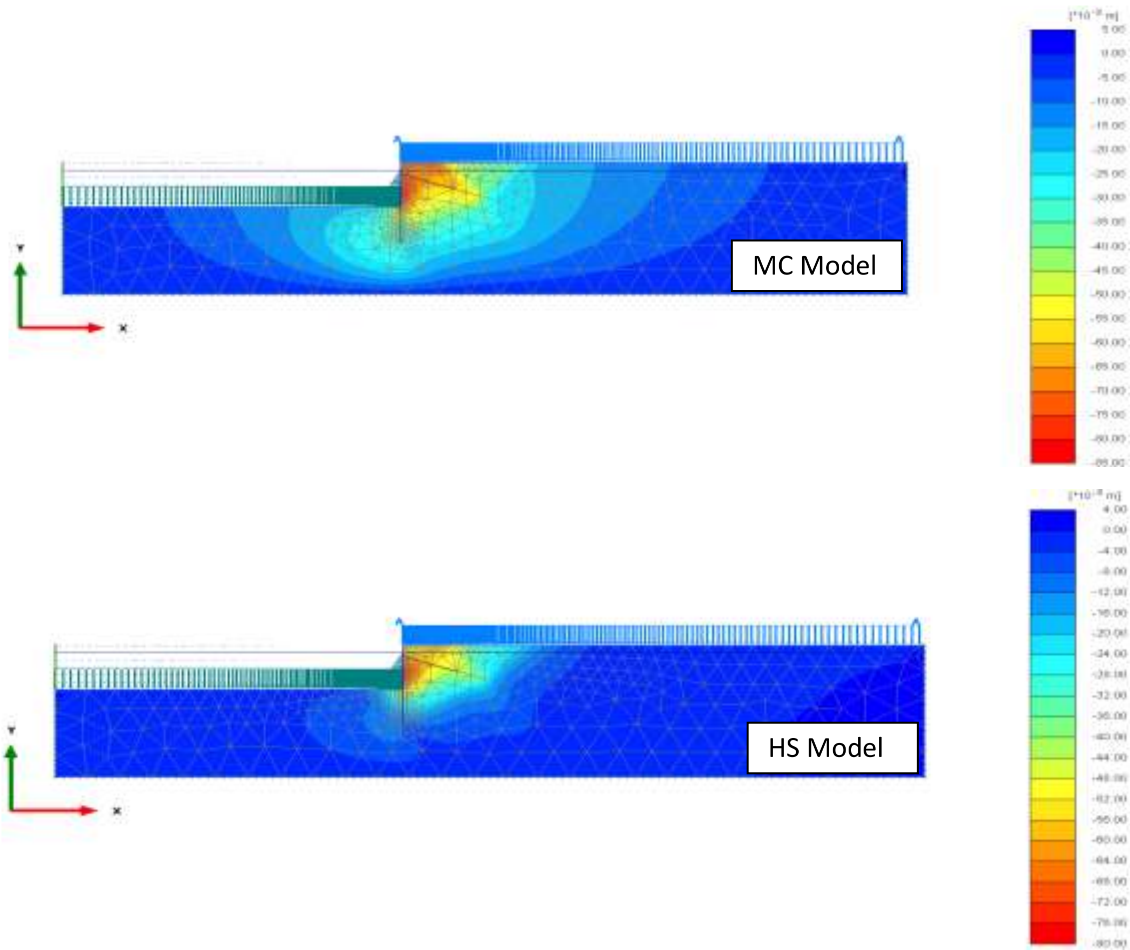


Figure 6.10 *Comparison of MC and HS soil models for horizontal displacement.*

Within the strain range of 10^{-3}-10^{-6}, it is common for soils to exhibit non-linear stress-strain behavior where the variability of stiffness is readily observed. This does not indicate the soil has fully yielded, because the soil is able to almost recover initial stiffness upon unloading, it does however behave differently to the MC assumption of linear elasticity. This results of horizontal displacements occurring behind the retaining wall since the soil stress path remains in the linear elastic domain and the development of plastic shear strains are erroneously overlooked. A key feature of the HS model is its capability to capture the non-linear elastic stress-strain relationship that is typically observed in soils before yielding. This type of hardening mechanism behavior is referred to as deviatoric hardening. When a triaxial test is performed on a soil sample, the shear hardening phenomenon is observed due to the development of shear strains under confining stress conditions. This type of confining stress state is generally experienced by a soil element located behind a retaining wall, where the lateral resistance of the diaphragm wall, coupled with lateral soil pressure induces a deviatoric stress. As the development of plastic deformation increases, the degradation of soil stiffness occurs, thus resulting in diaphragm wall deflection and surface settlement. The MC model is unable to explain this type of behavior due to its elastic framework which restricts the modeling of plastic shear strain influence.

A reverse trend is observed for bending moments (Figure 6.11). Table 6.2 summarizes the results of maximum anchor force and calculated factor of safety for the MC and HS model.

Table 6.2 *Summary of the effect of grouted body length on structure behavior*

Model	**Max. Anchor Force** *(kNm/m)*	**F.S**
MC	339	1.46
HS	377	1.48

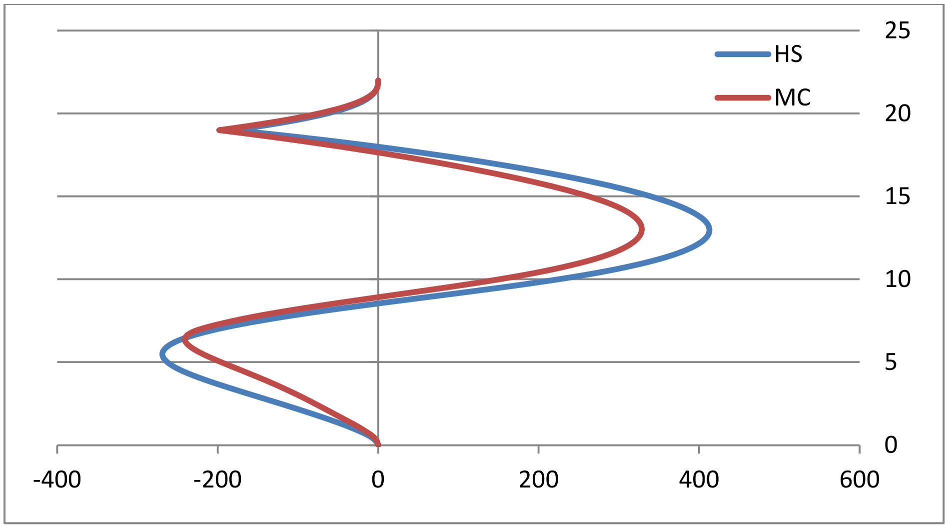


Figure 6.11 *Comparison of wall bending moments with MC and HS soil models.*

6.4 CONCLUSION

A numerical study using finite element analysis was conducted to investigate the effect of location of the anchor and the influence of the constitutive model on the behavior of anchored sheet pile walls. Based on the results of presented study, several conclusions can be drawn, which are useful for enhancement of the sheet pile quay walls in cohesionless soils. The rehabilitation of sheet pile quay walls using additional grouted tie-rods has a significant role on the performance of deepened quay wall systems. The performance of the sheet pile wall will increase with the increase of the depth of the grouted anchor in terms of factor of safety and final excavated depth. Also the maximum bending moment occurring along the sheet pile wall has been considerably reduced by increasing the depth of the anchor up to -5.0 m for this case but obviously, this leads to an increase of the maximum original anchor force and maximum horizontal displacement. Therefore, a trade-off is required to balance the increase of original anchor force and horizontal displacement and the reduction of maximum bending moment and factor of safety.

In the second part of this contribution the influence of the constitutive model on the results of finite element analyses of an excavation has been demonstrated. The results illustrate that the elastic-perfectly plastic constitutive models such as the Mohr-Coulomb model are not well suited for analyzing this type of problems and more advanced models are required to obtain realistic results. Strain hardening plasticity models are in general a better choice and produce settlement troughs being more in agreement with expected behavior.

CHAPTER 7
CONCLUSION

7.1 CONCLUSION

A series of numerical studies using finite element analysis were conducted to investigate the behavior of anchored sheet pile walls. The effect of several parameters on the wall and soil movements and internal forces of the structure was investigated. The parameters considered were; anchor depth, anchor inclination, grouted body length, grouted body area and effect of constitutive model. Based on the results of presented study, several conclusions can be drawn:

1- It was found that finite element software, PLAXIS is a powerful tool for investigating the behavior of a stabilized wall by soil anchorage and sheet pile wall.

2- The rehabilitation of sheet pile quay walls using additional grouted tie-rods has a significant role on the performance of deepened quay wall systems. The anchored wall system and surrounding soil show more stabilized behavior when the grouted anchors are used.

3- The effect of anchorage is so clear that the lateral movement can be kept within permissible values. Plaxis generally overestimates the lateral deflections and similar studies bring up same conclusions but thanks to Plaxis the rather large deformations can be predicted in good agreement with the real behavior. This helps to avoid extensive discussions about permissible deformations during construction.

4- The installation of the anchors is to be recommended for all retaining structures in a manner of the stabilization of the structure and the lateral movements can be kept under control. Their installation is simple and they are not very expensive and they can be easily recuperated for future use on similar sheet pile walls.

5- The maximum bending moment and horizontal displacement occurring along the sheet pile wall and also the maximum ground surface settlement have been considerably reduced by increasing the pertaining parameters of the system but obviously, this leads to an increase of the maximum original anchor force. Therefore, a trade-off is required to balance the increase of original anchor force and the reduction of maximum bending moment and displacements.

6- The grouted anchor inclination has a great effect on the system's performance and an inclination up to 20° increases the performance enhancement of the system for this study.

7- It has been experienced that the extent of soil investigations and laboratory data are limited for this study. This introduced some difficulties in idealizing the soil profile as well as deriving the soil parameters. As a result, the success of the FEM analysis may be affected due to this limitation.

8- The computed displacements i.e. lateral wall deflections and settlements are usually compatible with the values reported at the literature at similar excavations.

9- The maximum horizontal displacement and maximum vertical displacement have been decreased 30% and 60% respectively in 3D modeling in comparison with 2D model.

10- The factor of safety has been increased 24% and the maximum force in anchors has not been changed significantly in 3D modeling in comparison with 2D model.

11- It seems that with the 3D modeling we obtain more realistic results that it could be verified with further investigations. These particular results may be due to the 3D modeling, which is based on a more precise representation of the structural elements, including particulars that could not be represented in the 2D case, such as vertical slabs that make the subsurface slab more rigid, and also using the full length piles in all phases.

12- The performance of the sheet pile wall will increase with the increase of the depth of the grouted anchor in terms of factor of safety and final excavated depth. Also the maximum bending moment occurring along the sheet pile wall has been considerably reduced by increasing the depth of the anchor up to -5.0 m for this case but obviously, this leads to an increase of the maximum original anchor force and maximum horizontal displacement. Therefore, a trade-off is required to balance the increase of original anchor force and horizontal displacement and the reduction of maximum bending moment and factor of safety.

13- The results illustrate that the elastic-perfectly plastic constitutive models such as the Mohr-Coulomb model are not well suited for analyzing this type of problems and more advanced models are required to obtain realistic results. Strain hardening plasticity models are in general a better choice and produce settlement troughs being more in agreement with expected behavior.

14- The HS model provided a competent result in comparison to the MC model. This was directly due to the model's incorporation of deviatoric and volumetric hardening mechanisms, stress-path dependent stiffness, soil dilatancy and the expansion or contraction of the yield surface with respect to plastic straining.

15- As it does not differentiate stress paths (e.g., loading, unloading and reloading) and account for soil creep behavior, MC model is unsuitable for analysis of excavations especially in sensitive creep soft clays. HS model can give reasonable prediction at shallow excavation depths. While, it would significantly under-estimate wall deflections at greater excavation depths because it does not account for substantial creep behavior of clays at high unloading stress levels.

16- The present study can be developed and refined and a deeper study in this specific subject would be of great interest. It would also be preferable to perform field measurements in order to determine the real behavior of the sheet pile wall. Therefore, by comparing the measured field results with the various constitutive models a better understanding of the behavior of these anchored sheet pile walls will be concluded.

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