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CONCEPTS FOR IMPROVED LATERAL SUPPORT SYSTEMS

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16. Abstract

Recommends improved designs for further research. Emphasis is upon developing procedures to understand and control displacements adjacent to deep excavations. Concepts include both analytical procedures and construction techniques.

Suggested methods to improve performance are (1) Use of stiff walls and pressurized wells in cohesive soil; (2) Prevention of ground loss in "running ground"; (3) Tiebacks and internal bracing combined in a hybrid support system which will provide superior performance over each system alone; and (4) Earth reinforcement concepts, both horizontal and vertical, designed to diminish the deformability of the retained soil mass.

Finally, tests sections are proposed to evaluate the concepts advanced in the report.

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PREFACE

This report examines concepts for improving the performance of lateral support systems; it also recommends areas of future research including details for prototype test sections.

The recommendations contained herein are based on the research conducted in preparing the three-volume report "Lateral Support Systems and Underpinning" (Volume I Design and Construction, Volume II Design Considerations, and Volume III Construction Methods). The extensive review of displacements presented in Volume II, Design Considerations, identified factors which contribute to displacements and thus provided the basis for the development of these ideas. The ultimate objective, of course, is to minimize risk, cost, and time of construction.

This publication is produced under the sponsorship of the Department of Transportation research program to advance the technology of tunnel construction.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the assistance of Dr. Lewis Edgers of Tufts University, Medford, Massachusetts, who contributed heavily to Section 3.30, "Time Dependent Movements".

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INNOVATIVE CONCEPTS

1.00 INTRODUCTION

The concepts presented in this report fall into three major categories:

a) Existing Contruction Techniques. Suggestions are made as to how some existing techniques may be more effectively utilized to mitigate potential sources of displacements.

b) Analytical Techniques. Analytical techniques are proposed relative to:

1) Parameters affecting lateral displacement of tied-back walls and,

2) Parameters governing lateral creep of tied-back walls in heavily overconsolidated clay.

c) "New" Construction Techniques. While the methods inherent in these techniques are presently available, the proposed applications and the rationale behind these applications are new.

The rationale behind each technique are discussed in detail and areas for future research are identified.

2.00 EXISTING CONSTRUCTION TECHNIQUES

This section calls attention to use of existing technology for the specific purpose of reducing displacements, usually in situations where the applicability of the existing method (or principle) is not commonly recognized.

2.10 STIFF WALLS

Goldberg, et al (1976b) (Displacements) conclude that as the wall-support system became stiffer, the magnitude of the movements in the adjacent soil mass decreased. The effect is particularly noticeable in clays and is related primarily to the "uncontrolled deformations" below the last placed wale level rather than to simple flexure between wale levels already in place.

The stiffness of the wall-support system can be increased by either increasing the wall stiffness (EI) or decreasing the spacing, L, between bracing or tieback levels. The controlling parameter is the stiffness factor, EI/L^4.

When stiff wall-support systems are required, a diaphragm wall is generally used. Another way of achieving the same objective is to decrease the spacing below levels of support. As a practical matter, there is a limitation on the proximity of spacing between strut levels because of their infringement upon the work area. On the other hand, tiebacks could be used effectively in this regard--either alone, or in combination with struts. An application of tiebacks to provide an intermediate level of support between tiebacks is advanced in Section 4.00.

Another technique is the use of "floating temporary" struts, set during excavation midway between the last placed strut level and the next lower strut level. Again, the objective is to mitigate settlement from "uncontrolled deformation" (see Figure 1).

The general features are:

a) To prefabricate the temporary struts in pairs, attached to a waling member.

b) Design the struts as telescoping units so that they can be expanded and retracted.

c) Expansion of telescoping units is necessary to preload against the sheeting. Retraction is necessary so that the tandem pair of telescoping struts can clear the next lower level of struts and wales.

d) Recover the telescoping units after completion of the cut and reuse on another section.

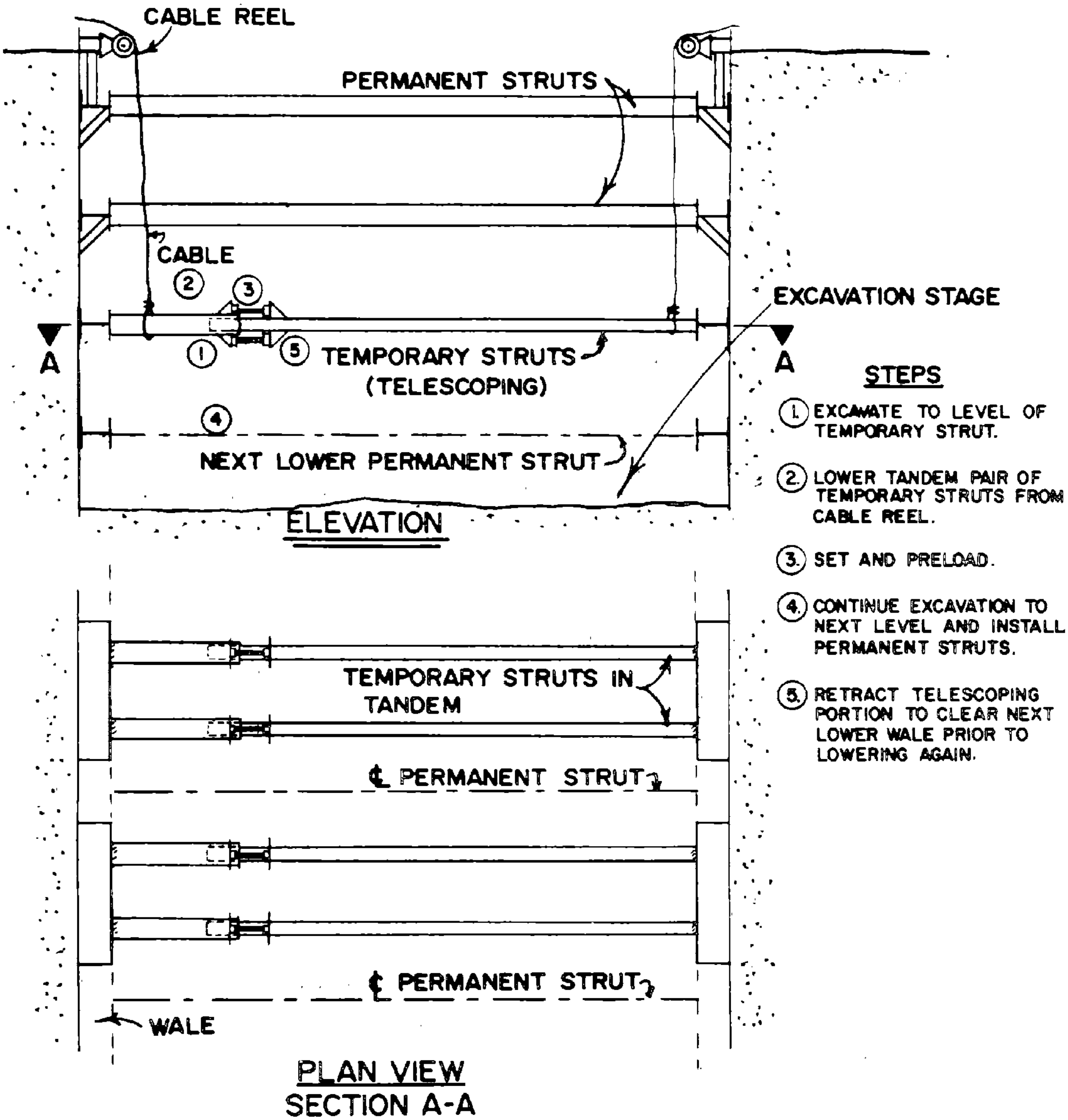


Figure 1. Temporary struts.

2.20 PERMANENT WALLS

The use of diaphragm walls, which are incorporated into the final structure and serve the dual purpose of temporary support walls and permanent walls, is well established in Europe. Historically, use of diaphragm walls in the United States has been primarily as temporary support walls only, with the permanent tunnel then constructed within the cofferdam in the conventional manner.

There are several disadvantages to the scheme of using the diaphragm wall as the permanent wall. The first disadvantage is seepage through joints in the wall or through holes made for tiebacks. These can, of course, be grouted. Also, an underdrain and pump system usually will be required.

Second, the appearance of an unfinished wall may be unacceptable. In stations and other areas where aesthetics are important, precast units may be used, or in the case of cast-in-place walls, a cosmetic facing may be constructed to improve the appearance. This may take the form of a brick wall offset within the interior of the diaphragm wall or may be mortar applied directly to the face of the cast-in-place wall. The former would be preferred if seepage through the wall is a factor.

2.30 RUNNING GROUND LAYERS

When an excavation is made through a soil profile that contains potentially running soils (silts and fine sands below the water table), it may be difficult to prevent loss of ground with use of soldier pile walls with horizontal lagging. Because of their slow rate of drainage and tendency to remain saturated for long periods, dewatering is difficult and time consuming. This is especially true when the stratum of concern is interbedded with and/or underlain by impervious deposits within the depth of excavation. In this latter case, the final mop-up of water is slow because there is effectively little or no head just above the impervious layer.

2.31 Grouting or Freezing

Figure 2 illustrates a case where either grouting or freezing techniques could be used to stabilize a running ground deposit. If these deposits are stabilized, a soldier pile and lagging wall may be used to support the excavation without fear of ground loss.

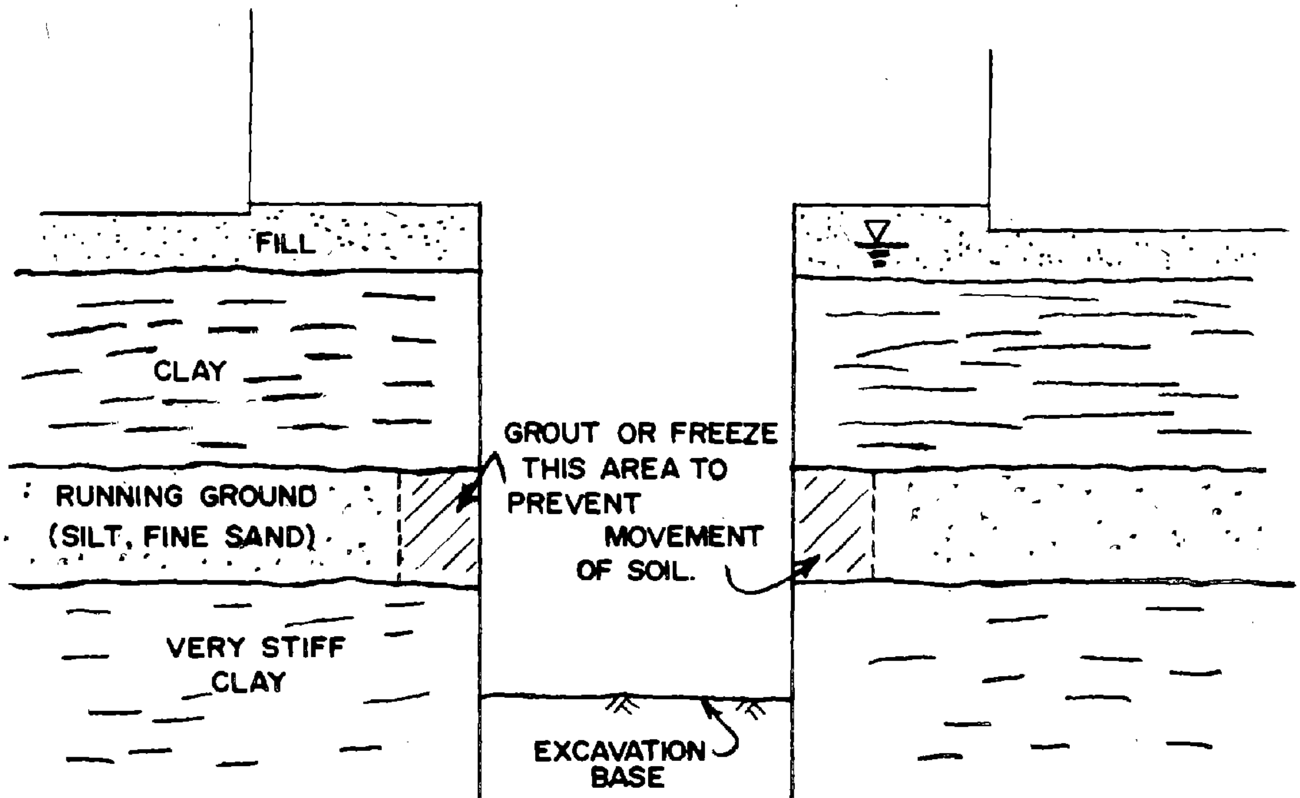


Figure 2. Grouting or freezing techniques to stabilize running ground (soldier pile wall).

It is believed that freezing will be less costly than grouting. First, grouting is done from the surface, whereas freezing may be done locally within the excavation. Second, the soil types of concern are relatively fine-grained and therefore will require the more expensive chemical grouts and more sophisticated techniques; and third, freezing need only continue long enough to permit cutting the face, installing and backpacking the lagging. This eliminates one of the major cost factors of a freeze wall--the maintenance of the freezing plant for prolonged periods. Indeed, a portable freezing unit may be applicable in such situations or alternatively, liquid nitrogen may be used, thus eliminating the freezing plant.

2.32 Vertical Sheeting

As with grouting and freezing, vertical sheeting may be used with soldier pile walls to prevent ground loss when "running ground" is exposed. The schematics shown in Figure 3 are based upon techniques presented by Weissenbach (1972) and illustrated in Chapter 2 (Soldier Pile Walls) (Goldberg, et al, 1976c).

The procedure, simply stated, is to use conventional lagging where the threat of ground loss is absent. Where potentially running ground is encountered, vertical steel sheeting is driven offset slightly from the soldier pile flange. The vertical sheeting is restrained by a steel wale and wood blocking.

3.00 ANALYTICAL CONCEPTS

This section discusses some new ideas that may be used to evaluate tied-back wall stability and the movements of the wall.

3.10 INTERNAL STABILITY (COFFERDAM) ANALYSIS

The method assumes that the tiebacks embody the soil mass behind the wall and that the soil mass can be idealized to act as a double wall cofferdam or a deep beam. Using the method of analysis described in Teng (1962) (from Terzaghi, 1945), an expression for the stability of the cofferdam (beam) is obtained.

Figure 4 illustrates the idealized loading conditions for this case. As is the case for a beam in flexure, the maximum shear stress occurs on the neutral axis. The location of the neutral axis and the direction of the maximum obliquity are complex functions of the magnitude of external loadings,

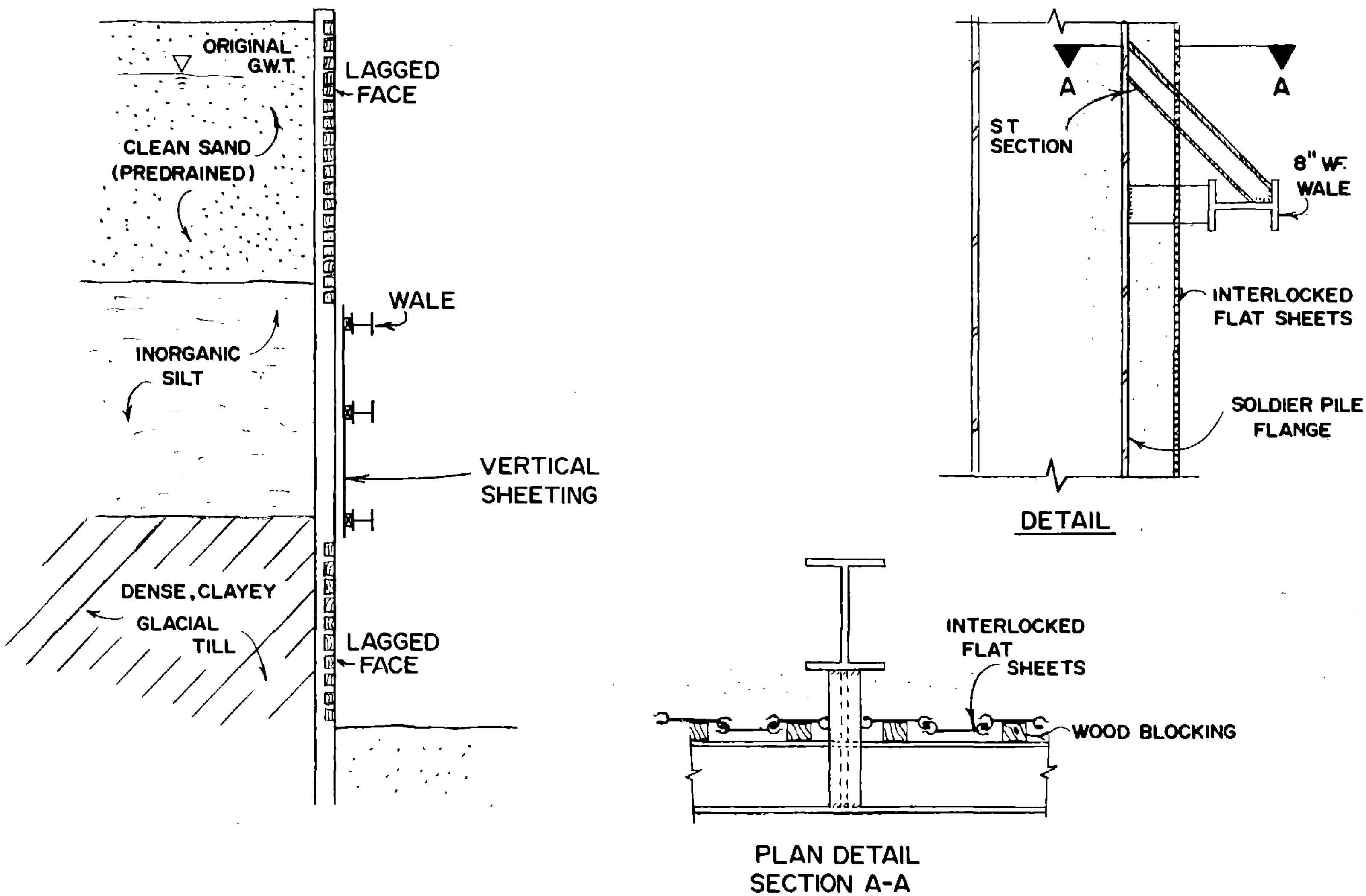


Figure 3. Vertical sheeting to stabilize running ground.

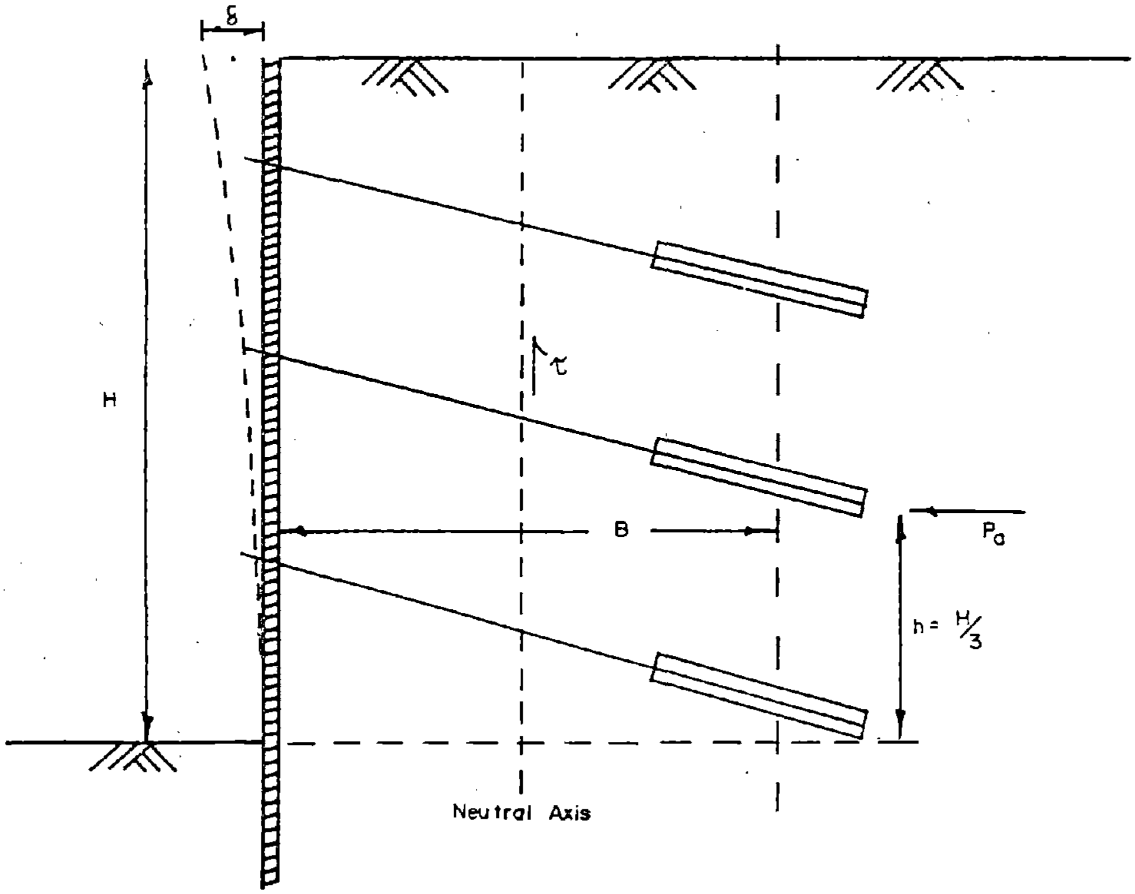


Figure 4. Sketch of equivalent cofferdam for tied-back wall.

the unit weight of the embodied soil, and the strength and deformability of the embodied soil. Therefore, in engineering practice, the assumption is made that the maximum shear stress occurs on the vertical midplane of the cofferdam. The shear force can then be computed as:

$$V_{max} = \frac{3M}{2B} \qquad \text{Eq. 3.10-1}$$

where:

$$M = \text{moment} = P_a \times \frac{H}{3} = \frac{\gamma K_a H^3}{6} \qquad \text{Eq. 3.10-2}$$

B = effective width

The shear resistance at any point on the assumed failure plane is:

$$s = \bar{c} + \bar{\sigma}_h \tan \phi \qquad \text{Eq. 3.10-3}$$

where:

K_a = coefficient of active earth pressure

γ = unit weight of soil

s = shear resistance

$\bar{c}$ = cohesion intercept

ϕ = angle of internal friction

$\bar{\sigma}_h$ = effective normal stress on the failure plane (horizontal stress)

However, assuming a vertical failure plane, it can be shown from Mohr's circle that $\bar{\sigma}_h$ is not related to $\bar{\sigma}_v$ by active earth pressure, but as follows:

$$\bar{\sigma}_h = \frac{1}{1 + 2\tan^2\phi}\,\bar{\sigma}_v \qquad \text{Eq. 3.10-4}$$

$$\therefore\ s = \bar{\sigma}_h \tan \phi = \frac{\tan \phi}{1 + 2\tan^2\phi}\,\bar{\sigma}_v \qquad \text{Eq. 3.10-5}$$

(for the case of $\bar{c} = 0$)

The total shear resistance for a backfill with a unit weight, γ, and height of backfill, H, is as follows:

$$S = \frac{1}{2} \gamma H^2 \frac{\tan \phi}{1 + 2 \tan^2 \phi} \quad (\gamma = \text{constant}) \qquad \text{Eq. 3.10-6}$$

The safety factor against internal shear failure therefore becomes:

$$\text{F.S.} = \frac{\text{available shear resistance}}{\text{maximum shear force}} = \frac{S}{V_{max}} \qquad \text{Eq. 3.10-7}$$

If the heights and unit weights of the backfill and driving soils are equal, the factor of safety becomes:

$$\text{F.S.} = \left(\frac{2B}{H}\right) \frac{\tan \phi}{1 + 2 \tan^2 \phi} \frac{1}{\tan^2(45 - \phi/2)} \qquad \text{Eq. 3.10-8}$$

The foregoing expression is not proposed as a precise analytical tool. Rather, it is suggested as the basis for research to develop a semi-empirical technique to evaluate stability.

Fundamentally, the equation suggests that the safety factor is proportional to: a) the width (B) of the zone of earth embodied by the tiebacks b) the depth (H) of the excavation, and c) the strength parameter (ϕ) of the soil. In summary:

F.S. is a function of $\frac{B}{H}$ and ϕ.

The double wall cofferdam analysis also forms a basis for predicting horizontal movement at the top of the wall by the following procedure:

$$\delta = \alpha H = \frac{\tau}{G} (H) \qquad \text{Eq. 3.10-9}$$

where:

δ = Horizontal movement at top
α = Angular rotation due to shear
τ = Average shear stress
H = Height of wall
G = Shear modulus

Based upon equations 3.10-1 and 3.10-2, the shear force on the "neutral" axis is:

$$V = \frac{3M}{2B} = \frac{\gamma K_a H^3}{4B} \qquad \text{Eq. 3.10-10}$$

The average shear stress is then:

$$\tau = \frac{\gamma K_a H^2}{4B} \qquad \text{Eq. 3.10-11}$$

Substituting equation 3.10-11 in equation 3.10-9:

$$\delta = \frac{\gamma K_a H^3}{4 GB} \qquad \text{Eq. 3.10-12}$$

Nendza and Klein (1974) state that the deflection of the soil, due to the mobilization of shear, is equal to $= \frac{K_a \gamma H^3}{6GB}$.
They do not show the derivation, but it is believed that the non-uniform distribution of shear stress accounts for the difference in the magnitude of the predicted displacements between this equation and equation 3.10-12.

Through comparisons with a number of case histories, the observed displacements are generally significantly larger than those predicted by equation 3.10-12. Certainly, a great many factors affect movements which cannot be accounted for theoretically. Ground loss, settlement of the wall, flexure of the wall, and movements below the base are but a few of these factors.

As with the expression for safety factor (Eq. 3.10-8) the writers do not propose equation 3.10-12 as a rigorous solution, but it does demonstrate a valid principle for tied-back walls--namely, that horizontal displacement at the top is proportional to the active coefficient (K_a) and the third power of the excavation depth (H). It is inversely proportional to the shear modulus (G) and the width of tieback zone (B). In summary:

$$\delta \text{ is a function of } K_a,\ H^3,\ \frac{1}{G},\ \text{and } \frac{1}{B}$$

These principles form the basis for further research to develop a semi-empirical relationship for prediction of displacements at a tied-back excavation.

3.20 MOVEMENT BELOW EXCAVATION BASE

Large tied-back wall movements originating below the excavation base have been reported by Breth and Romberg (1972) and Nendza and Klein (1974) among others. Nendza and Klein present a technique for predicting movements below the excavation base caused by relief of stresses on the excavation side and the consequent lateral load that the soil below the inside of the excavation must carry. This movement can be computed as follows:

$$\delta = \frac{\Delta\bar{\sigma}_x \, B/2}{E} \qquad \text{Eq. 3.20-1}$$

where:

δ = horizontal movement of the tied-back system occurring uniformly along the height of the wall

B = width of the excavation

E = horizontal Young's Modulus of elasticity of the soil at the base of the tied-back wall

$\Delta\bar{\sigma}_x$ = horizontal relief at the base of the excavation due to removal of soil from within from excavation

$\Delta\bar{\sigma}_x = (\alpha_1 + \alpha_2)\, \gamma H$

where:

γ = unit weight of excavated soil

H = depth of excavation

α_1, α_2 = simple numerical factors based on geometry of the excavation

Base shear movements, as computed by the above equation were compared to movements from shear deformations within the tied-back soil mass (Section 3.10) for a number of typical cuts in cohesionless soils. These simple analyses indicate that relative to the cofferdam shear analysis:

1. Base movements become less significant as the depth of the cut increases.

2. Base shear becomes more significant as tieback depth increases i.e. stability increases.

3. In all situations analyzed, base shear was highly significant, accounting for 50 percent to more than 90 percent of wall movement.

Also, computed tied-back wall movements, which include the effects of base movements, agree reasonably well with measured movements and deformation patterns as reported in a number of case histories.

There is no question that the equation presented above requires considerable development before it can be confidently applied. For example, in wide excavations the effective width of the excavation, B, must attain a limiting value. Also improved methods for computing $\Delta\bar{\sigma}_x$ must be developed.

Nevertheless, the basic premises of the equation are valid, namely movements are:

1. Directly proportional to the amount of stress relief, $\Delta\bar{\sigma}_x$, which is related to the pre-excavation overburden stress, γH;

2. Directly proportional to the excavation width, B (or effective width in wide excavation);

3. Inversely proportional to modulus E.

Application of this equation appears reasonable--certainly as a first order approach. Furthermore, the equation seems, quite correctly, to predict the large base movements that have been observed at a number of tied-back systems. At present, it is recommended that this equation be applied only with great discretion but that it serve as a basis for further analytical and empirical development based upon observed tied-back wall base movements.

3.30 TIME-DEPENDENT MOVEMENTS

In several excavations (St. John, 1974; Burland, 1974; Cole and Burland, 1972) time-dependent lateral movements of the soil were observed. These movements were particularly noticeable in tied-back and cantilevered support walls in heavily overconsolidated clays and occurred uniformly from the tops of the wall to a significant depth below the excavation base.

An attempt has been made to model these movements by means of two simple analyses. One analysis uses a model of primary consolidation (swell) in which lateral wall movements occur from relief of lateral pressures and consequent swelling of soil immediately behind the tied-back soil mass. That is,

$$\delta = \frac{T\ C_{hs}}{1 + e_o} \log \frac{\bar{\sigma}_{ho}}{\bar{\sigma}_{hf}} \qquad \text{Eq. 3.30-1}$$

where,

δ = lateral swell of tied-back wall

C_{hs} = horizontal primary swell index

e_o = void ratio of the soil

$\bar{\sigma}_{hf}$ = final average horizontal effective stress within the soil plug

$\bar{\sigma}_{ho}$ = initial (geostatic) average horizontal effective stress within the soil plug

T = lateral extent of zone behind tied-back plug subject to stress relief.

There are, of course, difficulties in correctly estimating a number of the parameters in the equation, such as C_{hs}, T, and $\bar{\sigma}_{hf}$ and this most likely should provide the focus for any future development of the model. However, the model quite logically predicts that the time-dependent swell depends upon the swell index of the soil C_{hs}, and the amount of stress relief behind the tied-back soil plug, $\bar{\sigma}_{ho} / \bar{\sigma}_{hf}$.

The second analysis uses a model in which the time-dependent wall movements occur because of secondary, rather than primary, swell of the soil mass immediately behind the tied-back soil plug. In this instance,

$$\delta = C_{\alpha h} \cdot T \cdot \log \frac{t}{t_c} \qquad \text{Eq. 3.30-2}$$

where

δ = lateral swelling of tied-back wall

$C_{\alpha h}$ = coefficient of secondary compression for horizontal swell

T = lateral extent of zone behind tied-back plug subject to stress relief

t = total elapsed time after initial relief of stresses

t_c = total elapsed time after initial relief of stresses at which the process of secondary swell initiates

As is the case with the primary swell model, there are difficulties in selection of the above parameters and this should provide a basis for further development of this model.

The results of the simplified analyses as compared to measured time-dependent movements indicate the following:

1. Movements computed by means of the primary swell mechanism are greater than those observed (St. John, 1974). This may be from improper selection of parameters for the cases at hand and may not in fact, be an indication that the assumed mechanism is invalid.

2. Movements computed by means of the secondary swell mechanism indicate reasonable agreement with measured time-dependent movements reported by St. John (1974). The rate of movement based upon this model of secondary swell also agrees reasonably well with measured rates.

Both mechanisms appear theoretically reasonable and warrant further study, especially in selecting parameters for use in the pertinent equations. The most promising approach, at least initially, would be the collection of additional performance data for correlation with soil properties, $C_{\alpha h}$ and C_{hs}, and field instrumentation for determination of the magnitude of stress relief ($\bar{\sigma}_{hf}$ vs. $\bar{\sigma}_{ho}$) as well as the extent of the affected area, T.

It must be emphasized, however, that although the models proposed above represent promising innovations to the state-of-the-art, they are, nevertheless, simplified models of very complex phenomena. There is no question that movements are not entirely one-dimensional, nor that stress conditions are nearly as uniform as implied by the above equations. Factors, such as shear deformations, time-dependent stress relaxation and redistribution, and changes in soil properties (especially modulus) with time, complicate the problem. Fortunately, a significant amount of work has been done in analyzing other complicated time-dependent geotechnical problems (Watt, 1969 and Edgers, 1973). These analyses, of necessity, use sophisticated laboratory tests and computerized finite element deformation techniques. One of the significant parameters considered in these analyses is the change in soil modulus with time. Physical phenomena that have been modeled include stress relaxation and time-dependent soil movements (creep) beneath embankments and building foundations. These factors certainly relate to the problem at hand.

There are other similarities between these complicated analyses and the tieback problem. For example, lateral creep of the soils below the base of the excavation may also be a cause of the time-dependent movements. It is well known that cohesive soils will continue to strain with time at constant stress levels (creep). Embankments on clay have exhibited this type of behavior (Edgers, 1973), and it has also been observed that large creep movements may occur at depth, apparently from transfer of stress through the deposit (stress relaxation). If this behavior also occurs in tied-back excavations (no internal bracing to restrain wall movements), one would expect the observed time movements to be deep seated, occurring below the base of the excavation. The limited data available on tied-back wall movement indicate that movements below the base of the excavation do in fact represent a large portion of the total movement. After the base elevation is reached, the wall will deflect laterally, a uniform amount from the top of the wall to a significant depth below the excavation base.

Further discussion of these newer analytical tools may be found in Sections 3.40 and 5.30.

3.40 DISCUSSION

The new analytical techniques presented in this section deal with tied-back wall stability and deformations. The cofferdam analysis is imperfect; however, it is believed that this method can be used in a preliminary analysis of stability. The analysis does identify key points in tied-back wall behavior and therefore may be further refined.

The deformation of a tied-back wall is complex. Shear in the earth mass within the tieback zone causes some movement while compression of the soil at the base of the excavation appears both theoretically (Nendza and Klein, 1974) and in practice to be a major source of wall movement. Further study into the nature of these movements is required particularly with respect to time-dependent movements. In permanent tied-back structures these time-dependent movements could be significant.

A research effort in tied-back wall movement should include in situ monitoring of support systems to provide an improved data base for further analytical developments. Emphasis should be placed on measurements of time-dependent movements and movements originating below the excavation base.

Initial analytical developments should include improvements in the relatively simple analyses for cofferdam stability and shear deformations, base movements, and time-dependent primary and secondary swell. This effort should consist of analysis of performance data for improved selection of significant parameters in these analyses such as $\Delta\bar{\sigma}_x$ (Eq. 3.20-1); T, C_{hs}, and C_{hf} (Eq. 3.30-1); and T and C_h (Eq. 3.30-2).

On the other hand, any extensive long term, research effort should consider in more fundamental terms the complex behavior of tied-back systems. Significant parameters would include primary and secondary swell behavior of soils, soil shear modulus, and changes in soil shear modulus with time. These soil parameters may be determined by means of laboratory testing. In addition, the complicated geometry and stress conditions in tied-back systems warrant mathematical analyses that are more sophisticated than the simple analyses described above. One promising approach would be the application of existing finite element analyses. Many finite element analyses include time-dependent formulations to describe soil behavior.

4.00 TECHNIQUES TO REDUCE DISPLACEMENTS

4.10 HYBRIDIZATION OF TIEBACKS AND INTERNAL BRACING

4.11 Background

Several factors cited by Goldberg, et al (1976b) suggest that tieback installations should result in less displacement than strutted excavations. Prominent among these are:

a) Greater prestressing with tiebacks.
b) No need for strut removal and rebracing.
c) The embodiment of an earth mass by tiebacks, thereby making it less deformable.
d) Generally, less overexcavation below a support level.
e) More convenient to have closer vertical spacing.

On the other hand, with tied-back walls there may be more movement near the top of the wall than with internally braced walls.

Through hybridization of tiebacks and internal bracing, it is believed that the best features of both can be combined to reduce displacements that might otherwise occur.

An obvious prerequisite is that there must be a suitable zone in which to anchor tiebacks. These would be rock, granular soils, or very stiff to hard clays. In these soil profiles the beneficial effects of anchor prestressing are most notable.

4.12 Common Features

Some possibilities for tieback-internal bracing hybrids are shown in Figures 5 and 6. All schemes have the following features in common:

a) Struts at the top to prevent inward movement.
b) Tiebacks near the bottom to limit displacements during strut removal and rebracing.

4.13 Tiebacks and Struts Sharing Full Load

The two schemes presented in Figure 5 differ from the scheme shown in Figure 6 in that tiebacks in the latter case act only as "earth reinforcement". In the latter case, the internal bracing carries the full lateral pressure of earth, water, and surcharge.

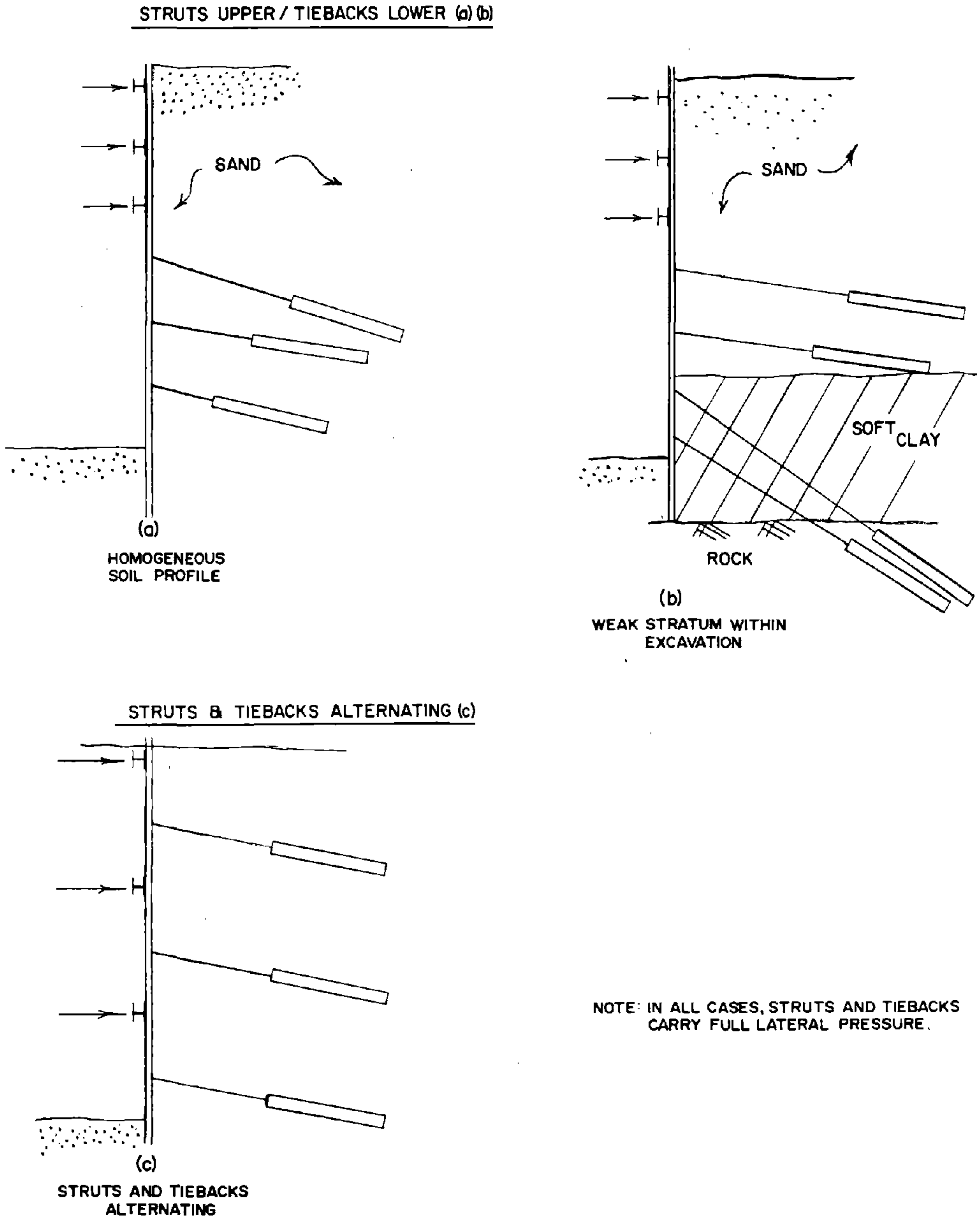


Figure 5. Tiebacks and struts sharing full load.

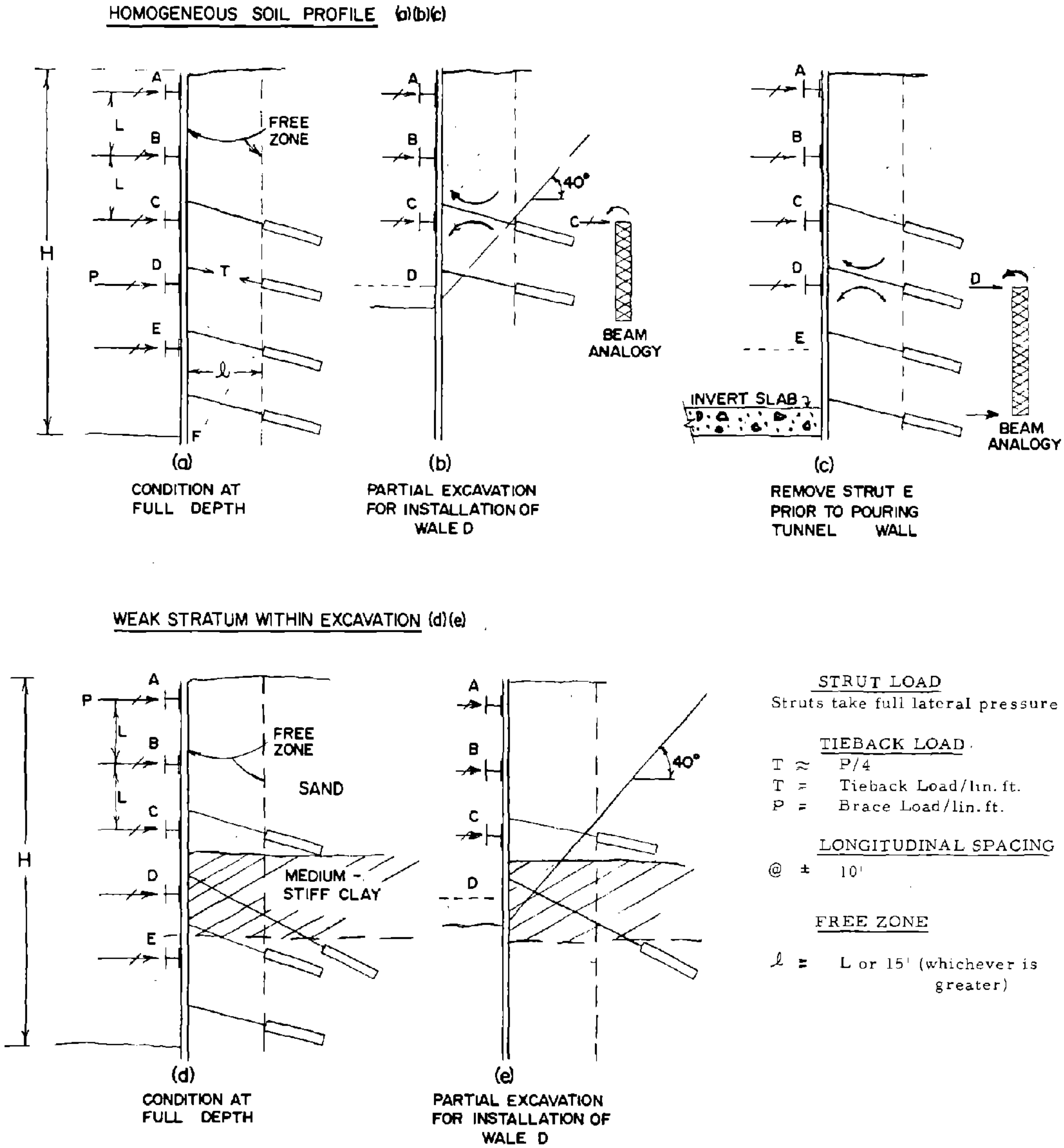


Figure 6. Tiebacks as earth reinforcement with struts taking full load.

The "struts upper/tiebacks lower" arrangement is shown in Figure 5a and 5b. This scheme is a natural outgrowth of the previously advanced concepts to prevent lateral movement at the top and to eliminate strut removal and rebracing near the bottom.

On the negative side is the question of load distribution and, because strain is prevented at the top, concern over possible load concentration. The risk is acute with only one strut at the top. With two struts, it is possible that the top strut could be unloaded due to rotation of the elastic line as excavation proceeds below the second strut. Concurrently, this may cause overloading the next to the top strut. For this reason, three struts would be preferred.

Figure 5b shows how tiebacks can be effectively used to limit displacements when the excavation penetrates a weak layer of clay. They provide the opportunity to have the support levels closely spaced without cluttering the work area with bracing.

The second method has tiebacks and struts at alternate levels. The arrangement shown in Figure 5c provides a top strut (to prevent top movement) and a bottom tieback to eliminate the strut removal and rebracing at the deepest portion of the excavation.

It is believed that this method has less contingency concerning overloading because struts are evenly dispersed rather than concentrated at the upper portion of the wall as in the scheme shown in Figure 5a.

Also, this method would seem to offer inherent advantages with regard to the benefits of preloading. Normally, braces are preloaded to about 50 percent of the design load. Under the arrangement with alternating tiebacks (usually tested to 125 percent and locked-off at 75 to 100 percent of the design load) it appears to be possible to preload bracing to higher than normal without fear of overloading. The reasons for this are:

a) Local anomalies tend to be masked out by the earth mass embodied by tiebacks.

b) There is evidence to indicate that there is better control over load distribution with tiebacks.

c) Tiebacks can be restressed far more easily than can bracing after initial installation.

d) Bracing and tiebacks can be preloaded at the same time to achieve improved load balance. For example, say that a bracing level is preloaded temporarily to 50 percent of the design load, and the excavation is then advanced to the tieback level. In this sequence, it is believed that the braces could be preloaded to 75 percent of design load concurrently with tiebacks being stressed to 100 percent of the design load.

As a final point, the substitution of tiebacks for bracing facilitates the excavation process by providing greater vertical distance between levels of bracing than in the case of bracing alone.

4.20 EARTH REINFORCEMENT

4.21 Background

The three major variables controlling wall movements are the stiffness of the support wall (EI), the spacing between support levels (L), and the strength and deformation characteristics of the soil. The objective of earth reinforcement is to strengthen and to improve the deformation characteristics of the soil through the mechanism of developing monolithic action of an earth mass.

The idea of reinforcing or improving the properties of an earth mass is not new. Two techniques which embody an earth mass as a monolith by the insertion of reinforcing elements are the "Reticulated Wall" and "Reinforced Earth". The Fondedile "Reticulated Wall" uses steel reinforcing in an array of pressure-grouted holes (Goldberg, et al, 1976c, Underpinning). More commonly, the term "Reinforced Earth" refers to a system of horizontal metal strips placed in the earth during backfilling to create a retaining structure. Obviously, this latter procedure is not directly applicable to cut-and-cover or soft ground tunneling projects.

Grouting and freezing are two other techniques used to reinforce the earth mass.

4.22 Tiebacks as Earth Reinforcement

Figure 6 shows a hybrid scheme in which braces carry the full lateral pressure. The purpose of tiebacks is primarily to reinforce the whole earth-wall system by the following mechanisms:

a) Shortening the span distance between wale levels by providing an intermediate point of support.

b) Removing the slack that will exist between wale, wall, and soil, especially in loose soil where a lagged wall is used.

c) Embodying an earth mass which effectively increases the stiffness of the wall. This is similar to increasing the "EI" of the wall. Goldberg, et al (1976b) (Displacements) comment on the importance of wall stiffness factor, $\frac{EI}{L^4}$.

d) Bridging across weak zones, as is shown in Figures 6d and 6e.

Suggested design criteria are based upon the following:

a) Free Zone - Lowest anchors must be outside the active wedge created by sequential bracing and excavation.

b) Longitudinal Spacing - Should be close in order to develop monolithic action in soil. Provide anchor at each soldier pile or at $\pm$ 10 feet. This is about equal to or slightly less than conventional distances between wales.

c) Load - Nominal only. A line load equal to 25 percent of the line load on struts is suggested. This is intuitively believed to be what is required to provide intermediate support between struts and at the same time, provide sufficient prestress in the earth for "beam action".

Tiebacks would not have to be proofloaded in the same manner as production ties in a conventional tieback installation. Perhaps 10 percent to 25 percent would require special tests to demonstrate capacities. The remainder would merely be stressed to 100 percent of design load, held for 5 minutes and then locked off. Typical loads would be in the order of 30 to 40 kips.

4.23 Ice Wall as Earth Reinforcement

The objective is to increase the EI of the wall to reduce displacements. Applicable situations are when a weak layer (e.g. soft clay) lies within, or occurs below, the depth of the excavation.

As has been noted by Goldberg, et al (1976b), one of the principal difficulties with excavations in soft clay is the question of "uncontrolled displacements" - that is, displacements occurring below the last placed strut level. Experience has shown that this often amounts to over one-half the total of all lateral displacements. Figure 7a schematically shows the development of lateral displacements with increasing excavation.

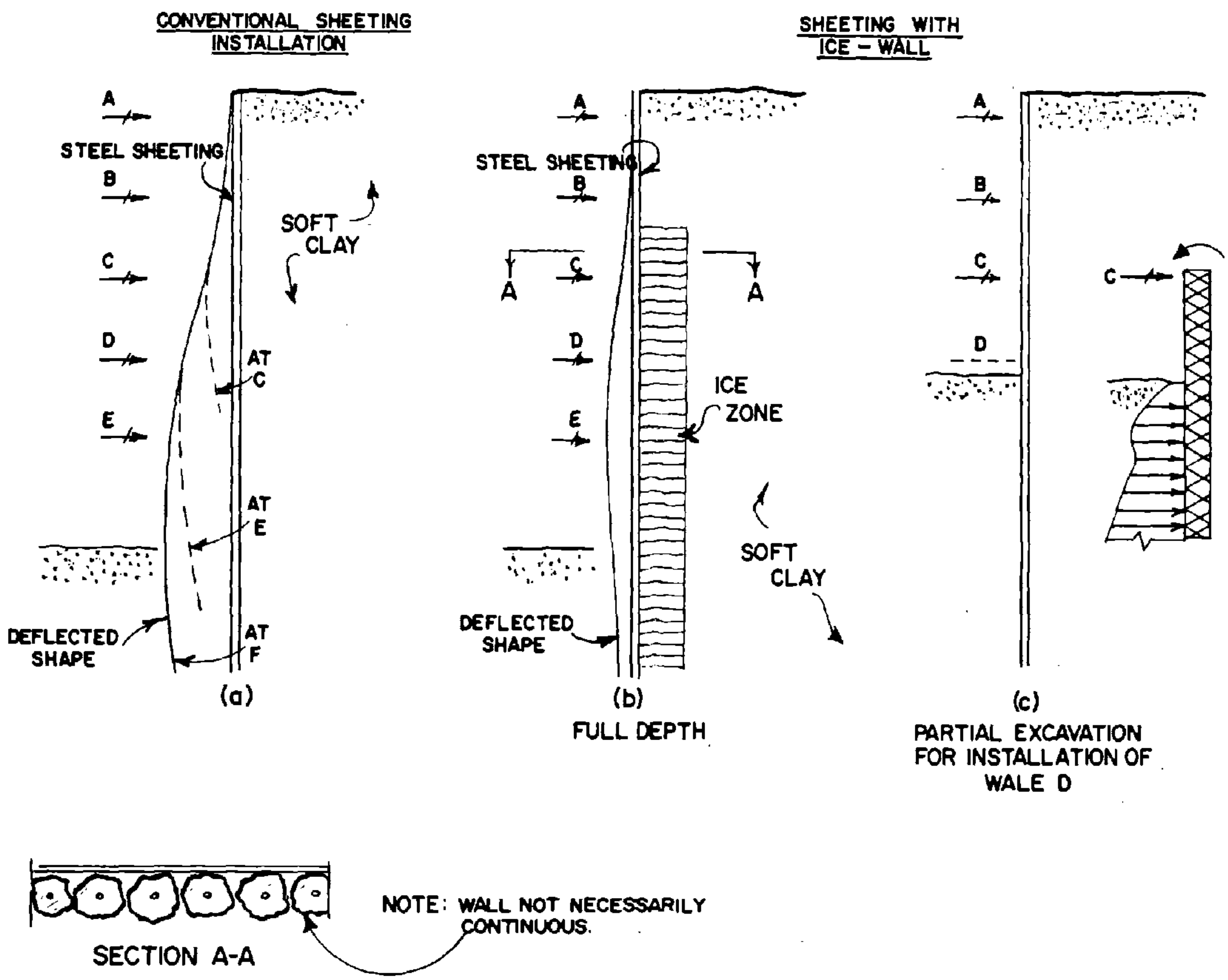


Figure 7. Ice wall as earth reinforcement.

The use of an ice wall to reinforce the earth in a situation as described above offers promise (see Figure 7). The primary objective is to stiffen the soil below the excavation in order to limit the so-called uncontrolled displacements. Because the ice wall is not the primary support system, it need not be continuous. Indeed, it would appear reasonable to have a series of discrete ice columns, analagous to soldier piles, to achieve the required stiffening effect.

It would only be necessary to maintain the freezing plant during the excavation stage, thus eliminating one of the major cost factors of a freeze wall-- the maintenance of the plant for prolonged periods after the excavation has "bottomed-out".

4.24 Vertical Soil Reinforcing

The principle presented here is much the same as that of the reticulated wall. However, it is felt that the concept can be expanded to cover many types of reinforcing and many different patterns or configurations. Reticulated walls are often designed to act as underpinning support or as a retaining wall. The purpose of the more general earth reinforcement concept expressed here is to strengthen the earth mass to supplement another ground support wall principally with the intent of reducing lateral movements.

As with the ice wall method described above, the objective is to increase the EI of the wall to reduce displacements. Again, this application is most relevant to the situation of relatively weak cohesive soils materials in which excessive strains below the last placed wale level is a major factor in causing displacements.

Figure 8 schematically illustrates this concept.

As conceived, the objective is not to introduce dowels, which act independently of soil. Rather, the objective is to develop composite action of soil and the reinforcing element much like reinforcing acts in concrete beams. Therefore, the reinforcing must be spaced close enough to make the soil and reinforcing act as a single unit. Soil arching and the ability of the soil to transmit shear stresses to the reinforcing are major considerations. The degree to which composite action develops can best be assessed by experimentation, perhaps by use of laboratory models.

Potential reinforcing elements would be augered, reinforced concrete piles. Large diameter piles, typically made with hollow stem augers and generally about 12 inches to 18 inches in diameter, are the same units used for foundation piles or for tangent pile walls (Goldberg, et al, 1976c) (Concrete Diaphragm Walls). Another type of element

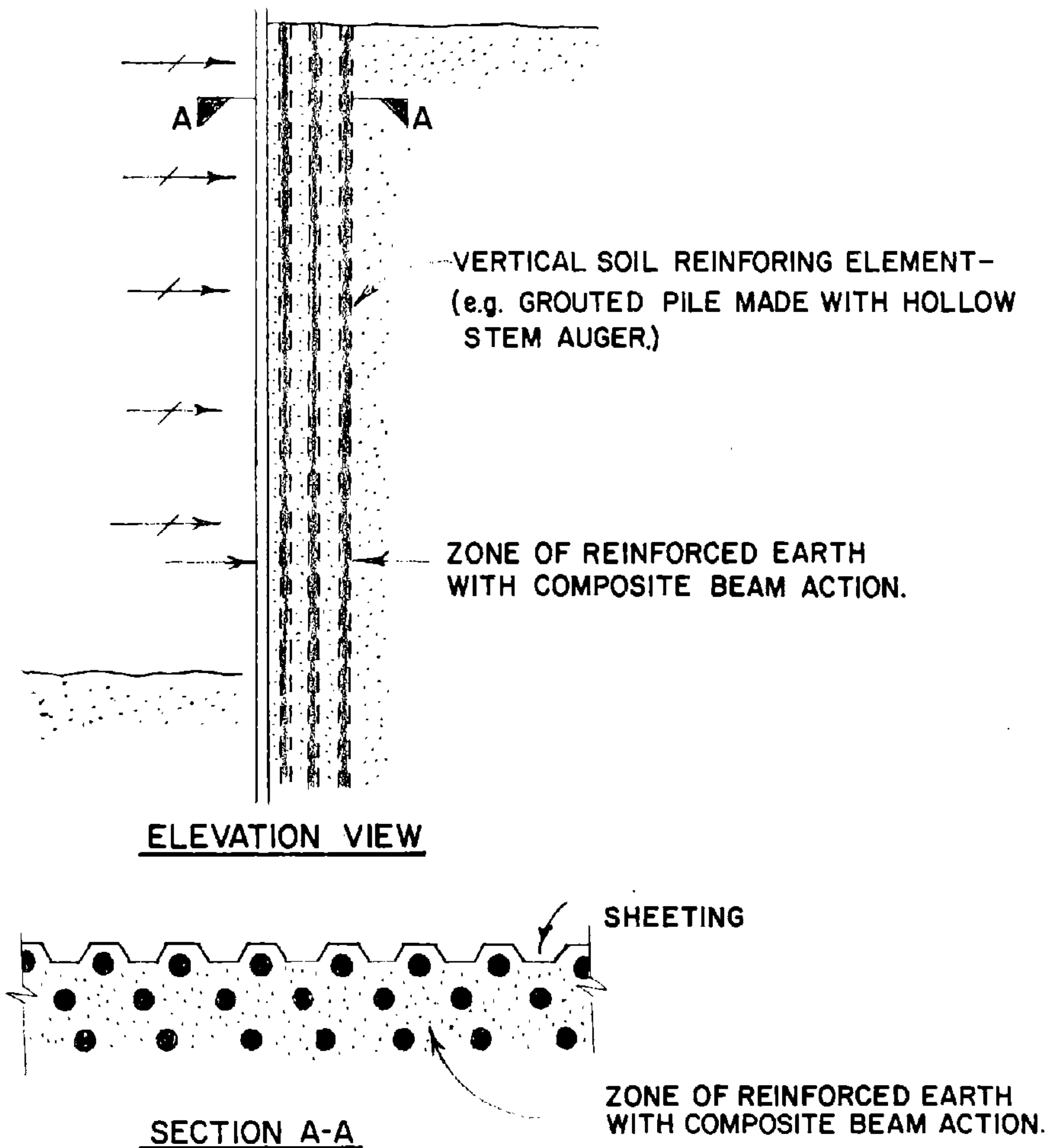


Figure 8. Vertical soil reinforcement.

would be the 4-inch to 6-inch diameter type reinforced with a single bar. Conventionally, these are made with percussion drilling equipment by first advancing casing and then pressure grouting as the casing is withdrawn.

4.30 PRESSURIZED WELLS IN COHESIVE SOIL

4.31 Background

As noted by Goldberg, et al (1976b) an excavation made in cohesive soils will lead to a seepage flow pattern toward the excavation. This flow pattern is independent of soil permeability and is virtually unaffected by the presence of a steel sheet pile wall.

The changes in hydrostatic stress associated with seepage leads to a time-dependent equivalent change in effective stress and consolidation of the soil. When the excavation is underlain by deep deposits of soft cohesive soils, stress changes occurring within the soil mass will lead to settlements outside of the excavation.

It is self-evident that a means to control changes in piezometric level would have a mitigating effect on settlement. Recharge wells may be used to maintain water levels in pervious strata. However, to the writers' knowledge attempts to maintain the piezometric level in clay or other impervious soils by means of wells have not been done.

The proposed technique, shown in Figure 9 is to install a line of wells for the full depth of the soft clay deposit. A purpose of the wells is to control the hydraulic head and seepage boundary conditions so that effective stress changes (and consolidation) will be kept to a minimum. Unlike recharge wells in pervious soils in which flow is large, the flow from wells in clay will actually be negligible because the soil is so impervious. The controlling parameter is to maintain sufficient piezometric pressure in a line of closely spaced wells rather than to diffuse a required volume back into the ground.

Figures 10 and 11 give the mathematical rationale for well locations and spacing. The equations presented therein are based upon relationships given for head loss in the vicinity of pumping wells as summarized by Fruco and Associates (1966).

The following summarizes the main points:

1. Suggested criteria for well size, locations, and head in wells is to maintain the piezometric level essentially at the normal ground water level.

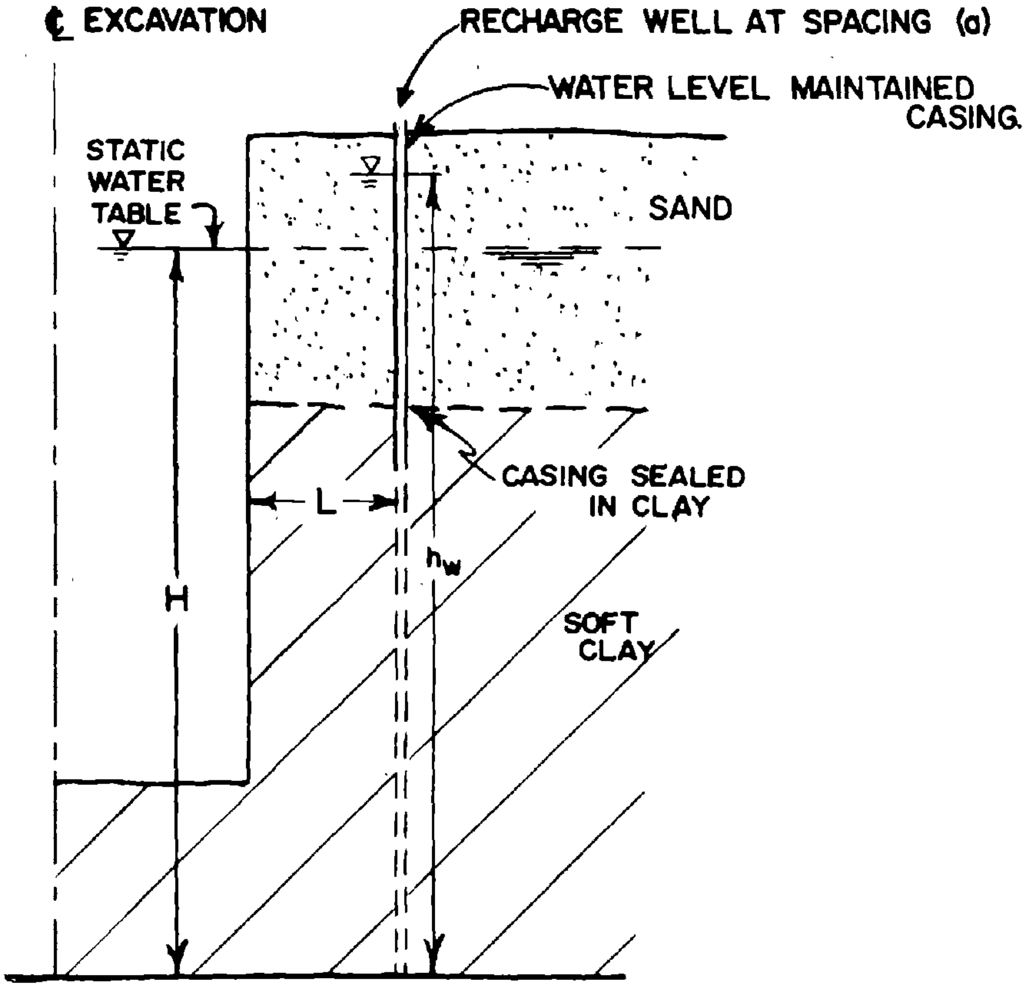
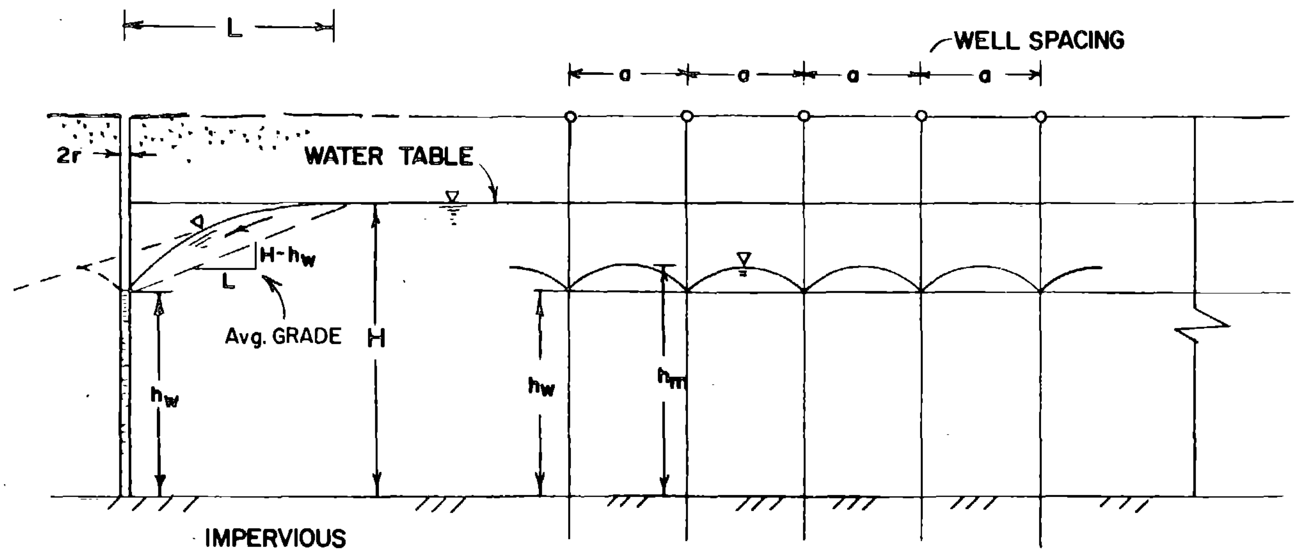


Figure 9. Recharge well to pressurize clay.

DRAWDOWN EQUATIONS



DRAW DOWN FLOW

By Dupuit:

(1) $q = k\,i\,A = \text{Flow}$

$$= k\left(\frac{H-h_w}{L}\right) \times \left(\frac{H+h_w}{2}\right)$$

$$= k.\left(\frac{H^2 - h_w^2}{2L}\right) \text{ per unit length}$$

(2) $Q \sim q.a = k\,a\left(\frac{H^2 - h_w^2}{2L}\right)$ per well

MOUNDING BETWEEN WELLS

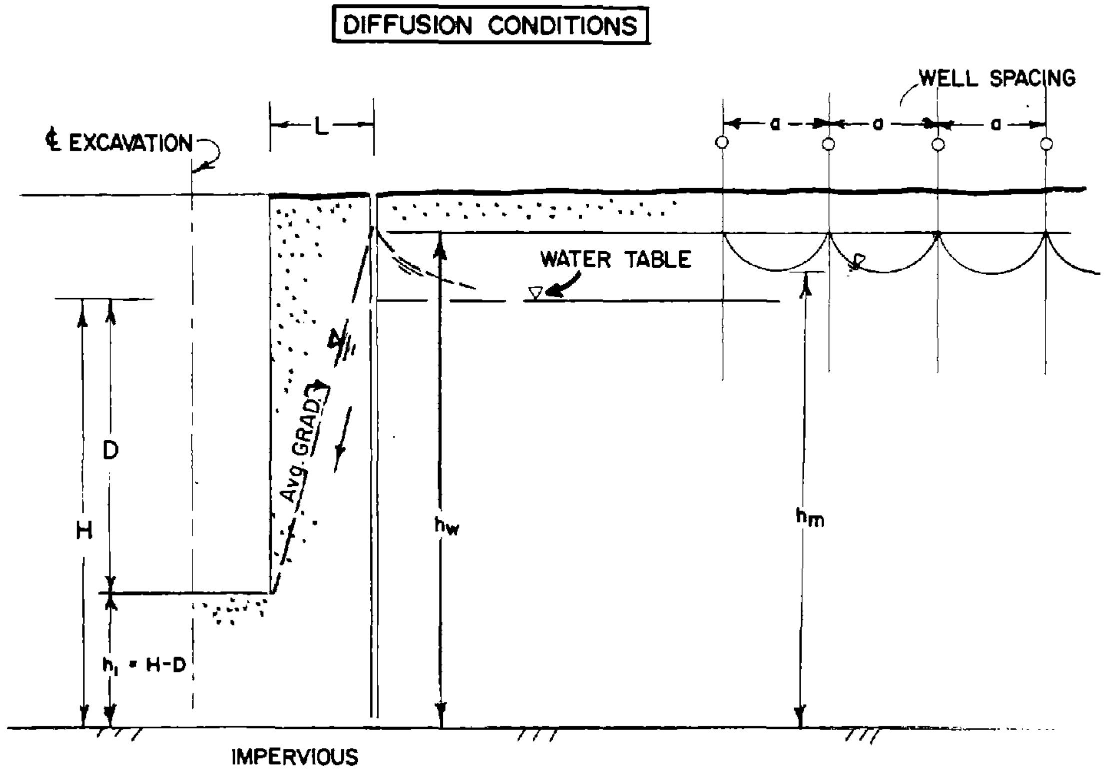
(3) $h_m^2 - h_w^2 = \frac{Q}{\pi k} \; \ln. \; \frac{a}{\pi r}$

Substitute Equation 2 for Q

(4) $h_m^2 - h_w^2 = \frac{a}{2\pi L}\left(.\ln \frac{a}{\pi r}.\right)\left(H^2 - h_w^2\right)$

$$= \beta\left(H^2 - h_w^2\right)$$

Figure 10. Mathematical relationships for drawdown.



By Dupuit

(5) Flow = $q = k\,i\,A$

$$= k\left(\frac{h_w - h_1}{L}\right)\left(\frac{h_w + h_1}{2}\right)$$

$$= \frac{k}{2L} \cdot \left(h_w^2 - h_1^2\right) \text{ per unit length}$$

(6) $Q \sim q \cdot a = \frac{ka}{2L}\left(h_w^2 - h_1^2\right)$ per well

(7) By comparison of Equation (4) with with Equation (6):

(8) $h_w^2 - h_m^2 \doteq \left(h_w^2 - h_1^2\right) \cdot \beta$

Values of β

$$\beta = \frac{a}{2\pi L} \cdot \ln \frac{a}{\pi r}$$

a/r \ a/L	0.5	1.0	2.0
10	.090	.180	.270
20	.147	.294	.441
30	.179	.360	.540
40	.202	.404	.604

Example:

H = 90'; Maintain well level at 10' above water table - ∴ h_w = 100'. Excavate 50' below water table; therefore D= 50' and h_1 = H-D = 40'

Use 12' ø wells at 20' back from excavation

r = 6" = 0.5', L = 20'

Try 15' spacing, a = 15'

$a/L = 15/20 = 0.75'$ $a/r = 15/0.5 = 30$

From table, β = 0.270 (by interpolation)

Find water level at saddle point between wells

Equation 8: $h_w^2 - h_m^2 = \left(h_w^2 - h_1^2\right)\beta$

$h_m^2 = (100)^2 - (100^2 - 40^2)(.270) = 7730$

$h_m = 87.9$

$\Delta h = h_w - h_m = 100 - 87.9 = 12.1'$ or 2.1' below water table- say o.k.

Figure 11. Mathematical relationships for well pressurization.

2. The controlling factor regarding water level is the saddle point midway between wells. The equation used to determine the piezometric level at the saddle point is shown by equation 8, Figure 11

$$h_w^2 - h_m^2 = (h_w^2 - h_1^2) \left(\frac{a}{2\pi L} \ln \frac{a}{\pi r} \right)$$

in which terms are defined in Figures 10 and 11 as follows:

h_w = head at well

h_m = head midway between wells

h_1 = head at base of excavation

a = well spacing

r = well radius

L = distance from excavation to line of wells

3. Note that the equation is independent of soil permeability and flow. Rather, the equation is a function only of geometry.

4. The head (h_m) is extremely sensitive to the distance L. As L becomes small, the average gradient from the wells to the excavation increased very rapidly. Under these conditions the equation shows clearly that the well spacing, (a), must be reduced to maintain the required head (h_m).

5. Conversely, if the wells can be stationed far from the excavation (large L), then the well spacing (a), can be increased, while still maintaining the required head h_m.

6. With pervious deposits overlying the clay, the well casing ought to be sealed off in the clay, to prevent flow into the pervious soil, and to maintain the required head in the clay.

5.00 SUMMARY AND NEEDS FOR FUTURE RESEARCH

5.10 EMPHASIS

The main purpose of the techniques and concepts advanced within this section is to contribute to the mitigation of damages caused by displacements occurring outside the excavation area. The objective of course is to improve performance, reduce cost, and to eliminate underpinning where possible.

The rationale for these concepts stems largely from the effort undertaken during preparation of the companion reports (Goldberg, Jaworski, and Gordon, 1976a, 1976b, and 1976c). This brought into focus a number of factors inherent in various methods and/or soil conditions which contribute to displacements in the adjacent ground.

5.20 EXISTING CONSTRUCTION TECHNIQUES

Section 2.00 presents innovations associated with existing construction techniques.

5.30 ANALYTICAL AND PREDICTIVE TECHNIQUES

The two main ideas advanced in Section 3.00 are:

a) Internal Stability (Cofferdam) Analysis of Tieback Walls

The relationships demonstrate that the stability of a tieback wall is a function of the ratio $\frac{B}{H}$ in which B is the effective width of the tieback zone (that is, the earth embodied by tiebacks) and H is the depth of the excavation.

A second relationship was developed which indicates that the lateral displacement is a function of the quantity $\frac{H^3}{GB}$ in which H and B are as described above and G is the shear modulus of the soil.

It should be recognized that these mathematical relationships are for conceptual purposes and are not intended to be rigorous in their application. However, they do provide a beginning framework for further investigation leading to semi-empirical analytical techniques.

It is recommended that laboratory research programs using models to investigate the concepts advanced herein be instituted.

b) Lateral Displacements in Overconsolidated Soils

Both field evidence and theory indicates that the relief of the high lateral pressures inherent in overconsolidated clays leads to a time-dependent lateral creep. The controlling parameters in this process are believed to be the initial confining pressure ($\bar{\sigma}_{ho}$), final confining pressure ($\bar{\sigma}_{hf}$), lateral extent of zone behind the tied-back plug subject to stress relief (T), and the primary and secondary swell characteristics of the soil (C_{hs} and $C_{\alpha h}$, respectively). More fundamentally, soil moduli (bulk and shear) and their changes with time are significant governing parameters.

Further research should focus on the development of the simple one-dimensional primary and secondary swell mechanisms by accumulation and analysis of tied-back performance data. This approach may offer some relatively immediate improvements.

On the other hand, the major thrust of any extensive long term research efforts should also focus on modification of more sophisticated techniques for analysis of time-dependent soil and foundation behavior. This will be a considerable effort but may offer greater long term benefits. One possible procedure would be to measure the stress relief and time-dependent change in soil properties in laboratory tests. Such tests might include overconsolidation of specimens of cohesive soil followed by rebound to zero vertical load and measurement of the subsequent stress changes and rebound with time. Plain strain testing which more closely simulates in situ stress systems might be incorporated. The soil properties measured in such tests would then be incorporated into geometry and stress conditions of a tied-back system.

c) Displacement Prediction

More data are needed on the effect of wall stiffness (EI/L^4) and stability number ($\gamma H/S_u$) on displacements when excavations are made in cohesive soil. Goldberg, et al (1976b) demonstrated from empirical data and finite element analyses that definite trends are apparent. A coordinated effort to gather and evaluate data from case histories should be undertaken.

5.40 TECHNIQUES TO REDUCE DISPLACEMENTS

a) Combination of Tiebacks and Internal Bracing

Three general schemes discussed in Section 4.00 combine internal bracing and tiebacks within a common excavation. In all cases, the objective is to take advantage of those characteristics of tiebacks and of internal bracing which serve to minimize deformations.

In two of the schemes, tiebacks are installed in the normal manner and designed to carry their share of the full lateral pressure caused by earth, water, and surcharge. In the third scheme, total lateral pressure is taken by the internal bracing. In this case, the tiebacks reinforce the earth mass and provide intermediate levels of support between levels of internal bracing. The load in these tiebacks is significantly less than normal tieback loads. The most promising research opportunities lie in actual prototype test sections, in which tieback equipment can be easily mobilized and will cause minimal, if any, interference with the normal construction procedure. Such a prototype research effort will provide a basis for comparison with control sections built in the normal manner by conventional means.

b) Ice Wall as Earth Reinforcement

The objective of this technique is to reduce the so-called "uncontrolled deformations" which occur below the last place strut level when excavations are made in weak, cohesive soil. Experience has shown that a very stiff wall (concrete diaphragm wall, for example) will dramatically reduce the displacements which might otherwise take place in a relatively flexible steel sheet pile wall.

The purpose of the ice wall is to temporarily freeze the soil below the excavation in order to make it stiffer and thereby reduce the so-called "uncontrolled deformations". After the excavation is complete and the lateral support system has been installed, then the freezing plant can be shut off and the soil allowed to thaw.

It is believed that this technique also lends itself to research investigation by means of prototype test sections.

c) Vertical Soil Reinforcing

The principle is much the same as that of the Reticulated Wall, in that reinforcing elements (most likely augered reinforced concrete piles) would be inserted into the soil at a predetermined spacing. As with the ice wall, the objective is to increase the stiffness (EI) of the wall to reduce displacements. Because of the many uncertainties concerning appropriate spacing, orientation, and indeed the potential economics of such a procedure, it is believed that the first step in a future research effort should be a laboratory program using models.

d) Pressurized Wells in Cohesive Soils

Experience has demonstrated that consolidation settlements caused by excavations in deep deposits of soft, compressible cohesive soils

soils is frequently unavoidable, largely because it is difficult to prevent lowering of the piezometric level within the surrounding medium. Diaphragm walls offer some promise because they are effectively impervious. Unless a seal can be achieved, even with diaphragm walls, a flow pattern develops to the base of the excavation which results in changes in piezometric head, changes in effective stress, and consolidation.

Interlocked steel sheeting has little or no effect in preventing changes in piezometric levels because the permeability of the sheeting is high relative to the permeability of the soil. On the other hand, in granular soils this is not true because the equivalent permeability of the sheeting is low relative to the permeability of the soil.

The need to prevent consolidation settlements has led to the idea of preventing changes in effective stress within the soil. Fundamental to the whole process is the concept that flow from these wells is effectively negligible. Unlike recharge wells in pervious media which must diffuse large volume of water back into the ground, the primary function of pressurized wells is to maintain head levels within predetermined criteria to control seepage boundary conditions. With regard to future research, this technique is also believed to be adaptable to a prototype test section in which piezometric levels and displacements could be monitored and compared with control sections constructed in the normal manner. Because pore pressure changes (and consolidation) is time-dependent, it is fundamental that sufficient time be allowed to determine whether or not the wells are effective.

6.00 PROPOSED TEST SECTIONS

6.10 TECHNIQUES TO BE STUDIED

Prototype test sections are believed appropriate for the following techniques:

a) Combination of tiebacks and internal bracing
(See Section 4.13 and Section 4.22)

b) Ice wall as earth reinforcement
(See Section 4.23)

c) Pressurized wells (See Section 4.30)

Objectives are first to determine the improvement that these techniques offer in regard to displacements, and second, to study their effect on the support system load.

6.20 GENERAL REQUIREMENTS

a) No major structure nearby which may impose special requirements for performance or which may impose surcharge loading.

b) Soil:

1) Tieback and internal bracing combinations shall be in competent soils (sand, sand and gravel, cohesive sand, lean sandy clay). Avoid soft to medium clay and potentially expansive overconsolidated clay.

2) Ice walls and pressurized wells should be used in deep soft to medium clay. The bottom of the sheeting should terminate in soft to medium clay.

3) Provide basic soil data concerning index properties, strength of cohesive soil, and strength and creep characteristics of frozen soil.

c) Depth of cut should be at least 35 feet for the case of soft to medium clay and at least 55 feet for the case of competent soils.

d) Wall types should be interlocked steel sheet piling in soft clay and either soldier piles with lagging or interlocked steel sheet piling in competent soils. Concrete diaphragm walls shall not be used.

e) With a soldier pile wall, exercise extreme care to avoid ground loss. (For example: predrain soil before exposing face and install piezometers to verify; avoid running ground; backpack behind lagging; insure surface drainage away from cut; etc.).

6.30 CONTROL AND EXPERIMENTAL SECTIONS

The recommended approach is to monitor performance at a control section and at a companion experimental section. The control section is constructed in the usual way, except that the many variables associated with the construction method are isolated by the imposition of strict compliance with pre-established criteria - for example, preloading of bracing, sequence of operations, limitation of overcut below support levels, techniques for installing lagging, etc. All of these standards are also applied to the experimental section.

Ideally then, the objective is to have identical soil conditions and identical construction procedures (except for the technique being studied) at the control section and at the companion experimental section. The effect is to isolate all variables other than the variable of the technique.

Figure 12 shows the general layout of control and experimental sections. The main features are:

a) The control section (B') could be flanked by one or two experimental sections provided soil conditions are constant. For example, one experimental section (B'') could be tiebacks as earth reinforcement (Section 4.22) and other experimental section (B''') could be tiebacks and struts sharing full load (Section 4.13).

b) The distance, A, between control and experimental sections should be at least 3H in all cases except for the pressurized well scheme. In this case, the distance, A, should be 5H or more.

c) Displacement observation points, both vertical and horizontal, are common to all techniques as located on Figure 12 within the monitoring section. Also, an optical survey should be made on the top and the face of the wall.

d) Everything is symmetrical and redundant about the centerline of the cut.

e) Measure load at tiebacks and on struts at two instrumented stations within Zone C, the monitoring section, as shown in Figure 12. Each instrumented station shall be near the outer extremity of Zone C. Preferably, there shall be one noninstrumented station separating the two instrumented stations.

f) Instrumented struts should bear symmetrically on a continuous (full moment capacity) section of wale.

g) Measure load on all tiebacks within a distance equal to the strut spacing and centered on the instrumented station. For example, say we have two tiebacks to each strut. Then, loads should be measured on each of the two tiebacks flanking each instrumented strut.

h) Special requirements for piezometric monitoring in clay are shown in Figure 13. The sketch is drawn for the "well scheme", but it applies with minor modifications to the ice wall. Sufficient observation wells and/or piezometers must be installed to define ground water conditions.

i) It will be essential to determine the thickness and degree of continuity of the ice wall.

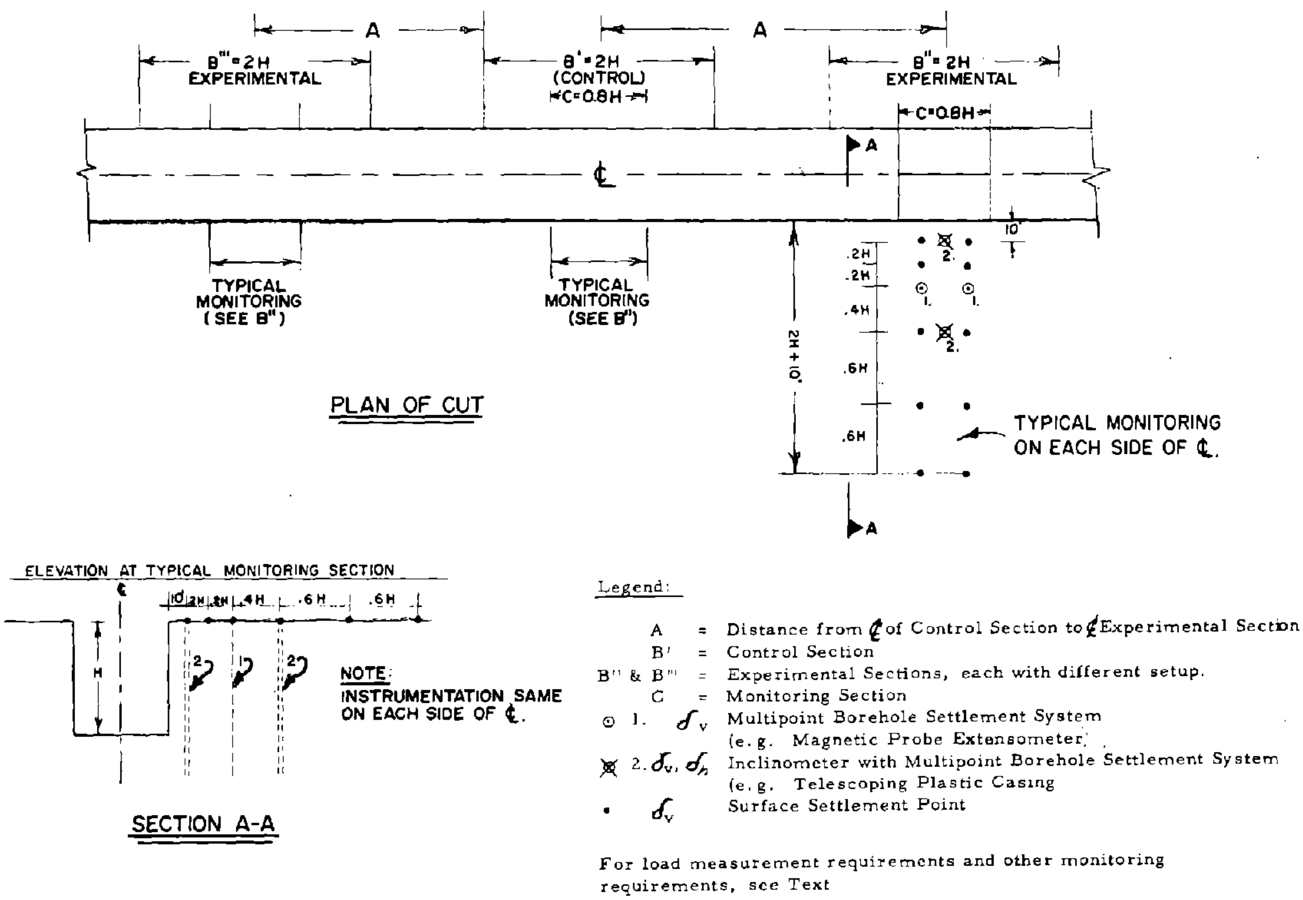
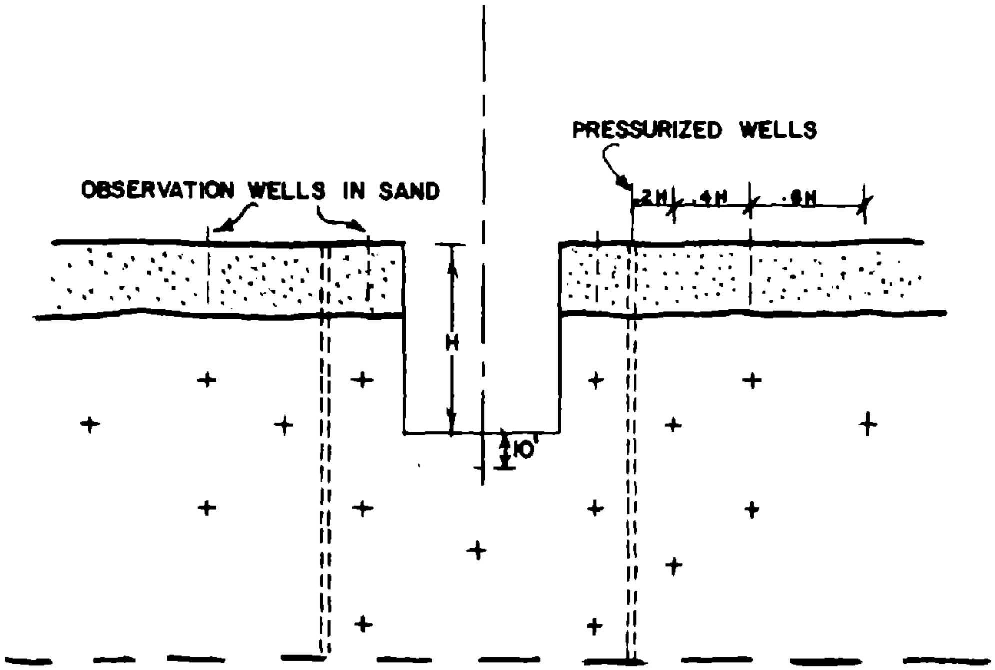


Figure 12. Control and experimental sections.



LEGEND:

+ INDICATES PIEZOMETER TIP IN VERTICAL PLANE MIDWAY BETWEEN WELLS.

Figure 13. Piezometer locations in soft clay.

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