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DESIGN OF TIED-BACK RETAINING WALLS

BY

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The undersigned certifies that he has read and recommends to the Faculty of Graduate Studies and Research for acceptance, a report entitled DESIGN OF TIED-BACK RETAINING WALLS submitted by Dennis W. Kerr in partial fulfillment of the requirements for the degree of Master of Engineering.

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TABLE OF CONTENTS

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ABSTRACT

In recent years the need for deep excavations with steep side slopes has become increasingly important. Design criteria for support of the slopes include a balance between safety-flat slopes, functional performance - unobstructed working area and minimum cost - steep slopes.

Elimination of convention internal struts and replacement with ground anchors formed in the retained soil mass is becoming a more common support system for deep urban excavations. Responsibility is heightened in urban centres where the integrity of adjacent buildings, transportation routes and services must be maintained.

This system of construction using ground anchors as support for retaining walls is referred to as a tied-back wall.

The factors affecting the performance of a tied-back wall are outlined in the following chapters.

A review of the literature on research work, case histories and current design techniques on tied-back walls has enabled the development of a suggested design procedure.

CHAPTER 1

INTRODUCTION

A variety of earth retention schemes are currently employed by designers to permit deep excavations in soils and soft rocks. In most cases these are composed of a wall and a support system.

The common walls in use are of two types:

1. Stiff - concrete walls formed by slurry trench techniques.
2. Flexible - soldier piles or steel sheet piles.

Support may be offered to the walls by:

1. Internal struts and walers.
2. Cross-lot bracing.
3. Construction of upper floors with subsequent excavation.
4. Tie-back walls.

In recent years with the development of more sophisticated construction materials and techniques tie-back walls have gained increased acceptance as a support system. Its versatility allows application in a variety of geologic conditions, hydraulic conditions, and excavation techniques.

Contractors have found the relatively simple and flexible system of ground support provides a clear unobstructed working space. In addition, flexibility is allowed in construction procedure and machinery to be used.

Designers have also found the system to be advantageous because:

1. Pre-stressing of the tie-backs allows one to control settlements by reinstating the original stress conditions.
2. Lighter wall sections may be employed since wall stresses can be controlled by the application of more anchors.
3. The wall is constructed from the top down, hence, the insitu strength of the soil may be employed, not that of a remolded compacted soil.
4. The need to underpin adjacent structures may be eliminated as a result of settlement control.
5. The tie-back stresses can be controlled.
6. Blasting may be carried out relatively close to the anchored wall without damage to either the anchor system or the sheet pile.
7. Positioning of anchors to meet local conditions is possible.
8. Tensioning of the anchors provides information on existing ground conditions behind the wall.

Disadvantages in the use of this type of support system also exist which include:

1. The need for competent workmanship to ensure no damage to adjacent underground services.
2. Containment and disposal of drilling water. This water may also weaken the contained soil.
3. Difficulties in differentiating between anchor creep and anchor failure. Replacement of high level anchors is expensive and difficult.
4. Methods of monitoring performance may cause problems. For example, jacking of anchors is difficult at high levels.
5. The vertical reaction induced on the wall by inclining the anchor presents the need for a good foundation.
6. In congested areas anchorage beneath adjacent structures may be essential. Lawsuits may follow as a result of trespassing.

Technological advances which have aided in the development of tie-back walls include high early strength cement grout, expanding agents, accelerators, resin grout, granular anchorage systems and pressure grouting to increase the anchorage capacity. In addition, better quality steel to reduce anchor rod creep and corrosion and utilization of cable strands instead of rods to increase capacity has aided in the development of high quality, high capacity, versatile anchors (12, 25, 83). On Figure 1 is shown a typical configuration of a tied-back retaining wall.

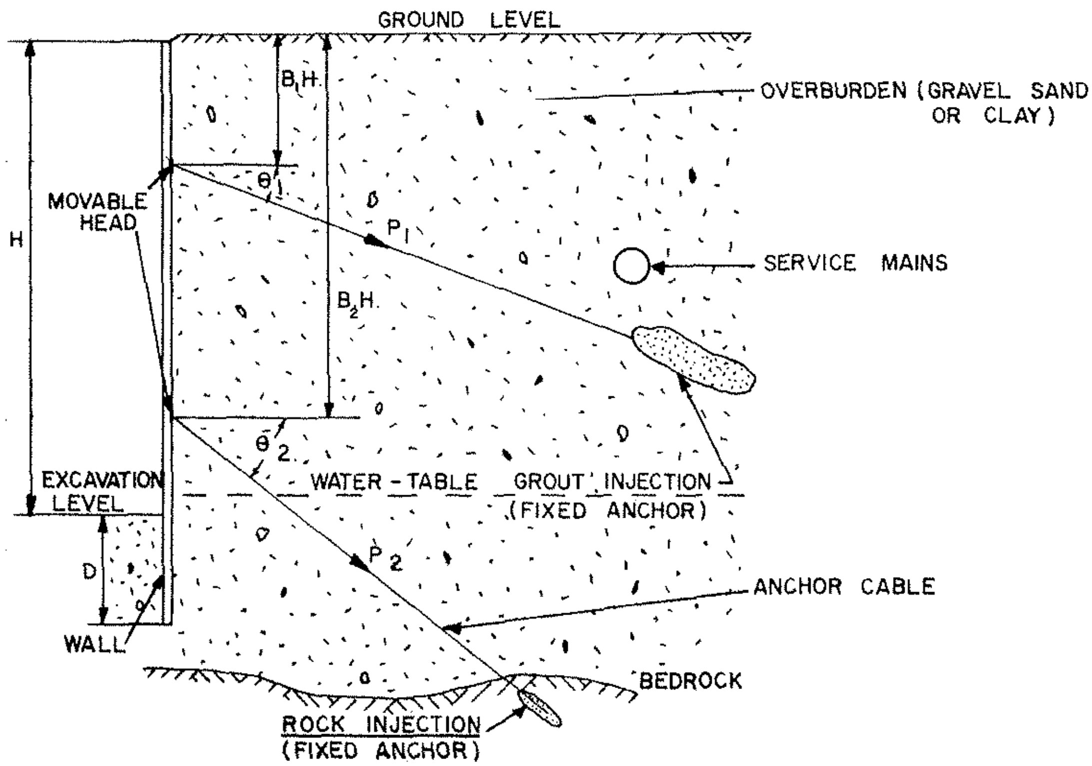


Figure 1: Principle of tie-back wall. This method of wall support eliminates struts which in turn brings large economic and construction advantages.

From its first use as a pre-stressed rock anchor at the Cheurfas Dam in Algeria, anchorage systems have developed to include support to prevent flotation of structures while unloaded such as dams, cofferdams and dry docks, tunnel support, support of excavations in restricted areas, underground storage tank design, reduce overturning moments in tall buildings, anchorage for thrust blocks, pre-loading to reduce settlements and anchorage for pile and plate bearing tests (25).

Extensions of rock anchorage systems to soil anchorage systems was a natural process. Morrison and Coates (57) demonstrated that rock mechanics principles are merely an extension of soil mechanics principles. In discussing factors which affect rock slope stability Muller (58) suggests that material properties, geometry, ground water effects, internal structure and effect of adjacent structures are the predominate factors. To this list Lambe (44) adds time effects, support system used, construction technique and transient load effects as further factors affecting braced excavation stability in soils. While the mass structure is very important to rock slope stability (59) it is of lesser importance in soils unless one is dealing with fissured clays (63). Stiffness of the unstable mass, however, is important to the stability of both materials (25, 54, 63).

The normal sequence of construction of a multi-anchored tied-back wall in soil or soft rocks is initial placement of the wall in the ground to the desired depth followed by excavation to the first anchor level. This row of anchors is installed, pre-stressed and the excavation process is repeated.

The mechanics of wall performance, if one neglects the effects of wall translation and anchor wire interaction, has been explained by Hanna and Matallana (32). As excavation proceeds, earth pressure acts on the wall. This results in a shear stress being set-up at the soil-wall boundary along with a stress change in the surrounding soil due to stress relief. Insertion and stressing of anchors modifies the earth pressure acting on the wall. Inclined anchors will magnify the shear stresses set-up at the soil-wall surface. Hanna (29) suggests that these shear stresses migrate down the wall during the process of excavation and, hence, a large toe load is developed in a manner similar to that of friction piles. The combination of continued excavation and anchor stresses at lower levels adds to this base load. Excessive settlements and bearing capacity failure may follow.

The factors which affect wall performance include excavation depth relative to wall embedment, wall-base forces, wall-ground movements, anchor flexibility, design assumptions, soil characteristics, stress strain history of the soil, construction technique and workmanship.

The design of an excavation support system is unique in that construction of the system must be from the ground surface down as excavation progresses, an auxilliary wall support system is required, the wall is usually made up of a number of interconnecting prefabricated structural members and although one knows that due to the flexible nature of the wall, displacements will occur, the actual magnitudes are difficult to predict (63).

The use of a tie-back system is particularly advantageous in this regard since case histories (64) have shown that reduction of wall movements and corresponding ground loss can be controlled by limiting vertical strut spacing and prohibiting excavation below the support level until all supports are installed and pre-stressed. Since the tie-backs are installed at the excavation level no overexcavation occurs.

However, it must be noted that pre-stressing of the wall by an anchorage system to a stress level equivalent to the 'at rest' stage will not provide a 'no movement' condition because the release of vertical pressures during excavation has not been balanced and this change in loading along with instantaneous movements before pre-stressing produce horizontal movements which are not completely reversible (31, 32, 56, 63).

As implied above, the movement of soil adjacent to a deep excavation is responsible for ground loss at the surface. To preserve the status quo of existing structures and services an estimate of the magnitude of these settlements and their pattern of distribution is required.

Of the factors mentioned above which affect wall performance, workmanship is the most critical. Hence, theoretical solutions to these problems while being a valuable tool to aid one's engineering judgement, are not reliable since a mathematical simulation of workmanship is not possible. On the other hand, improved workmanship will not improve the performance if a theoretical solution indicates large movements, bottom heave or base failure. Alteration of the complete design is then required (63, 64).

Observations of wall movement have shown that the volume of settlement surrounding the structure is approximately equal to the volume of lost ground associated with inward movements of the vertical walls (63). Hence, surface settlement control implies control of lateral wall movements and bottom heave. While these movements cannot be eliminated entirely a judicious choice of anchor inclinination, level, spacing and wall flexibility will keep them to a minimum (31, 67). Peck (63) suggests practical innovative construction techniques which, while being more expensive to use, may result in better performance.

One may obtain a feel for the expected movements if a high quality soils investigation is performed to determine the soil profile and its variation along the proposed excavation as well as ground water conditions.

In cohesionless sands, negligible movements may be expected if adequate pre-stressing is performed (71). Relatively small movements may be expected in cohesive granular soils (79). In soft to medium clays large movements may be expected (22, 25, 59, 72) especially if over-excavation is permitted and the anchors are not pre-stressed. Peck (62) and Ward (88) suggest that the maximum overexcavation depth should not be greater than $\frac{2q_u}{\gamma}$. In stiff clays unless high lateral stresses exist (21) small movements may be expected since reduction of vertical pressure is also important in causing settlements as strength and stiffness increase. In fact, ground rise due to elastic unloading has been observed (63).

CHAPTER 2

ANALYTIC AND LABORATORY STUDIES OF MODEL WALLS

The use of an anchorage system to support rock masses has been common practice for a number of years.

The behaviour of the anchor under stress may be predicted with some degree of confidence (32). The behaviour of a wall under stress may also be predicted. However, when the wall and the anchorage unit are connected to perform as a unit the performance of this system is not as easily determined. The mechanics of the interaction between the wall and the anchor under stress has neither been well defined nor documented.

In an effort to remedy this obvious gap in soil mechanics theory, a large amount of research has been conducted in recent years by the use of both finite element techniques and field or laboratory testing. The factors which were found to affect the performance of the wall and design criteria are outlined below:

1. The shape of the earth pressure distribution envelope is governed by the wall flexibility and kinematics. If the wall is rigid and fixed at its base an essentially hydrostatic triangular distribution results. For top fixity a parabolic or trapazoidal distribution occurs. Rotation about the top support level for excavation depths exceeding

one half the wall height is usual and, hence, within the range of engineering interest a trapazoidal distribution occurs.

The amount of strain required to mobilize active earth pressures is a function of the stress strain history of the soil, the geometry of the wall and history of wall movements. Stress redistribution occurs as a result of wall movements. Reverse wall movements result in passive pressure mobilization at the top of the wall while the outward wall movements at the wall base mobilize partial active pressures.

The mobilized pressures are time dependent, a function of construction techniques and workmanship and anchor inclination. However, in design for minimum wall movements the suggested empirical lateral earth pressure coefficient is $\frac{Ko+Ka}{2}$, which is insensitive to anchor inclination, stiffness or wall stiffness. The effect of adjacent foundations is to intensify the magnitude, not the distribution, of the earth pressure envelope in a manner which conforms qualitatively with the predictions of Coulomb. The actual increase in total pressure on the wall is governed by the proximity of the foundation to the wall (5, 7, 9, 10, 30, 32, 67, 72).

2. The design anchor loads were mobilized for the trapazoid shaped earth pressure distribution whether the anchors were prestressed or not. However, load loss in inclined anchors cannot be prevented (30, 32, 67).

3. Wall movements were sensitive to design assumptions. Minimum movements were attained when the $\frac{Ko+Ka}{2}$, coefficient was used. Use of Ko alone resulted in large passive pressures at the top of the wall for horizontal anchor installation. For inclined anchors a larger ground loss occurred if Ko were used.

   Initial wall movements were basically horizontal followed by settlement of the wall. Settlement of walls with inclined anchors was an order of magnitude higher than those with horizontal anchors and increased with depth of excavation for both support systems.

   The location of the maximum movements is a function of wall rigidity.

(a) Rigid wall - at the base.

(b) Flexible wall - in the spans between anchor levels.

Note that for stiff walls smaller movements were experienced above and at the excavation line but greater deflections were observed immediately below excavation line than for flexible walls because flexible walls mobilize greater passive pressures at the excavation line. Wall movements increase rapidly with increase of excavation. Anchors will control the movements to a depth of .2H below the anchor level (10, 19, 30, 32).

4. Ground loss is insignificant if the depth of the excavation is less than one half the wall height. For greater depths of excavation the walls supported by inclined anchors experienced ground loss two orders of magnitude greater than horizontally supported walls. The zone of influence behind the wall extended to a distance 2/3 the wall height and was insensitive to anchor inclination although measured movements may occur up to a distance of twice the wall height from the excavation (10, 19, 30, 32, 67).

5. The magnitude and distribution of the wall base force is unknown (32).
6. The mobilized wall-soil friction which is proportional to wall flexibility is considerably less than Ø peak but the actual value is unknown (30, 32, 78).
7. Wall movements are a function of the soil load-deformation response and the interaction with the anchors. The tools for analysis are not known although Hanna (28) suggests some simple approximations (19, 32, 67).
8. The performance of tied-back walls is generally superior to braced walls due to design and construction techniques (10).
9. Excavation beyond the support level before support installation may result in deflections twice as high as those which would occur if overexcavation were not allowed (10, 19).
10. Pre-stressing limits movements. The effect on the earth pressure distribution is a function of wall stiffness. Stress concentrations occur at the anchor points in flexible walls and is generally uniform for stiff walls (10, 19, 32).
11. Wall deformations and settlements are inversely proportional to (a) wall rigidity

(increase wall rigidity by a factor of 32 reduces movements by a factor of 2) (10, 19); and (b) tie-back stiffness (increase stiffness by a factor of 10 and reduce movements by a factor of 2) (10, 19, 30).

12. Wall anchors significantly reduce the maximum bending moments in the wall. The practice of designers to use design bending moments based on earth pressure envelopes irrespective of the various parameters involved is a valid assumption on the safe side due to stress redistribution. The effect of highly stressed widely spaced anchors is small and may be ignored (10, 30).

13. Wall bending moments are inversely proportional to wall flexibility.

    Wall bending moments are inversely proportional to anchor inclination although the effect is small on stiff walls or when the excavation approaches the full wall height on flexible walls, due to movements (10, 30).

CHAPTER 3

CASE HISTORIES OF TIED-BACK-WALL USE

Anchors have been used for a number of years as a support mechanism. However, older case histories are lacking in both design details and performance data. Because of increasing importance of tie-back supported walls in the solution of temporary and permanent excavations and the fact that walls are relatively flexible, designers have been in considerable doubt as to the approach to follow. In addition, documented experience on which to base designs or assess field performance is minimal.

Some progress in this direction has been accomplished in recent years since the need to assess the performance of the support system as well as gain more confidence in the use of current design techniques and construction practice was recognized.

It is quite evident from analysis of case histories that design practice and construction technique is, if one accounts for local experience and designers preference, quite similar. A summary from case histories of the salient features of current design practice and performance of tied-back retaining walls follows.

1. Anchors are typically installed in inclined pre-drilled holes ranging in diameter from 3 inches to 24 inches. Anchor inclination on the order of 20° is usual with steeper

inclinations in the top row to clear adjacent services.

Pre-stressing of each anchor to 120% of the design load is common to allow for creep and relaxation. In addition, a selected number of anchors are stressed to 150% of the design load to confirm initial design assumptions and as a subsequent load test (3, 11, 18, 47, 50, 61, 62, 80, 89).

2. The required anchorage length which may be initially determined by analytical techniques is always confirmed by load tests. Fifteen feet is accepted as the minimum length and is terminated a minimum of 5 feet beyond the assumed failure surface. A variety of anchorage mechanisms are available (3, 11, 18, 25, 47, 59, 62, 80, 89).

3. The pre-stressed load on the anchor is usually constant with time. However, some relaxation (up to 75% of design load) may occur if the anchors are steeply inclined. These losses are a function of anchor length and soil type with performance being significantly better in stiff clays (11, 34, 50).

4. The wall is usually embedded 5 to 15 feet below the maximum excavation depth by driving, pre-drilling or slurry trench techniques. If

soldier piles are placed in pre-drilled holes they are backfilled with a lean sand cement mixture (3, 11, 50, 61, 80, 89).

5. For soldier pile installations covering of the exposed surface by gunnite or asphalt mix is common practice (24, 80).

6. Wall deflections are a function of wall stiffness, other things being equal (45).

7. The magnitude of anchor load dictates the type of cable and grouting techniques to be employed. Multiple cable strands may be used to carry larger loads than an individual rod. High early strength concrete, expanding agents, pressure grouting, accelerators and resin grouts are optional materials to be used in developing anchorage. The curing period varies with the above options between 3 and 7 days (18, 50, 61, 69).

8. The choice of the assumed failure surface is quite variable. It may originate at the toe of the wall, at the calculated minimum driving depth or at the base of the excavation. The inclination is governed by this choice - the closer the base of the excavation, the steeper the inclination of

the assumed failure surface as well as being influenced by Rankine's 45+Ø/2 failure criteria. Figure 2 shows some of the common assumed failure planes (3, 11, 18, 47, 50, 80, 89).

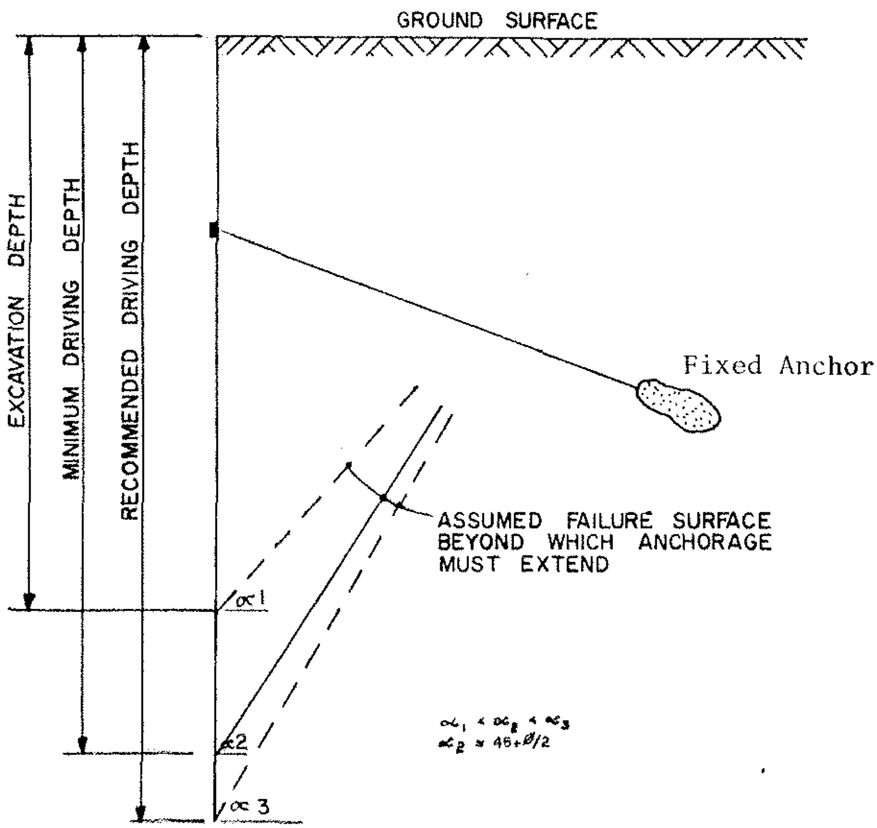


Figure 2: Assumed failure surfaces used in practice. The choice of origin and inclination of failure surface is governed by individual designer preference.

9. The design earth pressure distributions were usually trapazoidal while some rectangular distributions were employed. It is interesting to note that lateral earth pressure coefficients used were quite variable

and the variance was not necessarily dictated by soil properties (11, 47, 50, 67).

10. The factor of safety used for all design components was between 1.5 and 2 if a temporary scheme. Permanent support systems typically employed higher factors of safety with some means of corrosion resistance employed (11, 18, 47).

11. The use of finite element techniques is becoming more popular as a design tool especially in complex deposits and overconsolidated clays where experience is limited. Wall performance is usually better than predicted by this analysis since the anchor pre-stressing action reinforcing local shear zones results in a stiffer system (11).

CHAPTER 4

MAJOR CAUSES OF TIED-BACK WALL FAILURE

Research and development of tied-back retaining wall systems has not kept pace with increased construction requirements of recent years. This results in a neglect of soil mechanics principles, hence, excessive settlements or even complete failure may occur.

Planning of and design of anchored bulkheads to support soil masses requires more than a knowledge of earth pressure mechanics and design of structural systems. Consideration of the soil and support system as a structural entity is essential. The load and deformation characteristics of the soil must be evaluated as accurately as possible using existing theory. However, since mobilized earth pressures are a function of construction technique, which is further governed by workmanship, a rigorous application of the theory would not be useful. It merely serves to aid one's engineering judgement in developing a support system which will allow for construction blunders and unknown loadings. This will include selection of the correct geometry for anchorage, stressing members to realistic levels and limiting movements to acceptable levels (82).

The basic failure mechanisms (27, 65) shown in Figure 3 are:

1. Bearing capacity failure may occur if weak cohesive material exists at depth

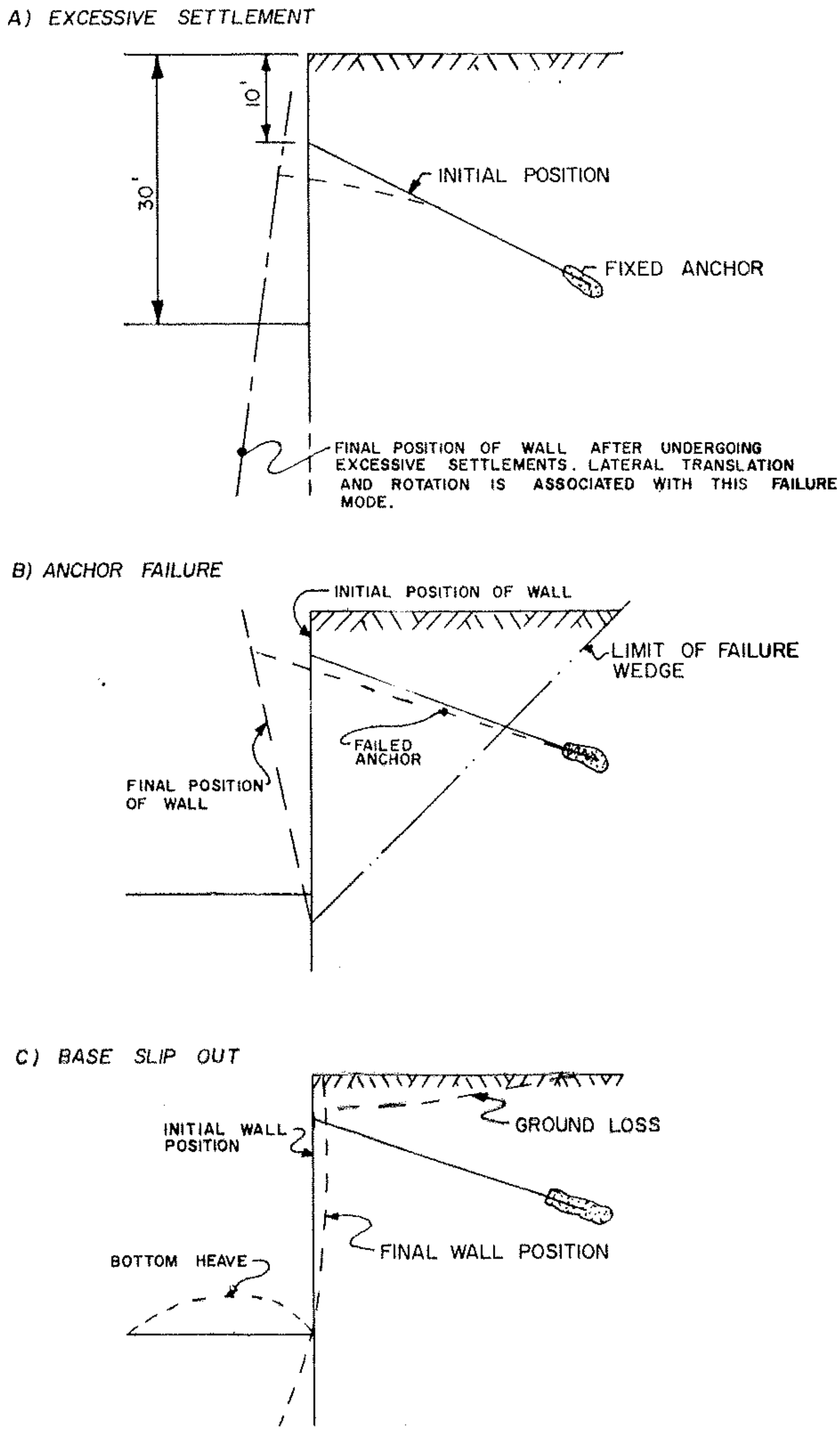


FIGURE 3. BASIC FAILURE MECHANISMS

below the wall.

2. Anchor failures due to:

   (a) underestimation of rod forces.

   (b) overestimation of anchor resistance.

   (c) anchorage within the failure wedge.

3. Base slip out as a result of insufficient embedment of the wall which restricts passive pressure mobilization.

The causes of failure may be summarized as:

1. excessive lateral earth pressure.
2. inadequate anchor support and pile embedment.
3. inadequate consideration of or allowance for deflections.
4. poor design details.
5. corrosion of components.
6. lack of consideration of construction operations and related structures.

It is evident from the basic causes of failure that inadequate earth pressure theories are not at fault. It is the neglect of backfill loads, construction operations, deflections, corrosion and design and construction details which are usually at fault.

Sowers and Sowers (82) report several case histories to support this statement.

CHAPTER 5

DESIGN MECHANICS

A. Introduction

The performance of a tied-back wall in resisting lateral pressures and minimizing displacements is highly dependent on the excavation process. The fundamental concept of the design process is the minimization of lateral movements by pre-stressing the anchors prior to full load application (27, 30, 32).

Unfortunately, a 'feel' for the adequacy of the design process could not be attained from older case histories (18, 20, 24) since details of wall stress, anchor stresses and displacements were omitted. Recent studies tend to be more thorough (3, 11, 47, 50, 61, 62, 80, 89).

In practice a wide variety of soil conditions exist. This coupled with local construction practice makes it important that a flexible design scheme be available and for it to be applicable. A review of the case histories demonstrates the variation of construction techniques, materials, excavation and anchor geometry, assumed failure surface and magnitude of earth pressures which are in common use.

The basic requirements of any design scheme are to:

1. Load members to economic levels - wall

member is sized by assuming an earth pressure envelope while the anchor load is based on a factor of the pullout capacity of an individual anchor.

2. Limit movements to tolerable levels. Instantaneous elestic deformations which occur upon unloading prior to anchor prestressing cannot be controlled. Tools to evaluate the magnitude and distribution of the movements are poor although Tschebotarioff was quite successful in a specific case (82). Finite element techniques provide reasonable estimates of wall movement (11, 56).

   Control of movements which result from the construction process may be restricted to negligible amounts.

3. Maintain overall stability. It has been common practice of designers to consider the wall member, ground anchor and overall system stability as separate entities. Clearly it is the interaction of the various components which dictates the behaviour of the system. An appreciation of this may be obtained by analysis of the construction sequence which results in progressive load

changes and subsequent wall movements. In addition, the stress relief results in changes in mobilized shear resistance of the soil and compressibility characteristics due to pore-water redistribution. Hence, time dependent soil properties results in time dependent load changes (9, 31, 63, 64).

Because the interaction mechanics of the various members is not well understood at the present time, a component design scheme coupled with engineering judgement is required.

The performance of the wall support system will reflect the designers ability within the framework of engineering judgement to incorporate:

1. The response of the soil to stress changes and deformations as a result of excavation.
2. The mobilized earth pressures.
3. The interaction between the anchor, wall and retained ground.
4. A reasonable estimate of the failure surface into the design process.

Lambe (44) points out the need to look more closely at stress paths as a useful design tool since an understanding of the design assumptions may be more easily understood. Larrsen et al (47) suggests that due to anchor pre-stress of the soil mass the soil strength conditions are changed and the actual factor of safety is unknown. In fact it is likely that the pre-stressed soil will behave as a gravity monolithic retaining structure of large dimensions for all practical purposes.

In summary current design practice is adequate as case histories indicate but refinement of the design technique, possibly resulting in more economical design, is required. The most important design variables include anchor geometry, pile embedment, excavation geometry and earth pressures all of which are assessed separately. A basic assumption in the design process is to base the distance between adjacent anchor levels on wall stresses induced by earth pressures.

However, one must be cautious when predicting these loads due to one's lack of knowledge of soil properties, boundary conditions, construction details and their variation with time.

The coupling of lateral wall movements and the development of the frictional force on the wall results in an inclined wall force at the pile base of unknown magnitude and direction (32).

The anchor loads and subsequent wall movements are very sensitive to design assumptions. In addition, the reduction of upper anchor loads during installation of lower anchors is not necessarily the result of creep but a measure of the soil-wall interaction (32) the mechanics of which are not well understood.

The bearing capacity of the wall is often overlooked. Wall movements which are both instantaneous and time dependent are a function of soil type, stress history, excavation geometry and design assumption. If movement cannot be tolerated then the Ko pressure coefficient may be used. But as stated in the section on laboratory testing, it may be uneconomic and unwise to use this coefficient as some movement will occur anyway. The average of the active and at rest coefficient is suggested for best performance.

While local stress concentrations are often ignored, allowance must be made for corrosion, surcharge, ice pressures, water pressures, machinery vibrations, earthquakes and time of application. If the structure is permanent a higher factor of safety is required and effective stress parameters should be employed (9, 23, 27, 53, 79).

Broms (6) suggests that good rules of thumb to follow are:

1. Anchorage must be behind the assumed failure surface.
2. Anchorage must be at least 25 feet below ground surface.

Lambe (44) and Peck (65) stress the need for one to look at the stability number $\frac{(\gamma H)}{c_u}$ in the design of the support system especially in clays. Redlinger and Dodson (70) present a graphical solution based on the work of John (40) for use in heavily jointed rock masses.

Breth and Wonaschuk (5) were not satisfied with the present practice of considering foundation weights as additional earth pressures and by superposition adding to existing earth pressures. A technique to calculate earth pressures imposed by adjacent foundations was developed. Bukovansky (8) developed techniques to determine design parameters in soft rock. Okusa (60) has shown that the maximum pressure exerted on the wall by a bedded soft rock mass will not be greater than the active earth pressure of a homogeneous soil.

In addition, developments in finite element application have been outlined previously.

B. <u>The Wall</u>

The actual wall to be used at a particular site is governed by local conditions and practice. Conventional walls are of two basic types (25, 31).

(a) Flexible - vertical sheet piling, interlocking steel sheet piles, soldier piles with lagging.

(b) Stiff - diaphragm walls, bored piles.

The stiff walls are usually installed by slurry trench techniques while flexible walls are usually driven.

Soldier piles may be placed in pre-bored holes and back-filled with a weak sand-cement mix if driving is too difficult (18, 25, 31, 80). Lagging is not always essential in soldier pile installations. Spraying of the retained soil surface with an asphalt mix or gunite (25, 80) may be sufficient to maintain the integrity of the soil by prevention of surface drying.

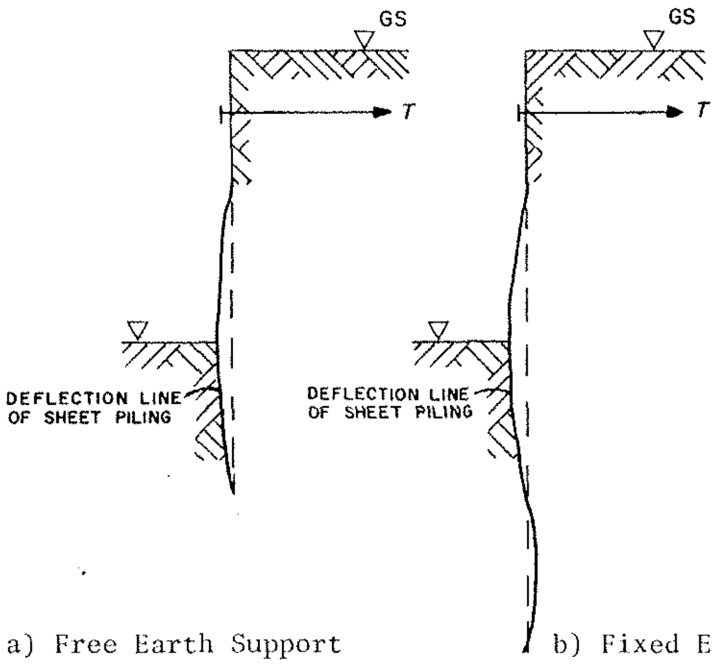
It has been shown that wall deflections are inversely proportional to wall stiffness. However, the effect is very small as the behaviour of the combined mass of soil and wall is primarily influenced by the stress deformation characteristics of the soil (63, 64).

In multi-anchored walls embedment from 5 feet to 1/3 the wall height has been employed to improve system performance (3, 10, 11, 16, 18, 20, 24, 47, 69, 71, 89). Analytical techniques to determine the required embedment are very difficult for this case. In singly anchored walls the depth of embedment is easily calculated.

The two accepted analytical procedures to be used in single anchor wall design are the free-earth support method and the fixed-earth method. Figure 4 shows the deflection characteristics of the wall for both methods of analysis.

The free-earth support method or the method of minimum penetration resistance is the oldest and most conservative design procedure (87) and is accepted in practice because of its simplicity and success (16, 31, 84).

Figure 4: Two methods for anchored sheet-piling analysis



a) Free Earth Support b) Fixed Earth Support

For this approach the wall is assumed to be inflexible. Thesefore, no pivot point exists below the excavation depth (41) and passive pressures are mobilized on the excavation side of the pile only (90). Due to the small embedment, no fixity of the pile occurs.

The depth of embedment will be that required to develop sufficient passive pressures to equilibrate the active pressure moments about the anchor point. Classical Rankine Theory is used to calculate these pressures. Jumikis (41) and Tschebotarioff (87) provide equations to be used in calculating both the depth of embedment and moments and shears imposed on the wall. This driving depth is based on a factor of safety of unity to prevent lateral toe yield. According to Danish Rules (16) for a factor of safety of 1.7 and 2 one must drive the pile to 1.4 and 1.7 times the calculated depth respectively.

It should be noted that overdriving the pile invalidates the conditions upon which the theory is based (that the soil below the dredge line has reached its limiting shear strength throughout the depth of embedment) (87). Therefore, some fixity will occur. Tschebotarioff suggests that for clays the factor of the calculated depth by which the pile is overdriven is the actual factor of safety since the strains required to generate active pressures are inhibited.

The sizing of the wall is based on bending momments and shear forces imposed by the lateral earth pressures which are dependent on wall type and construction technique for any given soil conditions. The maximum mobilized bending moments are a function of the pile flexibility and soil relative density (74). The bending moments can be kept to a miminum by a judicious choice of anchor level in multiple anchor walls.

For single anchor walls analytical techniques for evaluating the maximum bending movements have been developed (41, 74, 87) which include the reduction of the free-earth support bending moments at the excavation line which are overly conservative. Stroyer (84) suggests that a moment reduction factor = $\frac{2K_0}{1+K_0^2}$ may be employed. Rowe (73, 74, 75) has developed design charts for use in calculating the bending moments as a result of the reduction which occurs. In the discussions to Rowe's original paper (73), Terzaghi, Tschebotarioff and others caution the designer when using this technique which oversimplifies the situation. Subsequent work by Hanna and Matallana (32) and Casagrande (9) support this conclusion.

The reason for the moment reduction is not clear. Hanna and Littlejohn (31) and Tschebotarioff (87) suggest that it occurs as a result of redistribution of earth pressures and subsequent soil arching around the anchor level. Rowe (73) feels it is a result of small passive pressure increases since the moment is proportional to the cube of the span. Therefore, small passive pressure changes result in large moment changes.

For complex soil conditions, over-consolidated soils and multiple anchorage systems where experience is limited and case histories scarce, finite element techniques have been very useful (11, 77). Usually performance is better than expected as a result of a stiffening of the retained soil mass upon pre-stressing.

The fixed-earth support method is based on complete fixity of the toe to the embedment depth (31, 41, 87) and therefore must be driven to greater depths than for the free-earth support design (73, 85, 87). The pile is assumed flexible (41) and large passive pressures are mobilized on both sides of the sheet pile (90).

A complete set of equations for solution of this design approach are presented by Jumikis (41).

Because of the time consuming nature and the difficulty involved, this approach is not often used in practice. Blum (25, 41) has simplified the procedure somewhat in developing the 'equivalent beam' approach. Tschebotarioff (87) further modified this approach in developing the 'hinge at the dredge-line' technique but suggests that it is applicable only in sands. Tschebotarioff advises one to use the free-earth support method in clays (87).

It should be noted that much smaller bending moments are experienced using this design procedure and they do not conform to Danish practice (73, 85, 87).

C. The Anchor

The wall, whether stiff or flexible, derives its support from tied-back anchors whose function it is to restrict wall and ground movements to tolerable levels. To provide support the anchorage zone must be completely outside the assumed failure wedge and, in addition, equilibrate the system from a stability criteria (13, 25).

As mentioned previously the virgin stress state cannot be restored by pre-stressing but movements and fracture generation can be controlled (10, 42).

A variety of anchor types are currently used and include the tamanchor, underream anchor, SIF-TM anchor, Bauer anchor and the buttonhead anchor (6, 18, 25, 47, 61, 68, 69, 76).

Anchor holes are advanced by drilling - preferably dry drilling as wet drilling reduces anchorage capacity, especially in clay, and containment of circulation water is always a problem. The hole diameter will vary between 3 inches to 24 inches depending upon equipment and ground conditions. Coates and Sage (10) suggest that to facilitate anchor grouting the minimum hole diameter should be at least 1 inch greater than the anchor. Casing of the hole is often required in granular deposits. Hanna (27) outlines a general approach which may be followed when installing anchors.

The spacing, inclination and anchor length are governed by ground conditions, excavation geometry and design working load. Excavation depth, wall flexibility, allowable stresses and anticipated earth pressures dictate anchor spacing. The top anchor is usually 1/3 the depth of the excavation from the top of the wall (20, 25).

The usual range of anchor inclination is 20° to 45° with some as high as 65° (6, 20, 27). The steeper inclinations are used if economic anchorage in rock may be obtained or if it is necessary to avoid adjacent structures. The choice of inclination is governed by a desire

1. to anchor in suitable material

2. have sufficient cover over the anchor,
3. not to have interference in the zone of influence of adjacent anchorages,
4. avoid underground services and,
5. not impose too large a vertical force on the piling.

Hence, the top row of anchors is usually inclined greater than succeeding rows. Swedish practice (6) dictates that the distance between anchorage zones for adjacent anchorage levels shall not be less than 2.5 meters.

Poor performance of a retained wall is often the result of large anchor inclinations (6). Plant (67) has shown that significant advantages can be gained if lower anchor levels are less steeply inclined than upper levels. Further, if the wall rests on a rigid stratum it may be advantageous to incline the anchors slightly upwards.

The length of the anchor to be used is usually based on an assumed pre-existing failure surface which extends from the wall base inclined at an angle of $45+\phi/2$ degrees to the horizontal as shown in Figure 5 (18, 25, 31, 80, 89). This length is confirmed adequate by overall stability calculations. The anchor usually extends a minimum of 5 feet beyond this line to ensure it is founded on material outside the failure zone.

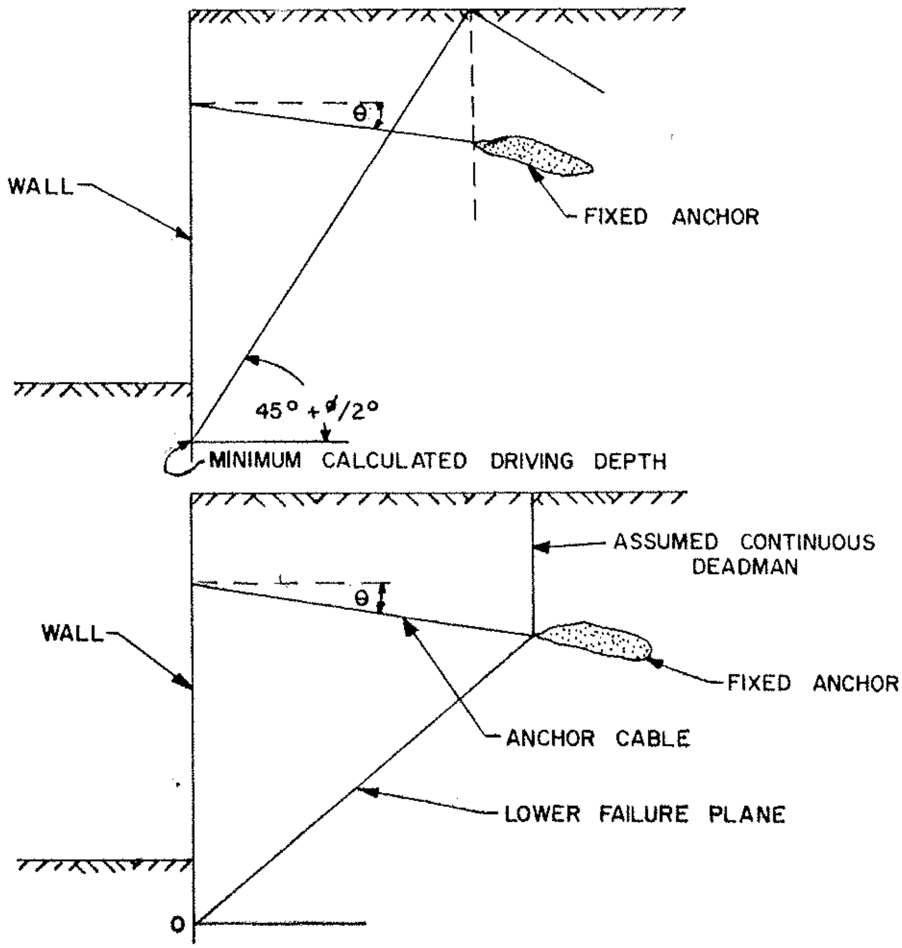


Figure 5: Method of determining anchor length. This method assumes that failure of the wall would take place along a failure plane.

The strength of the anchor is developed by grouting techniques or an apparatus at the base of the anchor which expands into the surrounding soil or rock.

Grouting techniques such as high pressure grouting, expanding agents and accelerators are used to increase the capacity of the anchor. The use of resin grout (25, 66) is becoming a popular alternative to the traditional cement grout as a means to develop anchorage. Stagg (83) recently demonstrated that introduction of granular material into the hole to provide anchorage is a practical alternative to grout. Use of high grade steel will aid in minimizing cable creep and reduce corrosion (20).

The anchorage length is dictated by the ultimate capacity of the anchors. In granular materials, the capacity is a function of grain size, grain size distribution, grout composition, injection pressure and geometric configuration of the hole. In cohesive materials the adhesion of the soil to the anchor is the governing factor (31, 32).

While the anchorage capacity can be predicted with some degree of confidence pull out tests should always be conducted. Because of this more sophisticated analytical procedures for determining capacity are not required. Culver and Jorstead (14) suggest that an anchor is assumed to perform satisfactorily if a minimum of relaxation occurs.

Anchor relaxation is the result of elastic deformation of the free length of the rod, elastic deformation of the retained length of rod in the grout, soil deformation on loading and rod grout slip.

Failure in soft rock is usually a severe localized crushing of the rock in the immediate vicinity of the anchor with anchor relaxation or bleed-off being the result. Large inelastic tensile strains at the base of the anchor indicates large single fractures may propogate out from the anchor and affect the strength of the entire rock mass (14). In addition, a catastrophic failure will probably occur along a zone of weakness whether it is parallel or perpendicular to the direction of anchor pull.

Analytical techniques are available to calculate stresses around an anchor (14, 43), anchor load relaxation with time (15), optimum anchor length (39), relationships between anchor spacing, length and design loads (46) and how to treat dynamic loads (79).

The actual details of anchor proof testing vary with the designer and local conditions. The test procedures are dictated by homogeneity of soil conditions, tie-back spacing, tie-back loads, consequence of failure and experience in the area. The usual procedure is to stress each anchor to 115% of the design load. A minimum of 5% of the anchors are pre-stressed to 150% or even 200% of the design load before relaxation to the design load to ensure adequate performance.

In addition it is desirable to check randomly 5% of the anchors two weeks after installation to ensure load loss does not occur (22, 47).

It may be pointed out that adequate anchor performance does not imply accurate prediction of mobilized earth pressures (11, 64). It merely indicates that the anchors have sustained the applied load with no distress and the lateral earth pressures were not greater than the prestress load. In terms of movements pre-stressing to these design loads greatly inhibits movements (47, 71). In fact, Rizzo et al (71) reports on the reduction of wall movements by an order of magnitude by pre-stressing the anchors to 110% of design load rather than 50%.

It may be beneficial to alter current practice to provide anchorage along the complete length of the cable. This not only improves corrosion resistance but provides additional strength by increasing the stiffness of the assumed failure mass and, hence, enhance its ability to resist movements. In addition, the anchors should not be tensioned to loads higher than those required to support the structure since this will produce high tensile strains at the anchor and possible fracture (14) as well as plastic zones at the anchor wall interface (56). Anchor wedging action may be reduced in bedded deposits by orientating the anchors so that the directional normal is parallel to jointing. This will also result in a stiffer system as the joints close up (14).

Methods for the design of earth anchors have been developed by Broms (6), Hanna (27), Jackson et al (37), and Littlejohn (49). They are all similar in that semi-empirical relationships relating soil type, shear strength, geometry, anchor socket, grouting and overburden pressures are developed and a linear shear stress distribution along the failure surface is assumed.

The capacity of the anchorage system must balance the earth pressure diagram and the vertical component of the anchor loads must not impose a wall force which would result in excessive deformation.

It has been the practice of the Committee for Waterfront structures in Germany to use the formula and charts developed by Blum to determine anchor loads and the method of Kranz to evaluate the anchor length required to equilibrate the system (25, 48, 49). The effect of a convex failure surface was ignored since the effects were small (6).

In North America, the required anchor load is taken as that required to satisfy horizontal equilibrium of the system (41, 73, 85, 87, 90). Hence, for single anchored walls the horizontal component of the anchor force must be the difference between the passive and active earth pressures.

In order to maximize anchor use Barron (1) has suggested that the optimum anchor inclination for a factor of safety of unity is given by:

$$\tan u = \tan (i+k)$$

u = angle of internal friction of retained material

i = inclination of failure surface

k = anchor inclination

This formula is based on rock mechanics principles where limit equilibrium analysis with a factor of safety equal to one and cohesion assumed zero is accepted practice (1, 25). Hence, it serves as an aid to engineering judgement for use in frictional materials but is not applicable to frictionless soils. Anchor inclination must be determined on the basis of experience and research and common practice, as stated previously, is to install them at angles between 0 - 20^{o}.

An integral part of this analysis is the assumption that all material behind a plane of maximum shear stress as calculated by Barron (1) behaves as a rigid solid mass. This is true if the slope of continuous discontinuities is greater than i. If less, than the plane of minimum excess shear strength may govern and the anchor should extend beyond the discontinuity which daylights at the base of the cut.

The design mechanics of the anchorage system are neither well understood nor supported by experimental work (3, 11, 14, 85). Theoretical studies have shown that the mobilized shear stress is a function of the relative magnitude of the modulii of deformation of the retained soil or rock and the anchor. However, empirical design assumptions are made regarding the proportions of load taken in side shear and end bearing and a correlation between ultimate resistance and individual anchor capacity, soil properties, and anchor geometry (25) is made. For the engineering range of modulus ratios the load carried in end bearing is small.

The tensile stresses induced by the anchorage system are significant within one anchor diameter of the anchor and are independent of the modulus ratio. Tensile stresses produce tensile strains which induce crack propogation and some anchor creep when the load is applied (12, 13).

The overall stability of the support system is attained through the placement of the anchors outside the failure wedge.

For rock slope engineering this failure surface is given by $\cot a = \cot i + \left(\frac{u \cot i - 1}{\sin 2i - u \cos 2i}\right)$

for a factor of safety of unity.

a = inclination of slope

i = inclination of the failure slope

u = friction developed on the failure surface

Barron et al (1) gave a graphical solution to this equation. In soil mechanics the assumed failure surface is inclined at 45+Ø/2 to the horizontal with the origin of the slope being at the calculated maximum driving depth from free-earth support considerations (27, 32).

The ultimate anchor capacity must always be confirmed by field tests. Procedures to predict this capacity have been developed based on the Mohr-Coulomb failure criteria (25, 27).

In fine sands and cohesionless silts the anchor capacity is developed by friction and the ultimate load is given by τ = Ko σ_v tan ϕ

= mobilized anchor-soil side friction.

Ko = factor based on relative density of soil.

= .5 for low relative density soils.

= 1 otherwise.

σ_v = overburden pressure.

ϕ = frictional resistance of the retained soil.

Ko is usually taken as unity due to stress concentrations and disturbance induced by drilling and injecting the grout under pressure.

For cohesive materials $\tau = \alpha$ cu where α is a factor depending on conditions. In stiff clays the capacity is generally low due to poor adhesion and structure. In addition, if wet drilling is used c_u is greatly reduced.

Hence, use α = 1 for soft to medium clays (q_u less than $5T/M^2$) or when dry drilling.

α = .5 in stiff fissured clays (q_u greater than $5T/m^2$) or when wet drilling (28).

For long term loading effective stress parameters must be used. The factor of safety to be used in cohesionless materials and soft-medium clays is 1.5-2. For stiff clays a minimum value of 2 is recommended and allowance should be made for structure.

Techniques which increase the effective anchor diameter have been developed to improve the capacity in clays such as gravel injection and multi-underreamed fixed anchor (25, 83).

In addition to anchor-soil failure, grout-rod slip may occur. The shear stress is assumed uniformly distributed on the rod and allowable values of $\tau = 2.4\sqrt{fc'}$ less than 160 psi (smooth rod), $\tau = .1fc'$ greater than 350 psi (rough rod) are commonly used with a factor of safety of 2.

The rod itself must not be overstressed.

D. Earth Pressures

The design load acting on a retaining wall is calculated using simplified assumptions and experience. The range of earth pressure distributions which may act on the wall are a function of design assumptions based on varying degrees of strength mobilization (33).

During excavation the walls deform and translate in characteristic fashions which have no resemblence to theoretical distributions based on Rankine or Coulomb Theories. Deformations at the top of the wall are less while those at the base of the wall are greater than those required to mobilize these theoretical pressures. Hence, the active pressure is not realized and the pressure distribution is not triangular. In fact, it has been shown (32, 87) that only if the wall rotates about its base will the earth pressure distribution be triangular.

If rotation occurs about the top or, as in the case of tied-back retaining walls and braced excavations, about the uppermost support level then a parabolic pressure distribution will result (32, 87).

Various authors (6, 16, 45, 50, 63, 65, 86) have suggested trapazoidal earth pressure distribution which may be used in design of braced excavations. Figure 6 shows the commonly accepted envelopes. The diagrams are essentially the same except for differences to account for local soil conditions and workmanship.

These earth pressure diagrams are an envelope to cover maximum anchor loads which may be expected over the life of the structure and are not necessarily the earth pressure at any one construction stage. In traditional braced excavations the decision to use active or at rest pressures is governed by tolerable ground movements. Hence, if lateral movements are tolerable then the structure may be designed using active earth pressures. However, in tie-back installations sufficient support is given so as to restrict the strains necessary to develop active earth pressures (65).

An analysis of the mechanics of movement is required in order to assess the magnitude of lateral stresses which may be mobilized. The pressure distribution is not 'at rest' due to yield on excavation and is not active due to anchor restraint (32). The actual distribution is a function of soil properties and construction technique.

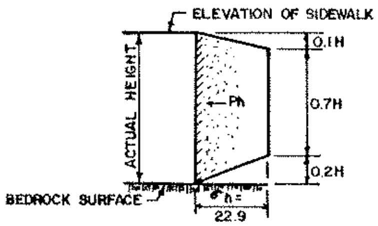
Larson et al (47) suggests that wall pressures should be based on 'partially mobilized active' since underestimation of pre-stress forces would be dangerous while

(a) BROM(6)

I Earth Pressure distribution in cohesive soils according to Rankine.

II Trapezoidal earth pressure distribution in cohesive and cohesionless soils.

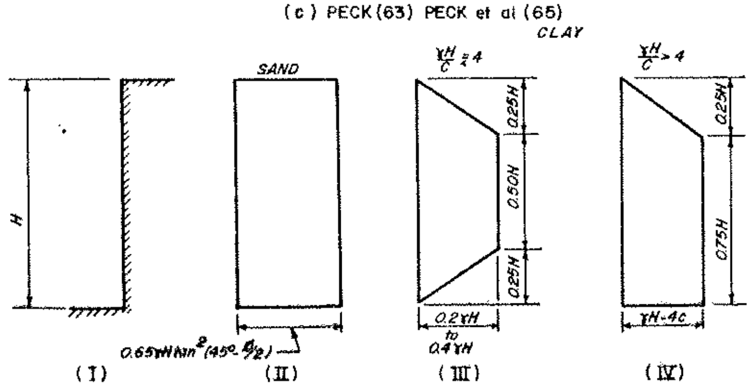
(b) LIU AND DUGAN (50)



1. PRESSURE DIAGRAM SHOWN IS ASSUMED VALID FOR PRESSURE CASE OF "PARTIALLY MOBILIZED ACTIVE".
2. USE COEFICIENT (K_a') EQUAL TO 80% OF K_a WHERE K_a IS BASED ON
   $\phi = 40°$
   $K_a = .8 \tan^2(45° - 40/2)$
   $= .8 \tan^2 25°$
   $K_a = .179$
3. USE FORMULA $\sigma_h = K_a' \gamma h$
   $= .179(125)H$
   $= 22.9H$
4. FOR H, USE ACTUAL HEIGHT FROM STREET TO ROCK, PLUS 3' TO ACCOUNT FOR SURCHARGE $P_h = \sigma_h(.85H)$.

Earth Pressure diagram used for 40-storey office tower, Boston.

(c) PECK (63) PECK et al (65)



(I) Sketch of wall of cut (II) Diagram for cuts in dry or moist sand. (III) Diagram for clays if $\gamma H/c$ is less than 4. (IV) Diagram for clays if $\gamma H/c$ is greater than 4.

(d) TERZAGHI AND PECK (First Edition) (86)

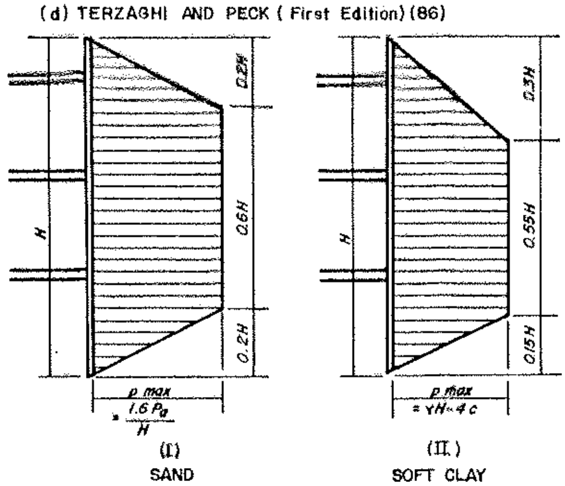


Figure 6: Apparent earth pressure diagrams for calculating loads on braced cuts.

overestimation would be uneconomical. He suggests the use of Ka' = .8 Ka. Even though performance of the structure involved was good, design based on these earth pressures is too small.

Hanna and Matallana (32) suggest the use of $\frac{Ko+Ka}{2}$ as the earth pressure coefficient in order to obtain minimum movements. They conducted a number of experiments to confirm this.

In addition to earth pressures which may act on the structure water pressures, freezing pressures, surcharge load, swelling pressures and pressures from adjacent footings must be considered (23, 31, 53, 55, 87). Prediction of the seasonal variation of these pressures while important is not possible due to the lack of data and the many variables involved. The structure of the retained mass may also be an important factor in evaluating earth pressures (63, 64). It is common practice to overdrive the wall by 20% (31, 86, 87) in order to avoid overexcavation and the resultant excessive stresses which would otherwise occur at the base of the wall. Peck (63, 64) re-analyzed the earth pressure diagrams as they are used today and introduced the stability number to account for the effects of deep soft clay deposits when choosing the lateral earth pressure coefficient in design.

Recent studies of mobilized earth pressures show that the construction sequence may be simulated quite well by finite element techniques (11, 31).

E. Factor of Safety

The factor of safety against failure must be chosen after consideration of the time of installation, whether permanent or temporary, the consequences if failure occurs and the knowledge one has of the design parameters.

The literature (6, 18, 24, 25, 30, 80) shows that the factor of safety is relatively constant for various design techniques if one accounts for local conditions. However, the actual definition of the factor of safety used is never given. Broms (6) suggests a modified version of the traditional definition:

$$F = \frac{\text{Sum of Passive Forces}}{\text{Sum of Active Forces}}$$

However, in recent years a new more consistent definition has emerged which applies a reduction factor to the strength parameters themselves. In terms of friction:

$$\tan \phi \text{ mobilized} = \frac{\tan \phi}{\text{Factor of Safety}}$$

A similar equation can be written for cohesion.

The accompanying table gives various factors of safety in current use. For a site with variable soil conditions stiff clays or soft rocks, these values are increased. Clay shales are particularly troublesome due to swelling characteristics, low residual strengths, slickensides, ubitiquous shear zones, variability, fissures and poor adhesion (55).

TABLE 1

FACTOR OF SAFETY USED IN PRACTICE

Component	Soil Conditions	Factor of Safety Permanent	Factor of Safety Temporary
Anchor	Course, Granular	2.5	1.5
	Fine to Medium Sand	2	1.5
	Stiff Clay	2.5	2
	Soft Clay	3	2
Rod Strength		1.5	
Grout - cable bond		1.5	
Grout - soil bond		1.5	
Wall stresses		1.5	
Bearing capacity		1.5	
Overall stability		2	1.5

The factor of safety for overall stability based on soil-wall, anchorage system interaction is typically 1.5 for temporary installations and 2 if permanent (26, 31).

Individual anchors should have a minimum factor of safety of 1.5 based on pullout tests to account for load decrease with time, ground variability, repetitive loading and grouping action.

Littlejohn (26) suggests that in multi-anchored systems the possibility of progressive failure exists. Hence, a minimum factor of safety of 1.6 should be used to allow for stress redistribution if anchor failure occurs.

CHAPTER 6

DESIGN EXAMPLE

A search of the literature on current design procedure for single anchored walls as listed in Chapter 5 reveals that a basic design technique is prevalent. A stability check of the system as a whole is either expressed or inferred in most cases. However, a definite procedure is not presented although Warsser (89) does suggest treating the retained soil mass as a retaining wall of large dimensions is practical.

The following design example is presented to overcome these shortcomings based on the geometry and soil properties shown in Figure 7. The active earth pressure coefficient Ka is used for simplicity.

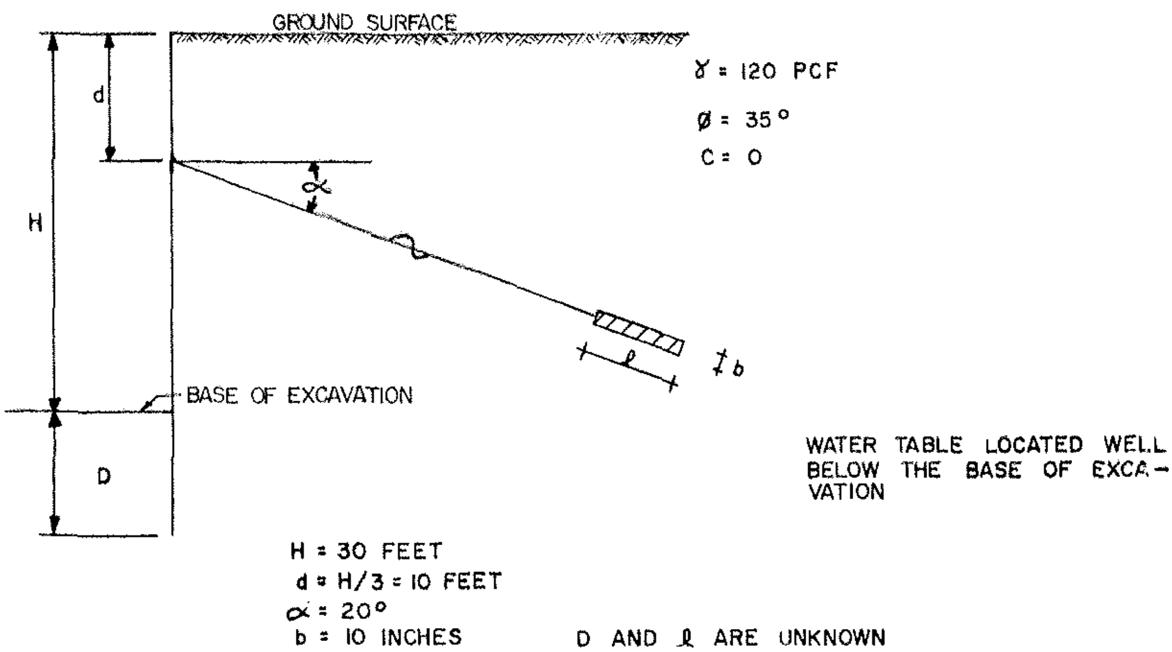


Figure 7: Geometry of design example.

1. Calculate Depth of Embedment and Anchor Load Using Free Earth Support Method.

This is a limit equilibrium method of analysis, therefore, the fully mobilized active pressures will act. However, the strains required to mobilize passive pressures are an order of magnitude higher than those required to mobilize active pressures, hence, a factor of safety of 2 will be applied to the mobilized passive pressure coefficient.

This definition of Factor of Safety results in a depth of embedment which is slightly more than the Danish rules require and slightly less than Tschebotarioff (87), Jumikis (41), or Peck et al (65) suggest. Then according to Rankine's theory the earth pressure coefficients are:

$$Ka = \tan^2 (45-\phi/2) = \tan^2 (27.5) = .271$$

$$\tan \phi mob = \frac{\tan \phi}{F} = \frac{\tan 35}{2} = .35$$

$$\phi mob = 19.5^{\circ}$$

$$Kp = \tan^2 \left(\frac{45+\phi mob}{2}\right) = 1.99$$

The required depth of embedment is that which mobilizes sufficient passive pressure moments to equilibrate the active pressure moments about the anchor point.

Using the equation of Jumikis (41) the required depth D is given by:

$$D^3 + \frac{1.5(\ (H-d)\ Kp - (2H-d)\ Ka)\ D^2}{\frac{3-N}{2}\ Kp-Ka}$$

$$- \frac{3 \quad H\ (H-d)\ Ka \quad D}{\frac{3-N}{2}\ Kp-Ka} - \frac{\frac{1}{2}\ H^2\ (2H-3d)\ Ka}{\frac{3-N}{2}\ Kp-Ka} = 0$$

For H = 30'; d = 10' $\qquad$ Kp = 1.99

Ka = .271

$$N = 1 + \frac{(F-1)}{F\ (1 + \sqrt{1-\frac{1}{F}})} = 1.293$$

F = Factor of Safety = 2

D = 13.1 ft.

To take into account the variations of strength and compressibility of the retained soil as well as allow for overexcavation it is good practice to overdrive the piles by 20% of this computed depth.

$$D_{req} = 1.2 \times 13.1 = 15.72 \text{ ft.}$$

The required anchor force is that required to satisfy horizontal force equilibrium. According to Jumikis (41)

$$T\cos 20 = \gamma\left(H\left(D + \frac{d}{2}\right) Ka - \tfrac{1}{2}D^2 (Kp - Ka)\right)$$

$$T = 10{,}345 \text{ lb.}$$

To account for the flexibility of the wall, stress concentrations, setting of the anchor and initial creep the anchors should be pre-stressed to 120% of this value

$$T = 10{,}345 \times 1.2 = \underline{12{,}415 \text{ lb}}.$$

2. System Stability

In order to obtain a measure of the interaction behaviour of the wall anchor-soil system the system stability must be analyzed. The factor of safety will be 1.5, therefore,

$$\tan \phi \text{ mobilized} = \frac{\tan 35}{1.5} = .467$$

$$\phi \text{ mob} = 25^{\circ}$$

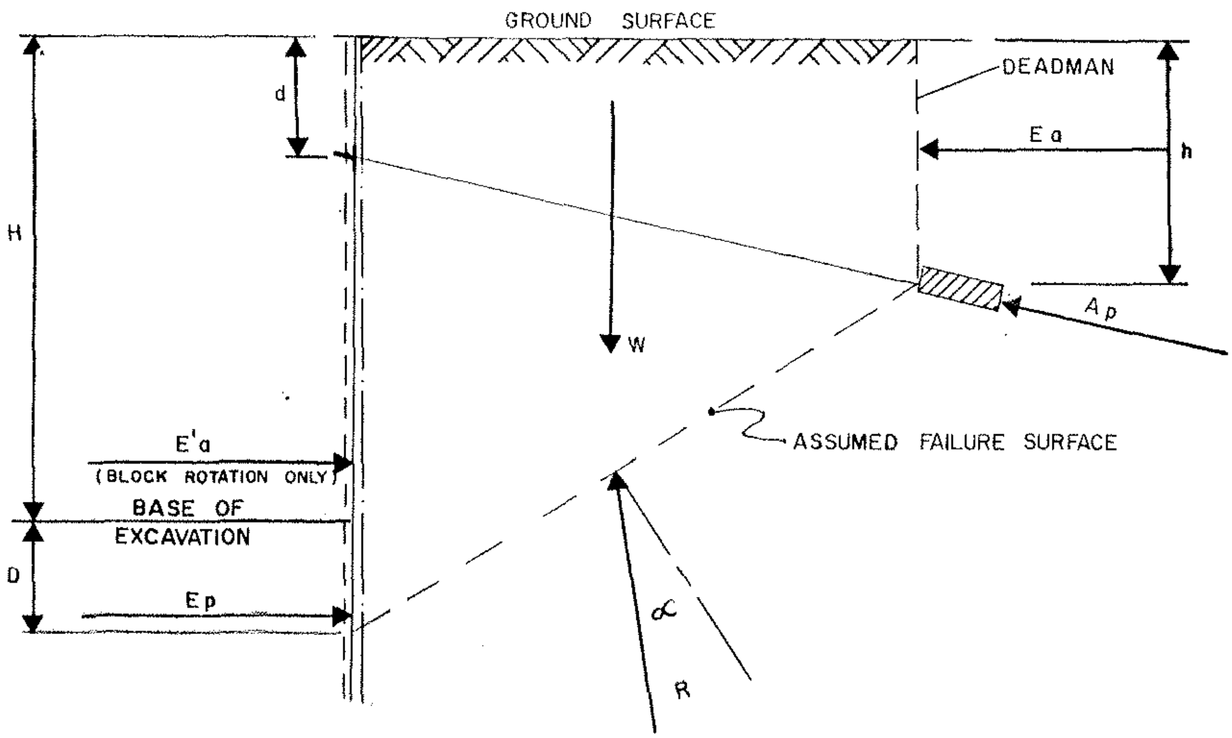
then:

$$Ka = \tan^2 (45 - 12.5) = .406$$

$$Kp = \tan^2 (45 + 12.5) = 2.464$$

Note that the factor of safety is applied to both pressure coefficients so that ϕ mob is consistent on all surfaces.

The stability of the system must be analysed in two steps: overall stability and block failure by rotation. The length of anchor required to equilibrate the system stability from either approach is determined and the longer anchor length is selected. For overall stability analysis the boundary of the system is shown in Figure 8 and a force



——·——·—— Boundary for analysis of overall stability

— — — — — — Boundary for analysis of block rotation

W = Weight of retained soil block

Ea = Active earth pressure on deadman

Ep = Passive earth pressure on the wall

R = Resultant force acting on the assumed failure surface

α = Inclination of resultant force to the normal to the failure surface

Ea′ = Active earth pressure exerted on the wall

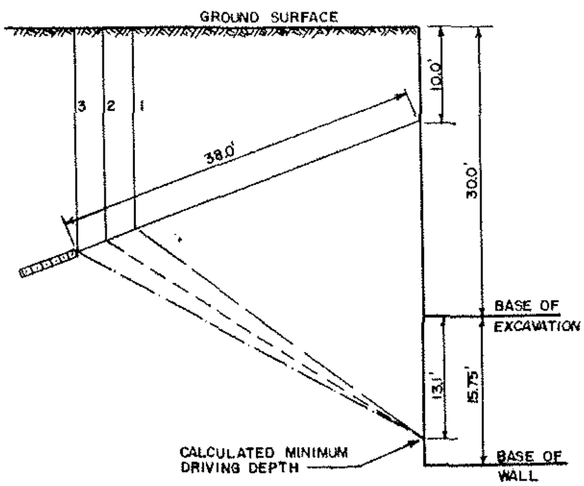
Ap = Anchor load

Figure 8: Force system to be considered in System Stability.

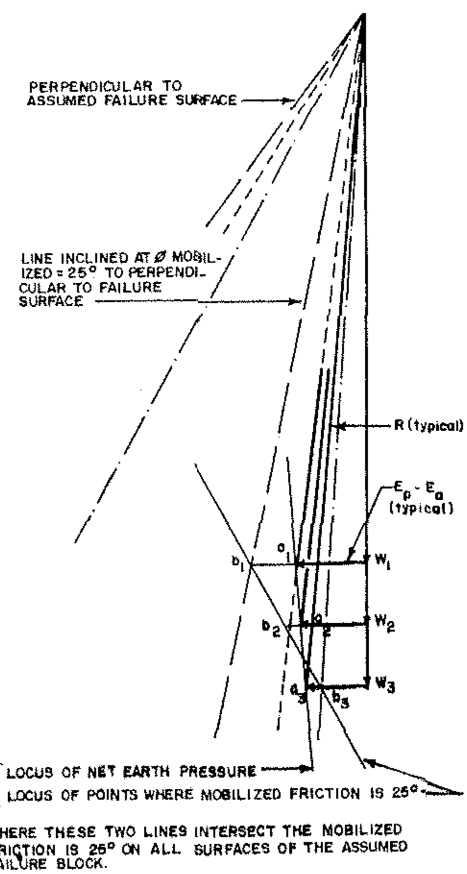
system is drawn. When the resultant force on the failure surface is inclined at Ø mobilized to the surface normal then the required length of anchor has been attained. The origin of the failure surface originates at the minimum penetration depth of 13.1 feet.

A graphical procedure is suggested using the force system shown in Figure 8. The solution is presented in Figure 9.

a) Assumed failure geometry



b) Force Polygon



From Figure 8

W = Area of block x unit weight of soil

$Ea = 1/2\ Ka\,\gamma\,h^2$

$Ep = 1/2\ Kp\,\gamma\,D^2$

See page 59 for the graphical procedure to use to determine the required anchorage length.

Figure 9: Analysis of Overall Stability.

Graphical Procedure for the solution of Figure 9 and Figure 10.

1. Calculate the magnitudes of all forces for each assumed failure block.

2. Draw the polygon of forces and locate a_1 at the intersection of the net earth pressure force and resultant force on the failure surface.

3. Through O draw the perpendicular to the assumed failure surface and then lay off a line inclined at Ø mobilized to this line. The intersection of the latter line with the extension of the net earth pressure line locates b_1.

4. Choose another failure surface and repeat 1, 2, 3.

5. Draw the 'b' line to obtain the locus of points where the resultant force is inclined at Ø mobilized to the normal to the failure surface.

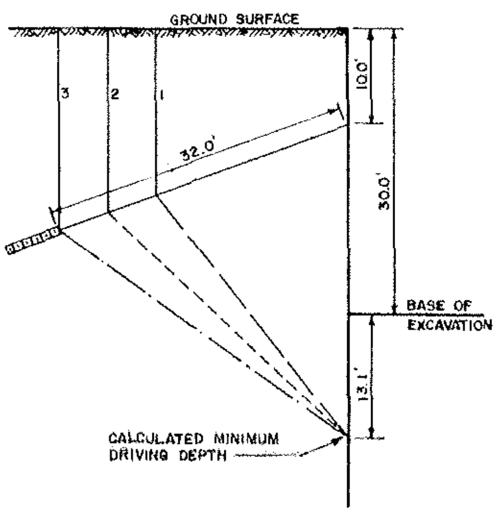
6. Draw the 'a' line to obtain the locus of points of net earth pressure where the resultant force is inclined at Ø to the normal to the failure surface.

7. The intersection of these two lines determines the origin of the resultant force which is inclined at Ø mobilized to the critical failure surface.

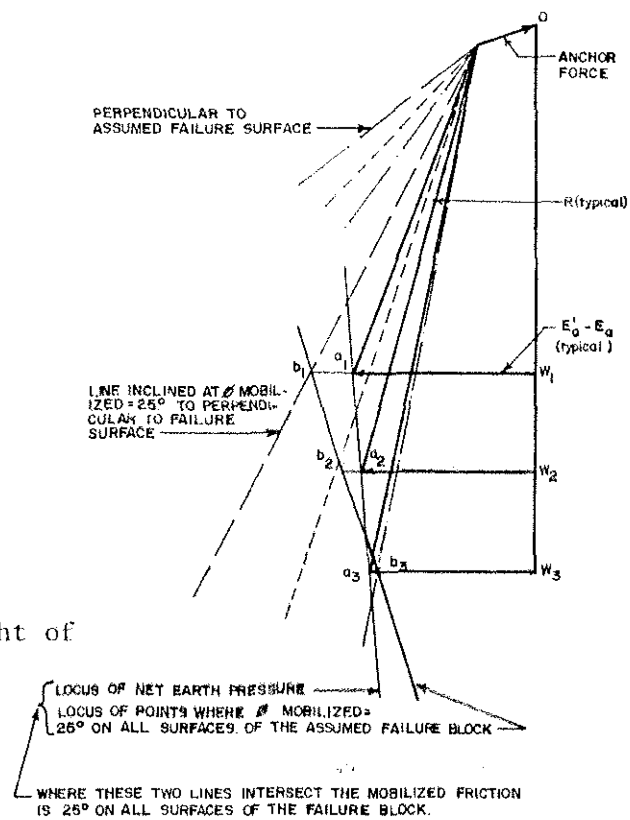
8. Large extrapolation is not recommended due to the non-linear nature of this analysis.

In order to investigate the failure by block rotation the wall is not included in the force system. Therefore, the anchor force must be included in the analysis. Again a graphical procedure is suggested using the force system of Figure 8. The solution is shown in Figure 10. Note that the passive force acting on the wall is excluded from this calculation and replaced by the active earth pressure which acts on the wall.

a) Assumed failure geometry



b) Force Polygon



From Figure 8.

W_1 = Area of block x unit of weight of soil

$E_a = 1/2\ \gamma Ka\ d^2$

$E_a' = 1/2\ \gamma Ka\ (H+D)^2$

T = Anchor force

Figure 10: Analysis of block rotation.

From both graphical solutions it is apparent that the system stability factor of safety is very sensitive to the anchor length.

The above analysis indicates that the required anchor length is 36.5 feet. (Figure 9)

3. Root Design

From above the computed anchor force is 10,345 lbs. For a factor of safety of 1.5 the root must be capable of carrying 1.5 x 10,345 = 15520 lbs.

A. Grout-Soil Interaction

The depth of embedment of the root from the stability analysis is 23 feet.

The pullout resistance is mobilized by the grout soil friction.

Therefore, $T = \pi d l \tau$

τ = frictional resistance at grout-soil interface

= $\sigma_v \tan \phi_{mob}$

= 120 x 23 x tan 25

= 1286 psf

l = root length

d = root diameter = 5" (construction technique)

Hence,

l = 6.15 ft.

B. Grout-Rod Interaction

The grout-rod adhesion offers the resistance to rod slip, therefore, $1.5T = \tau \pi \quad d \; l$

For smooth rod $\tau = .25f'_c = 25 \times 3000 =$

750 psi

For a 1.0" diameter rod $\sigma = \frac{F}{A} = \frac{10{,}345}{1.0 \times 3.14} \times 4$

$= 13{,}170$ psi which is less than 36000 psi

Therefore, $l = .55$ ft.

The required root length is 6.2 ft. from A. The final configuration of the wall is shown in Figure 11.

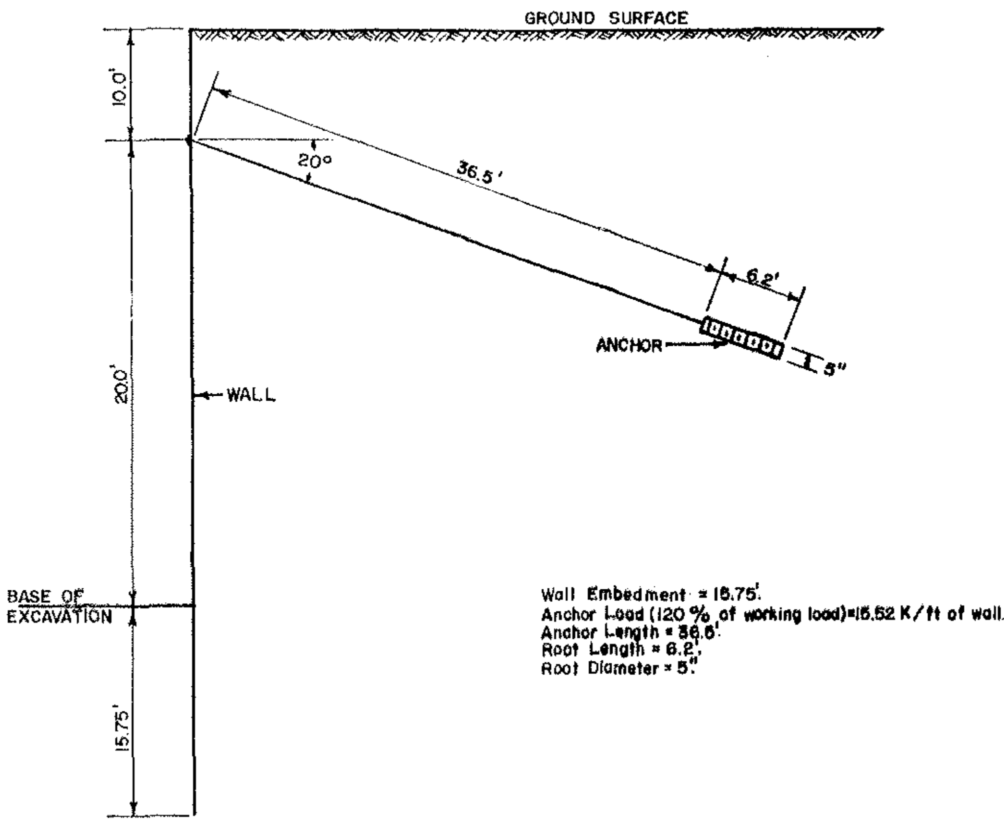


Figure 11: Designed configuration of the wall.

CHAPTER 7

TIED-BACK WALLS IN BEDDED DEPOSITS

In bedded deposits a pre-determined plane of weakness exists along the bedding planes. Hence, neither the active earth pressure nor conventional failure geometry will exist. The passive pressure acting on the wall will be much higher than for homogeneous deposits also.

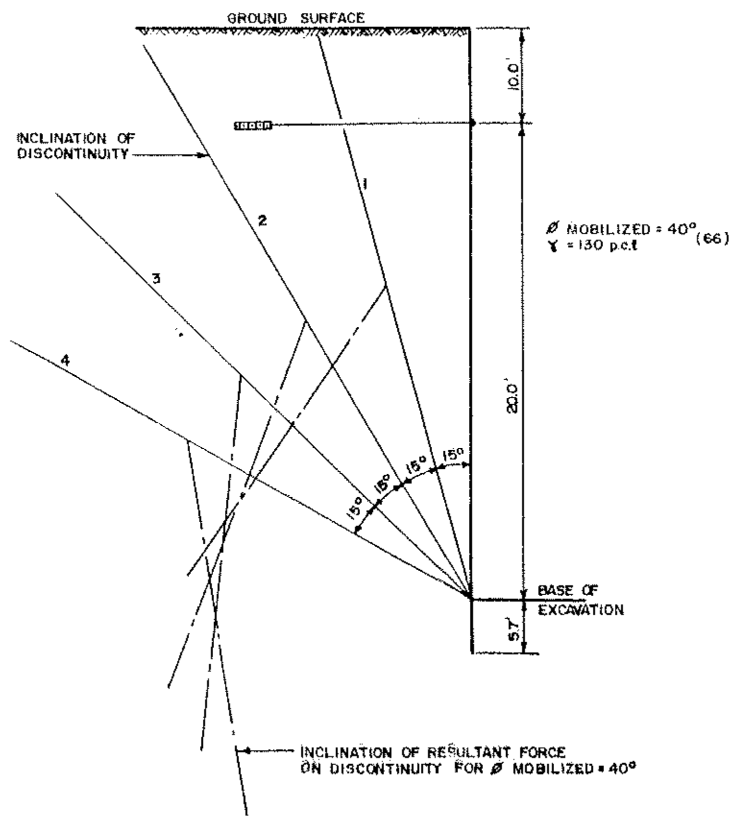
In order for one to gain an appreciation for the effect of discontinuities, the variation of the anchor load required to equilibrate the system is plotted as a function of the inclination of the discontinuity. In the free earth support method of analysis, a known active earth pressure is required to calculate the depth of embedment of the wall. However, due to the pre-existing failure plane the magnitude of the lateral earth pressure is unknown.

Hence, for the purposes of this analysis the active and passive pressure coefficients will be used to calculate the required depth of embedment. The factor of safety will be taken as unity to account for the unknown passive resistance which will be larger than for homogeneous soils.

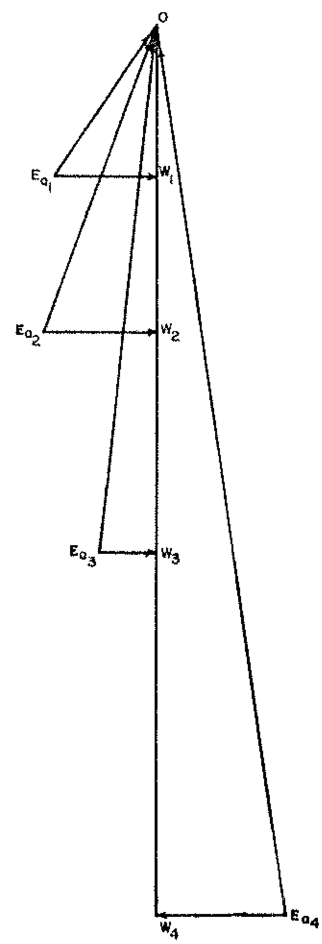
Using the equations as outlined in Chapter 6, the required depth of embedment to equilibrate the system is 5.7 feet. Figure 12 shows the variation between required anchor load and inclination of discontinuity. It is interesting to note that the maximum anchor load occurs where the lateral earth pressure is essentially equivalent to the active earth pressure of a homogeneous soil. Hence, the effect of the discontinuity is negligible.

This confirms the findings of Okusa (60) that the mobilized active (passive) pressures will not be greater (less) than those active (passive) pressures exerted by a homogeneous soil.

a) Configuration of Wall



b) Force Polygon



c) Variation of anchor load with inclination of discontinuity.

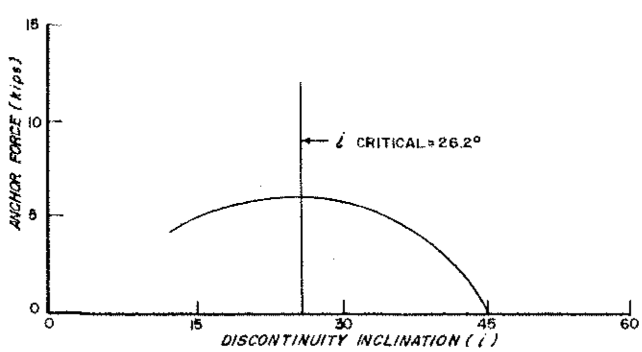


Figure 12: Effect of continuous discontinuity on net earth pressure.

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