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FACTORS TO BE CONSIDERED IN THE DESIGN OF BACKFILL
FOR LARGE DIAMETER FLEXIBLE METAL CONDUITS

By

David C. Cowherd, M.S., P.E.

It is a well understood fact that large diameter, flexible, corregated metal conduits depend for their support to a great extent on the surrounding backfill soil. In order to analyze such a structure it is necessary to consider the interaction between the pipe and the side fill soil around the pipe as a unit. The pressure generated on the pipe by fill or live loads is ultimately transmitted to the soil at the sides of the pipe and the movement of the pipe will eventually be the amount permitted by the backfill around the pipe. It is, therefore, of extreme importance that the backfill around the pipe be properly designed so as to withstand the loads generated by the pipe without allowing movement which will cause failure of the pipe. In general, in high fill situations, the conduit itself would withstand but a small percentage of the load placed on it by the fill material, therefore, the backfill becomes of prime importance.

In general, if the backfill around the pipe is of a granular nature and is compacted to a reasonable density (90% of the Standard Proctor dry unit weight) there is very little movement of the pipe. Since granular backfill does not generally constitute a problem and does offer sufficient lateral restraint at reasonable percentages of Standard Proctor, granular backfill is not considered in this paper. Plastic soil backfills however can constitute problems even at high percentages of compaction in some instances. The heretofore assumed safe degree of compaction of 85% of the Standard Proctor dry unit weight may not be sufficient in many cases where cohesive backfill is used. It is the opinion of this writer that whenever large diameter structures are considered, a testing program should be instituted to determine the required degree of compaction of clay backfills. Present practice is to merely assume that 85% or 90% of Standard Proctor dry unit weight will provide a satisfactory backfill and prevent excessive movement of the pipe. In terms of the Soil Modulus (E' in the Iowa formula) it is assumed that this degree of compaction will provide an E' of 700 psi or greater. Some observations by the writer on actual installations have indicated that this is not necessarily true and that some pre-installation testing of the proposed backfill soil should be provided before writing the specifications. The purpose of this paper is to outline the observations made on two relatively large diameter pipes which exhibited considerable movement and to make certain suggestions as to how the proposed backfill for a corregated metal pipe should be tested to determine the supporting power and probable deflection of a pipe under a given loading situation.

In order to design the testing program it is necessary to know some basic facts about the strength of clay-type materials under various different degrees of compaction, moisture contents, etc. It is also a well known fact that samples of clay soil compacted to a given percentage of Proctor compaction at moisture contents slightly dry of the optimum moisture content will have a higher strength than samples of the same soil compacted to the exact same degree of compaction but compacted at a moisture content considerably wet of the optimum moisture content. It is necessary to well understand this fact in order to properly determine the behavior of clay backfill around a pipe at given densities. It is also necessary to understand the moisture limitations on given clay-type backfills. For the purposes of this paper the following discussion of strength in relation to both degree of compaction and moisture content is offered. This discussion should be understood in order to properly evaluate the behavior of backfill around flexible conduits.

The strength of a clay soil can be said to vary with two major parameters, i.e., unit weight and moisture content. There must be a specification for each if the proper strength is to be built into the soil. For example, a sample of clay soil with a maximum Standard Proctor dry unit weight of around 120 pcf and an optimum moisture content of 15% will have a strength when compacted to 90% of Standard Proctor dry unit weight and a moisture content of 10% that is considerably higher than the same soil compacted to the same 90% but at a moisture content of 20% (see Figure 1). It is not sufficient, therefore, to merely specify a degree of compaction in dealing with clay backfills. This criteria applies to samples compacted at a given moisture content and then sheared at that same moisture content. It could reasonably be assumed, however, that backfill around flexible conduits which are carrying water will at sometime become saturated. In the testing procedure for the backfill soil it should be considered that the backfill will at sometime become saturated in a water carrying situation. The soil should, therefore, be tested in a saturated condition. Again, it should be observed that the strength of a soil compacted at near optimum moisture content then saturated and sheared will be considerably higher than a sample compacted to the same degree of compaction at a saturation moisture content and sheared. For example, a sample of clay soil compacted to 90% of the maximum Standard Proctor dry unit weight at optimum moisture content, then saturated and sheared, will have a much higher strength than the same sample of soil compacted to 90% at the saturation moisture content. This is true even though the final moisture content and percentage compaction when sheared will be exactly the same. This strength phenomenon will be used in establishing criteria for the design of backfill around flexible culverts. It is important to understand this phenomenon as it affects the type testing program which should be used in order to determine the requirements for backfill.

This paper will outline some basic criteria which have been evolved as a result of both a search of the literature, and as a result of the observations of two (2) large diameter pipes which to some degree exhibited excessive amounts of movement. Both of these large diameter pipes had a clay type backfill. The clay was a relatively heavy clay with reasonably high Atterberg limits and somewhat of a swelling potential. Each of these projects along with some of the observations made relative to these projects will be discussed below.

The writer was associated with the determination of why a relatively large diameter flexible pipe moved excessively in about 1966. This pipe was ten (10) feet in diameter and the clay soils around the pipe were found to have the index properties as indicated in Figure 2. It can be seen from this figure that this was a relatively heavy clay.

In order to determine the cause of the movement of the pipe, borings were made around the pipe to outline the relative degrees of compaction, relative moisture contents, and the degree of compressibility of the backfill around the pipe. A table outlining this information is included in this paper as Figure 3. It can be observed that the actual backfill around this pipe was at a relatively low degree of compaction and also had a relatively high moisture content in comparison to the optimum moisture content as determined by the Standard Proctor test. These data indicated the pipe moved because of inadequate compaction at high moisture contents.

It was at that time decided to undertake a program of testing to determine what degree of compaction would have been necessary for the backfill and whether or not the pipe would have exhibited the large amounts of deflection if a higher degree of compaction or lower moisture contents had been achieved. In accordance with this purpose, samples of the soil were compacted in the laboratory to various different unit weights and moisture contents and were then saturated. A confined compression test was performed to determine the actual stress-strain characteristics under various different soil conditions. It was then decided to try to use these data to predict the amount of side movement of the fill around the pipe at various different degrees of compaction and moisture content. As a result of this, the following testing program was decided upon as a method for determining the relative amount of movement. It was decided that the proper way to evaluate whether the backfill would have functioned adequately if properly compacted, would be to compact a sample of the soil at near optimum moisture content to a dry unit weight of 90% of Standard Proctor, then to saturate this sample and to shear the sample in a saturated condition. This was then done for various different degrees of compaction ranging between 85% and 100% of Standard Proctor.

In order to determine the difference in strength of samples prepared at various different degrees of compaction and moisture content, several samples were compacted at optimum moisture content and sheared at optimum moisture content; several were prepared at optimum moisture content, saturated and then sheared; and yet other samples were prepared at saturation moisture contents and sheared at these moisture contents.

The stress-strain curves prepared in this manner are shown in Figure 4 included in this paper. These curves indicate that for a given degree of compaction, the strength of samples molded at optimum moisture content and then saturated is about 3 1/2 to 4 times the strength of samples at the same density and the same final moisture content but prepared at a saturation moisture content. The strength of this material when not saturated but sheared at optimum moisture content is in the neighborhood of 10% higher than the strength when saturated after being compacted at optimum moisture content. In other words, the sample only loses about 10% of its strength when compacted at optimum moisture content and then later on becomes saturated; whereas if it is compacted to a given density at a saturation moisture content it is only about 1/4 as strong. To this point it had been decided that the proper method of test for backfill was to compact a sample of soil to the specified field dry unit weight at optimum moisture content, to then saturate the sample and shear it in a confined compression test with the confining pressure equal to the expected lateral pressure in the field. It was then decided to try to convert these stress-strain curves into expected pipe movement curves. This was done by generating what might be called pipe-soil interaction curves. The method in which these curves were generated was to consider the fill at the sides of the pipe as a soil column and to apply the stress generated by the pipe onto the fill to this soil column. It was first necessary to prepare an influence chart so that the degree of stress in the soil at any given distance away from the pipe could be determined. It was decided in this particular analysis to utilize a soil column equal to 2 1/2 pipe diameters as a criteria. The stress placed on the soil by the pipe was assumed to be trapezoidal in nature and an influence chart for these conditions was prepared. This chart is shown in Figure 5. The stress-strain curves and the soil influence graph were then used to generate the amount of movement in inches in any given area. This was done, for instance, by considering the stress in the first 2 foot section of the soil adjacent to the pipe. A stress was picked, the influence value for the first 2 feet of fill was determined and the strain for the first 2 foot increment of soil was determined from the stress-strain curve. This was done for each

2 foot increment throughout the entire 2 1/2 pipe diameters. The total strain in inches was then found by adding the strain in each individual 2 foot increment. In this manner a stress vs. a total movement was generated. The point generated indicates the amount of movement of one side of the pipe under a given horizontal loading. This can, if desirable, be translated into vertical movement and vertical stresses. This can be done by considering the stress distribution on the pipe and expressing the equivalent vertical loading for a given horizontal loading. The total vertical movement will be approximately twice the one side horizontal movement. A curve of vertical stress vs. vertical movement can be obtained in this manner. It is possible using the curves generated in this manner to calculate the amount of movement in inches at the side of the pipe or total vertical movement due to a given load on the pipe. With this curve it would be possible to determine fill height tables and deflections for any given degree of compaction.

These soil-pipe interaction curves are shown in Figure 6 of this paper as horizontal stress in psi versus one-side horizontal movement for various degrees of compaction and molding moisture contents. For this particular soil it can be shown that this material compacted to as little as 85% of Standard Proctor dry unit weight with a molding moisture near the optimum moisture content, then saturated and sheared would not produce a failure condition. This same material, however, if compacted to a dry unit weight of around 90% at a saturation moisture content would produce a large amount of movement of the pipe at relatively low fill loads. The value of specifying that the material be compacted near optimum moisture content is therefore readily apparent.

This same type of information was generated for a second project which also exhibited relatively large movements. This project consisted of a pipe arch approximately 28 feet in diameter with a rise of about 13 feet 10 inches. This arch was backfilled with a relatively heavy clay the properties of which are shown in Figure 7 of this paper. The backfill varied from a soil with a Standard Proctor maximum unit weight of around 107 pcf at an optimum moisture content of around 17% to a soil with a maximum Standard Proctor dry unit weight of around 123 pcf at an optimum moisture content of around 12%. It can be assumed that the actual backfill varied between these limits. The project specifications called for 90% of Standard Proctor dry unit weight and the backfill procedures were adequate. The backfill was compacted immediately adjacent to the pipe with hand compactors and with a small sheepsfoot roller away from the pipe. The pipe arch was placed in a large trench excavated for that purpose and approximately 10 to 15 feet of fill was to be placed over the arch. The arch was supported on a footing with a concrete bottom. Shortly after backfilling around the arch it was observed that large amounts of movement were occurring.

A program for measuring the amount of movement was then instituted and measurements of the movement were made over a relatively long period of time. Typical curves of this movement are shown in Figure 8 of this paper. It can be observed from these curves that considerable movement was occurring even after long periods of time. It was decided at this time to obtain samples of the soil and to test them similarly to the program previously outlined to determine the cause of the excessive movement. Samples of the soil were then obtained and confined compression tests were performed on various samples of the soil. These tests were performed at various different moisture contents, unit weights, and degrees of compaction as for the previously mentioned project. Basically, samples were compacted at 90% and 95% of the maximum Standard Proctor dry unit weight at optimum moisture content, saturated and then sheared. Other samples were then compacted at 90% and 95% at saturation moisture contents and sheared. These samples were sheared at a lateral pressure projected to be equal to the lateral pressure in the field. The stress-strain curves generated in this manner are shown in Figure 9. It can easily be observed from these curves that the strength of the soil compacted at optimum moisture content, saturated and then sheared is considerably higher than the strength of the samples compacted at saturation moisture contents to the same degree of compaction and then sheared.

Once the stress-strain curves for these soils had been determined, a soil influence chart was prepared. For this particular pipe which had concrete buttresses at the top, it was considered sufficient to assume the stress distribution the same as a large spread footing. The length of the soil column was taken to be the distance from the side of the pipe to the walls of the excavation. This influence chart is shown in Figure 10. Using this information and the stress-strain curves previously discussed, it was again possible to determine total lateral movement for a given stress level. Again, this could be translated to vertical movement vs. fill or vertical load. The curves generated in this manner are shown in Figure 11. These curves constitute the pipe-soil interaction curves for various degrees of compaction and moisture contents and are plotted in total vertical movement vs. vertical stress. From these curves it is possible to predict total amount of movement under various loading situations.

In this instance it was possible to compare the calculated amounts of movement to the actual movement as records were made of the actual movement. The only difficulty at this point was trying to correlate percentage of compaction actually achieved in the field vs. the laboratory compaction values. However, utilizing the field compaction data for various different stations, it was possible to make a comparison of projected values and actual values. The projected movements for various different degrees of compactions and moisture contents and the actual observed settlements are shown in Figure 12. It can be seen from this figure that these values agree quite well. It is therefore apparent that this procedure does provide a good method for projecting the movement of flexible pipes which is within the scope of operations of most soils laboratories.

One interesting item which came to light in the consideration of these pipes is that there is a phenomenon which is identified here as an initial set of relatively large pipes which may include an initial sitdown of the pipe itself as it takes up slack in bolts, plates, etc. This movement may also include some initial elastic shortening in the material of the pipe itself as well as some initial movement of the earth. This amount of movement is not indicated by laboratory testing and must be considered to be independent of laboratory testing techniques. This may be a relatively large movement for structures with backfill of clay, however, it is felt that this movement is small in comparison to the total movement.

Based on the information obtained in the above manner, several conclusions can be drawn that should help to more accurately evaluate the performance of backfill around a large diameter pipe. The heretofore procedure of assuming that if the backfill is compacted to a given percent of Proctor dry unit weight is not considered adequate, and at least any large diameter pipe which is to be installed with clay type backfills should be investigated with test borings previous to installation, and the backfill procedures and specifications should be written by a competent soils engineer who has performed adequate testing of the proposed backfill. If this is not done, even though a good effort is made to compact the soil as was done in one of the cases above, large movements and even possible collapse of the pipe can occur. It is, therefore, imperative to properly test the backfill material and to properly design the specifications for the backfill material. It should be realized that the stress-strain relationships will be different for different types of soil material and that a standard set of pipe interaction curves based on one type of material cannot be generated but it is necessary to test each individual case. The relative effect of moisture content of the soils will vary from soil to soil. The effect of molding moisture content as well as eventual degree of saturation will also vary from soil to soil. Each case should, therefore, be investigated individually and proper design procedures should be established.

The following is recommended as a test procedure or program preparatory to writing specifications for backfill for a large diameter flexible pipe with cohesive backfill. It is necessary in order to properly evaluate the system to know the following information. The nature of the pipe (whether round, arch, eliptical, etc.) must be determined since the stress applied to the soil is different for different kinds of pipes. The stress relationship of the pipe to the soil should be defined and established so that a reasonable estimate of stress in the soil can be made. Other physical criteria must be known such as the nature of the pipe and whether it is in a trench situation or a projected situation. A knowledge of the height of fill to go over the pipe must be determined. The nature of the material upon which the pipe will be placed must also be determined. Knowing this information, samples of the soil to be used as backfill should be obtained. It is recommended that these samples be compacted, some to 90% Standard Proctor at optimum moisture content, some to 95% Standard

Proctor at optimum moisture content. As a precautionary method these samples should then be saturated before testing. It is our opinion that the proper test to perform in this case is a confined compression test where the confining pressure is equal to the expected confining pressure in the field. This requires an estimate of relative degrees of coefficients of vertical pressure to horizontal pressure so that a reasonable determination of the lateral pressure to apply in the triaxial test can be made. It is then necessary to determine the stress-strain characteristics of the soil through this confined compression test.

With the data obtained in the above manner a curve relating vertical stress on the pipe to lateral movement of the sides of the pipe or vertical movement of the pipe can be determined. From these curves it is possible to determine the degree of compaction which should be established in the specifications. With enough data the limitations on moisture content in the field can also be determined. If the pipe is a positive projection pipe, it is recommended that 2 1/2 times the pipe diameter be considered as the column of soil which will move. Obviously for a complete trench pipe, the amount of backfill between the pipe and the edge of the trench should be considered as that amount of soil which will move under the load of the pipe. For the curves generated in this manner, it is possible to predict amounts of movement based on the actual fill height to be placed over the pipe.

Several things are to be noted about this criteria. It should first of all be understood that the above criteria has been developed independent of pipe stiffness, material, etc. The basis for this determination is that if the fill height is sufficiently high the pipe itself would fail under the height of the fill, therefore, the limiting criteria becomes not so much the strength of the pipe but the movement of the backfill around the pipe. It is, therefore, considered valid to assume that the pipe itself without any lateral support would fail and that the pipe will move the amount that the backfill beside the pipe will allow it to move. Within reasonable limits the above criteria apply independent of type of pipe as far as movement goes. This is not to say that the type of pipe is independent of the load on it because the wall stresses and the compressive stresses all cases.

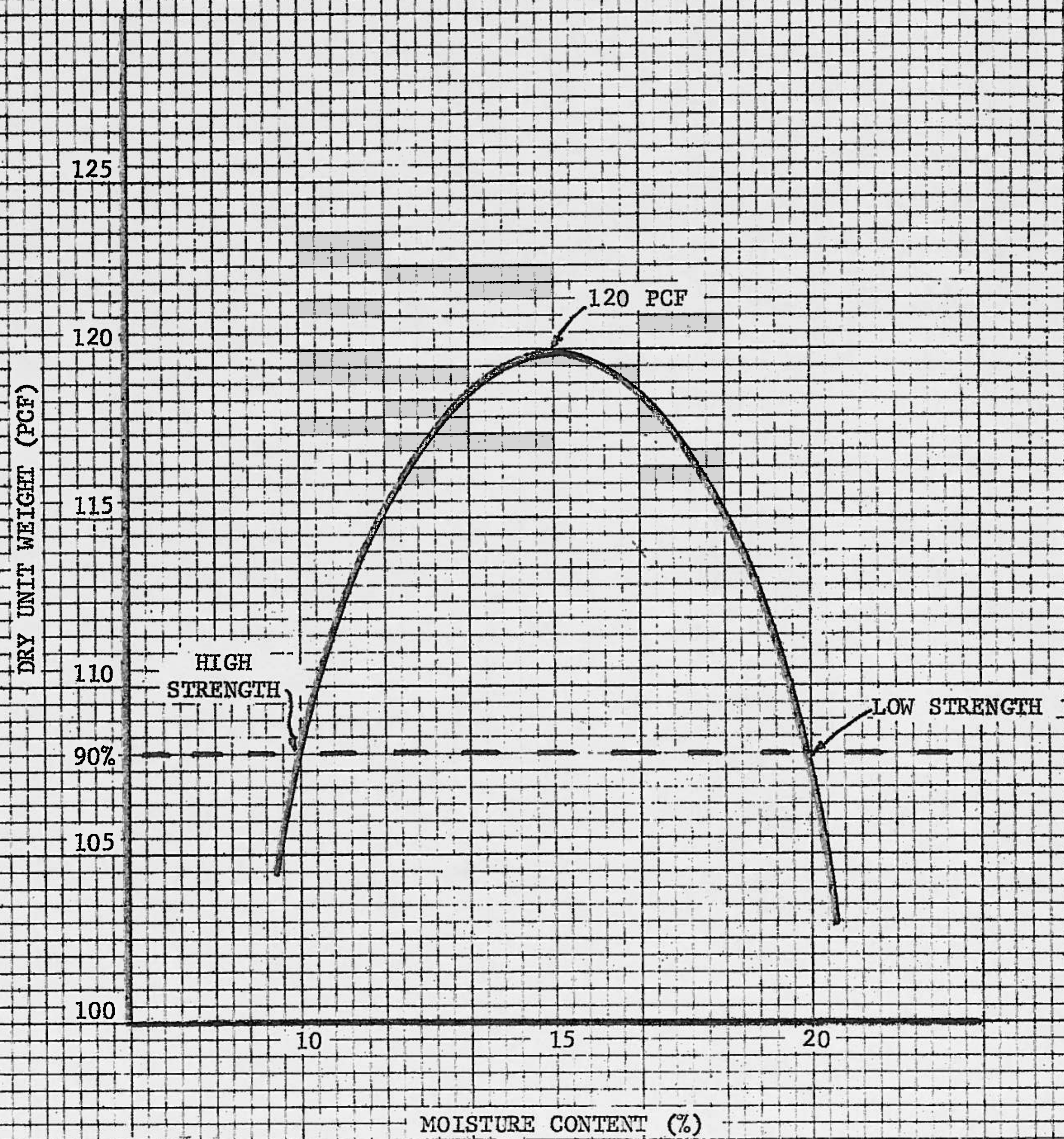
It is obvious from the foregoing discussion that the most important factor in determining the movement of pipes in cohesive type backfill is the selection of the test method and the unit weight and moisture content at which the proposed backfill material will be compacted. It is emphasized that in the writing of the specifications there should be a limit placed on moisture content as well as a percentage compaction specified. Some clay soils are much more sensitive to initial molding moisture content than are other clay soils. Many clay soils lose strength quite rapidly at molding moisture contents greater than the optimum moisture content. The second soil referenced above was a soil such as this and it was found that small changes in moisture content above the optimum affected the strength greatly. It is, therefore, imperative that both a strength and moisture content criteria be written into the specifications and be enforced in the field. In many instances in the field the moisture content control is more important than the control of the percentage of compaction.

It has been concluded that a confined compression test type analysis is the best method for analyzing soil behavior in this particular instance. It should also be observed that a consolidation type analysis could be used for this criteria, however, the conventional consolidation testing equipment confines the material rigidly and does not permit the application of various different lateral loads as would be expected in the field. It might be more proper to load the triaxial specimen to the expected load in the field and then to record movement over a period of time. Whether or not this could be done on a given project depends on the time available and the equipment available for testing. It would appear that the most accurate type of measurement for production of movement curves would be to load the sample of soil to the horizontal and vertical loading expected in the fill and to record movement over a relatively long period of time. This would take into account both what might be termed primary movement and secondary movement.

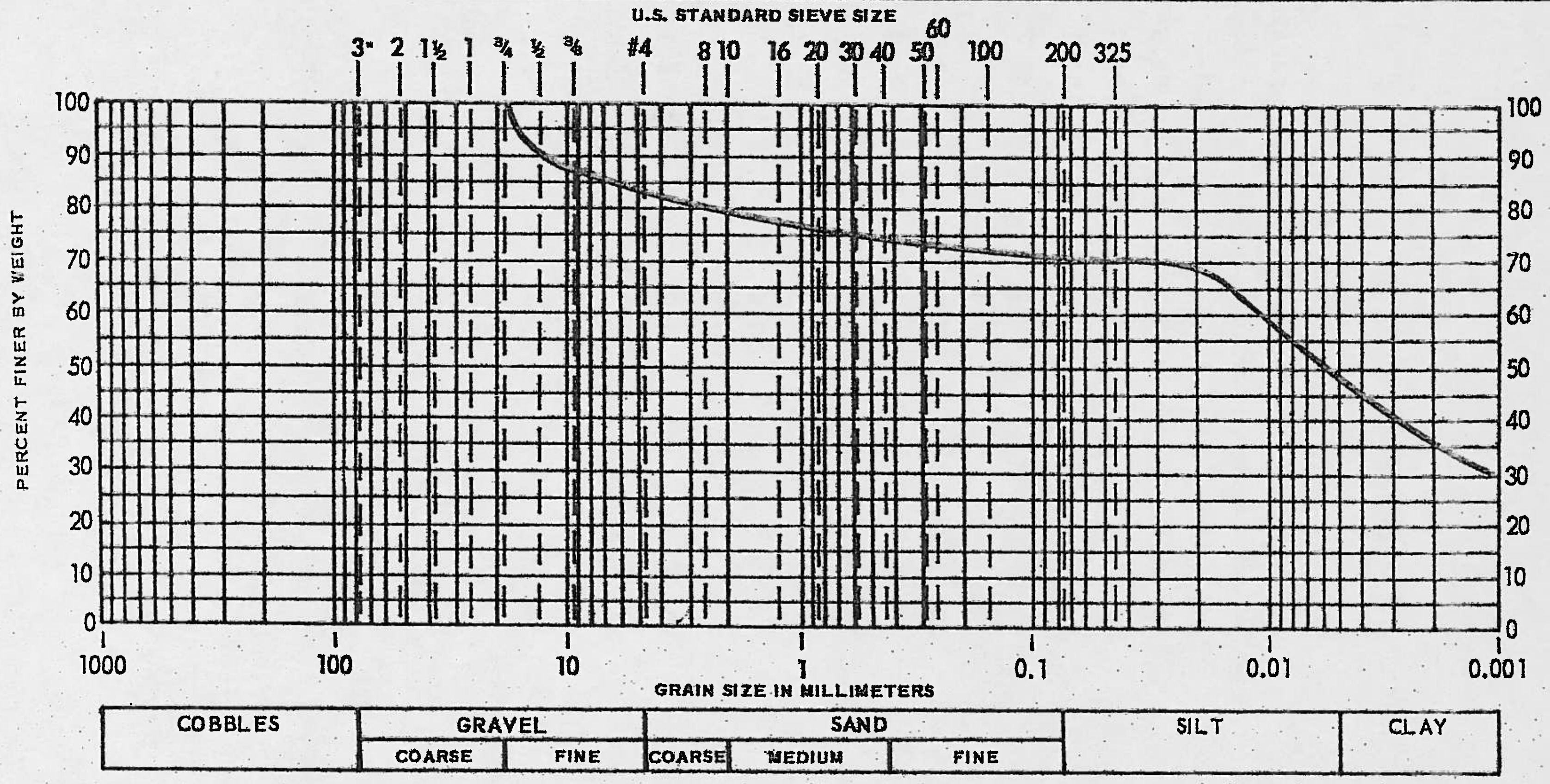
In conclusion, the movement of the soil laterally around large diameter corregated metal pipes can reasonably accurately be predicted with conventional confined compression testing. It should be observed that there is some sort of initial movement or set of the pipe that generated in the pipe are a function of pipe material, pipe thickness, etc., and must be determined for pipe design. This is merely a statement that the amount of movement of the pipe, once a high fill situation at which the pipe would fail if there were no lateral support is reached, is apparently independent of the type of pipe or thickness of pipe material within reasonable limits. The pipe must still be designed in accordance with ring compression and ultimate stresses in the pipe generated by the height of fill.

There is also another phenomenon which should be taken into consideration. This phenomenon has been variously described as time lag and refers to long term consolidation or creep of the material due to the loads placed upon it. This phenomenon has been catalogued, particularly in the case with the second pipe described above, and a consolidation test was performed. A load was placed on the consolidation sample and at the time of this writing has been in place for 6 months. This material continues to move under this load and is moving at a relatively constant rate. From observations made on the pipe itself indications are that this type of movement can be expected with clay type backfills and that this rate of movement is dependent on degree of compaction of the backfill as well as compaction moisture content, but that the time-rate of movement is relatively independent of degree of compaction and is dependent on soil type only. An evaluation must be made as to the significance and as to how much this, what might be termed secondary movement or long term creep, will amount to. It has been found that this movement might be significant in many areas, however, in an examination of the confined compression data and the projection of total amounts of movement in actual field conditions, it seems as if this secondary rate is quite small in comparison with the amount of movement which will be predicted through the confined compression testing. Based on the experience with this one pipe it is felt that if the movement indicated by the confined compression testing procedures referenced above were multiplied by a factor of 1.2 for the time deflection lag, a more accurate value of deflection would be determined. It is emphasized that this value is based on observations on this one pipe only and may not apply in is not explained by this testing. There is also some long term creep of the soil which will not be explained by the confined compression testing. This method is, however, considered sufficiently accurate for the calculation of amounts of movement for the design of the backfill around flexible pipes.

FIGURE 1


120 PCF
HIGH STRENGTH
LOW STRENGTH
125
120
115
110
90%
105
100
10
15
20
DRY UNIT WEIGHT (PCF)
MOISTURE CONTENT (%)

SOIL CLASSIFICATION SHEET



PROJECT	NATURAL % MOISTURE --	BOWSER-MORNER
BORING NO.	LIQUID LIMIT 44	TESTING LABORATORIES, INC.
SAMPLE NO.	PLASTIC LIMIT 21	420 DAVIS AVE.
DEPTH	PLASTICITY INDEX 23	DAYTON, OHIO
CLASSIFICATION	COLOR Brown	
U.S.C.S. - "CL" Silty Clay	REMARKS U.S.C.S.	
A.A.S.H.O. A-7-6(13) Clay	Specific Gravity 2.74	

FIGURE 2

2B-1A

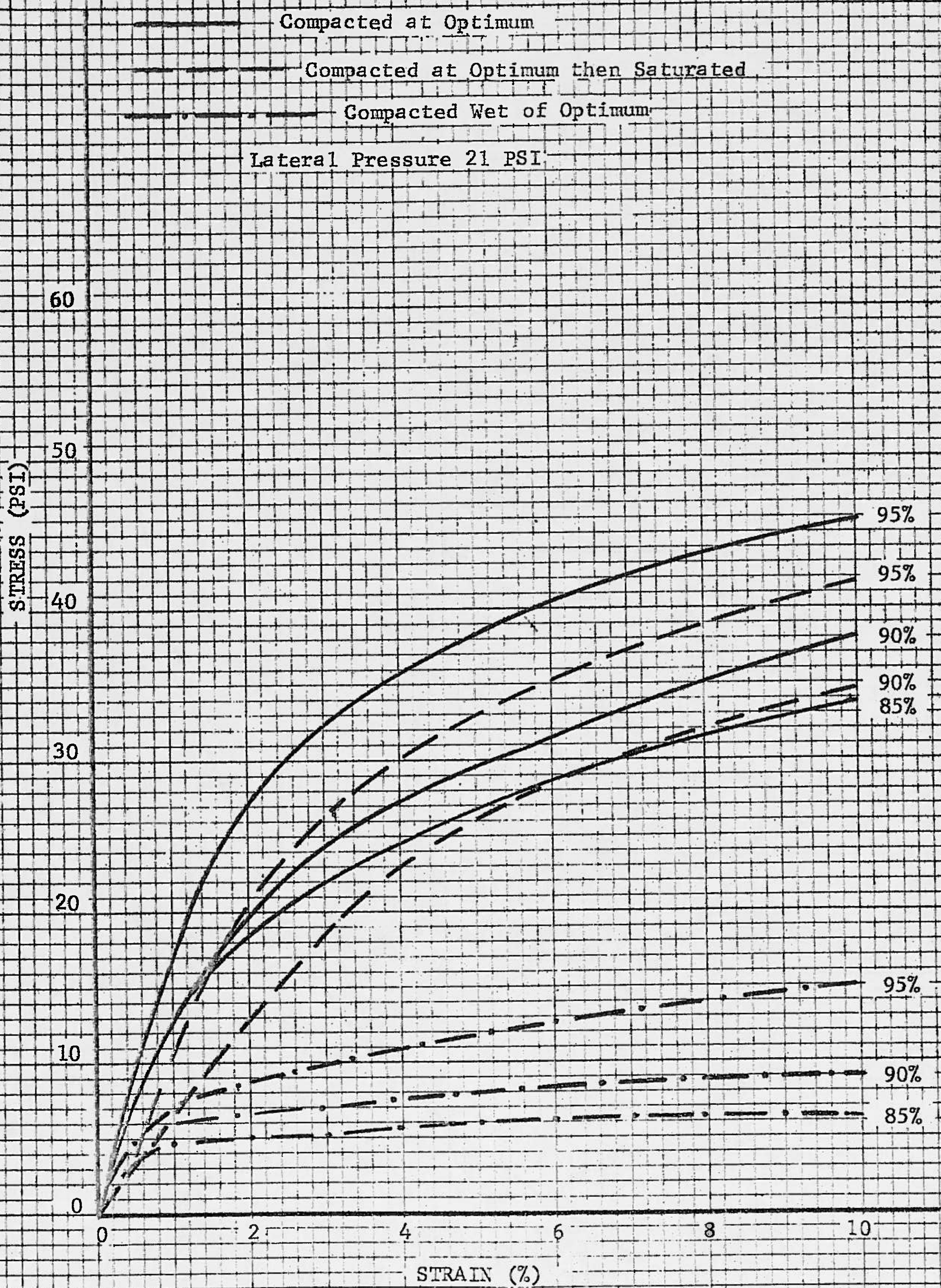
FIGURE 3

TABLE OF UNIT WEIGHT AND MOISTURE CONTENTS

Boring No.	Depth of Sample (ft.)	Dry Unit Weight (pcf)	Moisture Content (%)	Compaction (%)
1	10.1-10.6	112.4	18.9	99.3
1	18.0-20.0	106.7	18.4	94.3
1	23.0-23.5	113.4	19.6	100.2
1	28.2-28.7	96.4	29.1	85.2
2	8.0- 8.5	105.4	19.1	93.1
2	8.8- 9.3	124.9	12.5	110.3
2	18.3-18.8	100.6	26.5	88.9
2	23.8-24.3	108.9	21.0	96.2
2	28.0-29.0	112.3	18.9	99.2
3	20.0-20.5		20.4	
*3	22.0-23.5		17.2	
3	26.0-26.4	103.4	25.0	91.3
3	27.5-27.9	99.2	27.1	87.6
*3	29.5-31.0		30.7	
4	20.6-21.1	98.7	25.7	87.1
*4	22.0-23.5		18.0	
4	26.0-26.5	120.0	15.4	105.9
*4	28.0-29.5		9.5	
4	30.5-30.9	105.1	22.4	92.8
*4	32.0-33.5		23.1	
Inside Pipe (top 6")		115.6	7.7	
Inside Pipe (bottom 3")			14.5	

* Disturbed Sample

FIGURE 4



Compacted at Optimum
Compacted at Optimum then Saturated
Compacted Wet of Optimum
Lateral Pressure 21 PSI
STRESS (PSI)
0
10
20
30
40
50
60
STRAIN (%)
0
2
4
6
8
10
95%
95%
90%
90%
85%
95%
90%
85%

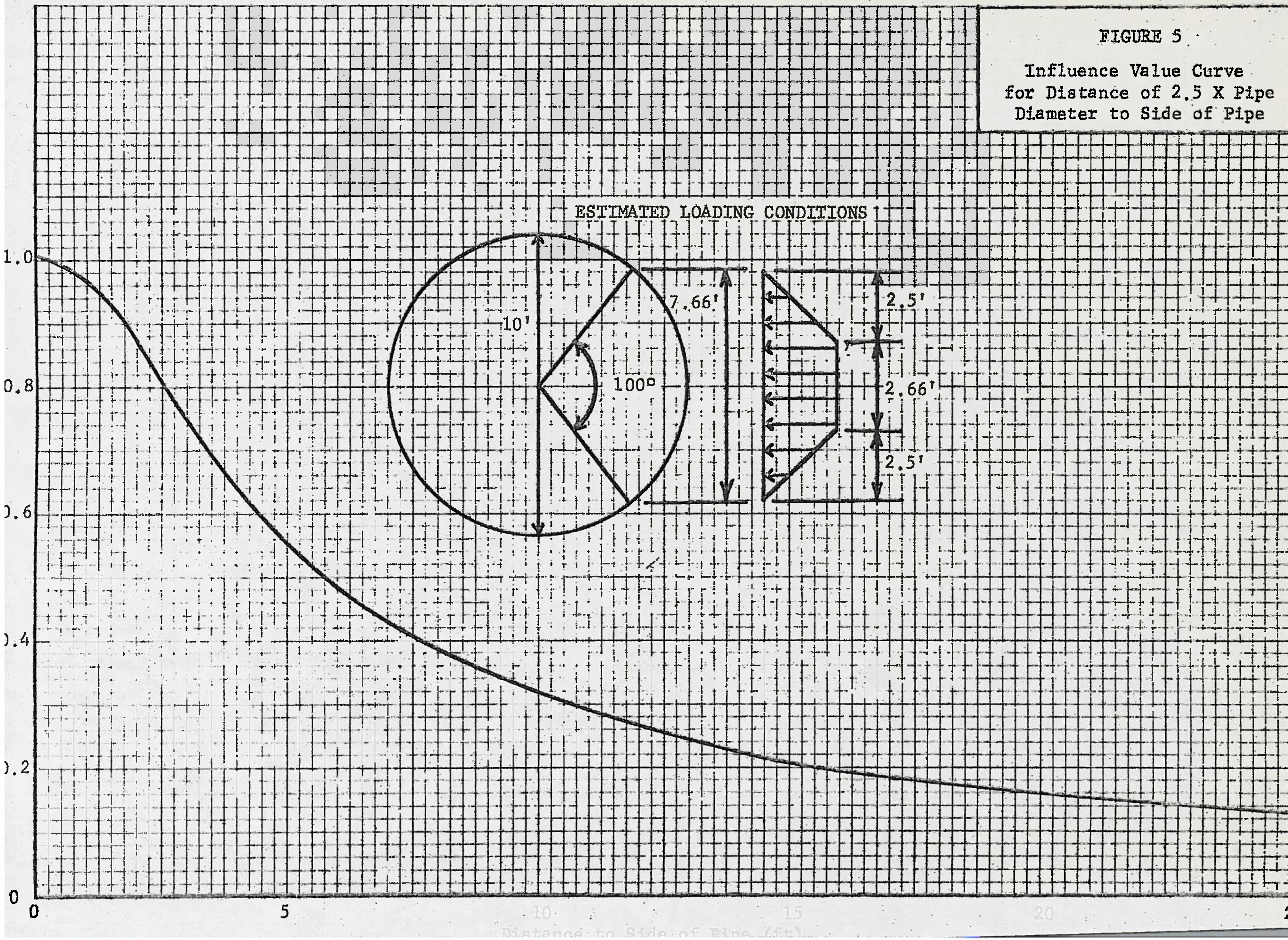
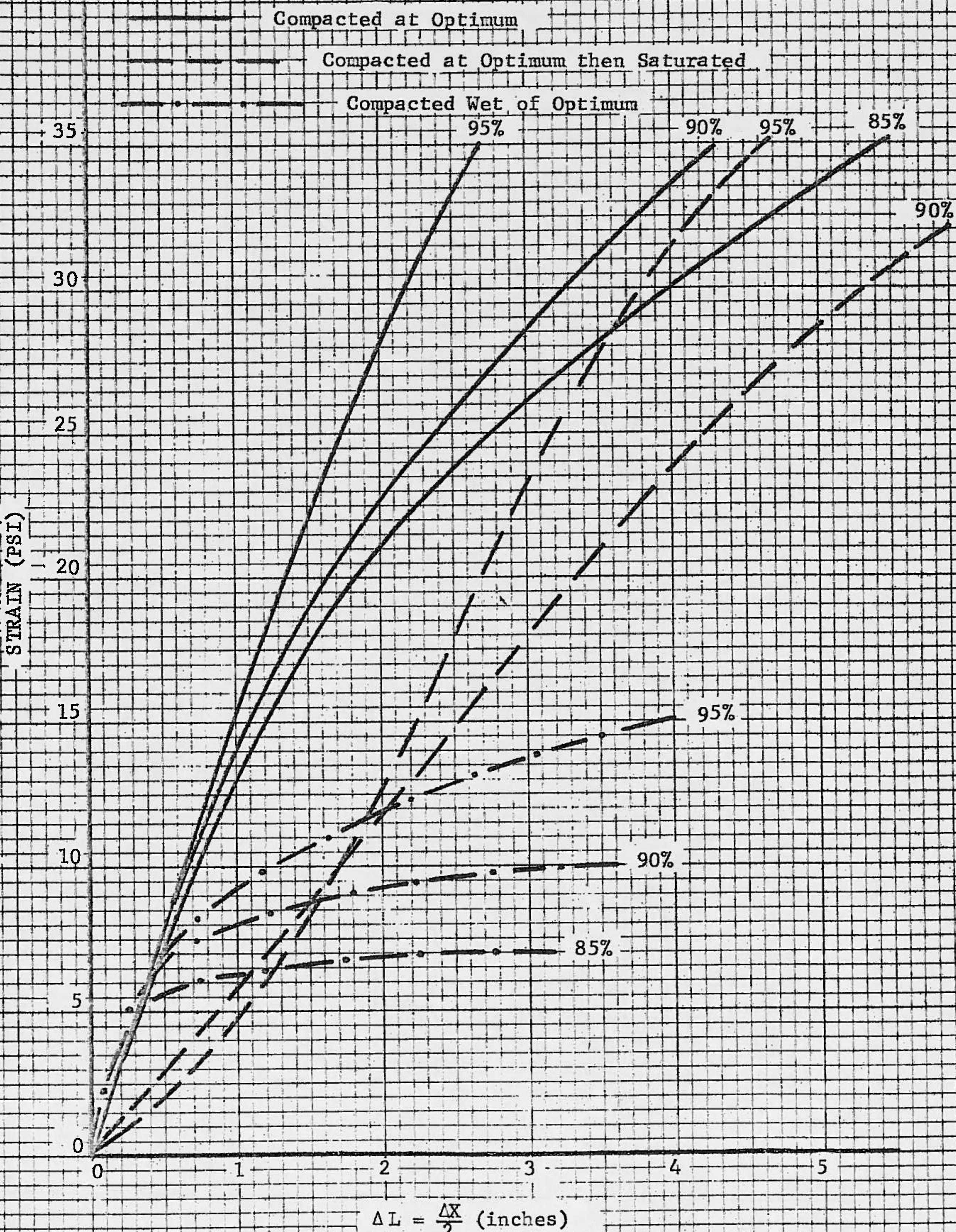


FIGURE 5

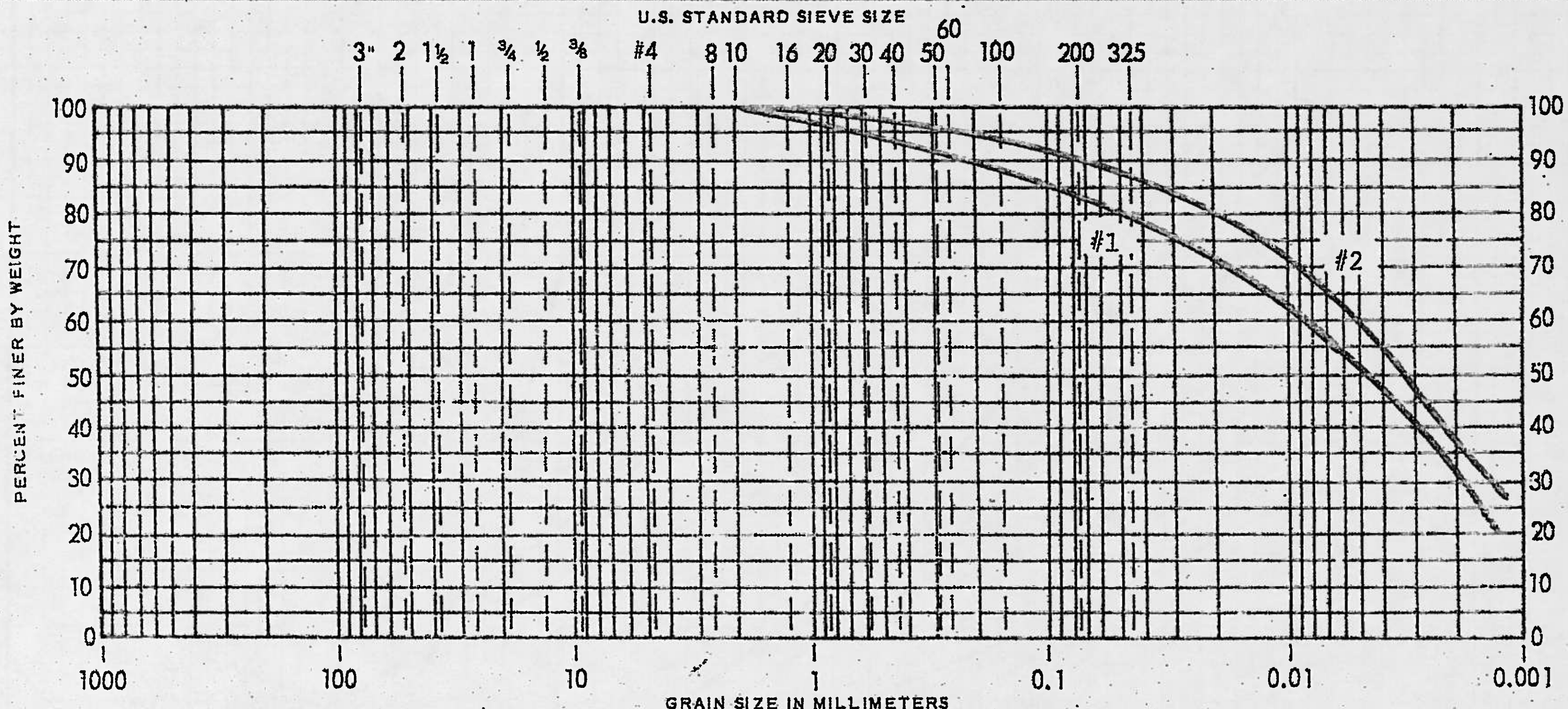
Influence Value Curve for Distance of 2.5 X Pipe Diameter to Side of Pipe

FIGURE 6

SOIL-PIPE INTERACTION CURVES



SOIL CLASSIFICATION SHEET



COBBLES	GRAVEL		SAND #1 – 17% #2 – 10%			SILT	CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	#1 – 31% #2 – 31%	#1 – 52% #2 – 59%

PROJECT

BORING NO.
SAMPLE NO.
DEPTH
CLASSIFICATION
#1 "CL" Silty, Clay
#2 "CL" Silty, Clay

	#1	#2
LIQUID LIMIT	37	42
PLASTIC LIMIT	21	21
PLASTICITY INDEX	16	21
COLOR	Red	Red

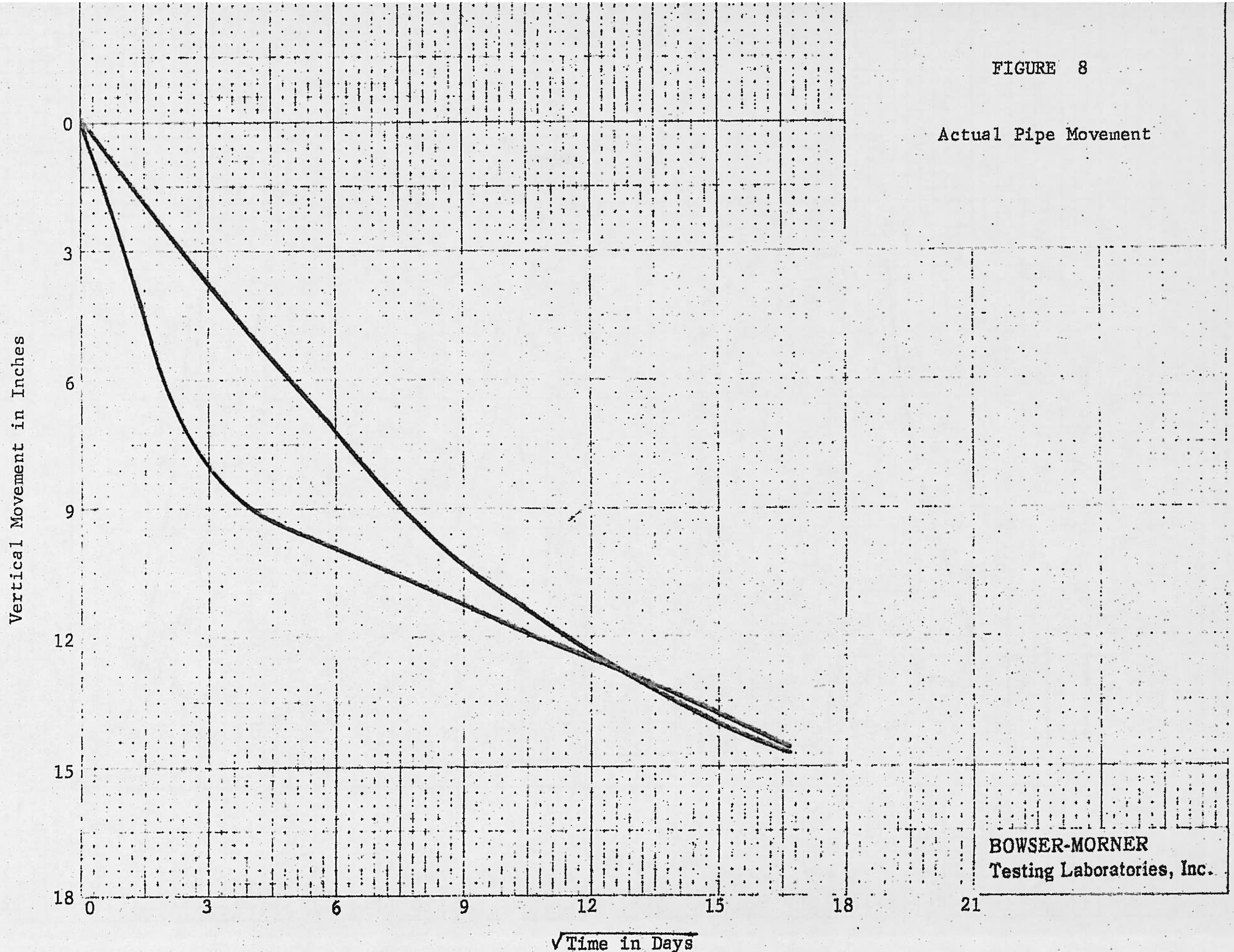
REMARKS "U.S.C.S."

BOWSER-MORNER
TESTING LABORATORIES, INC.
420 DAVIS AVE.
DAYTON, OHIO

FIGURE 7

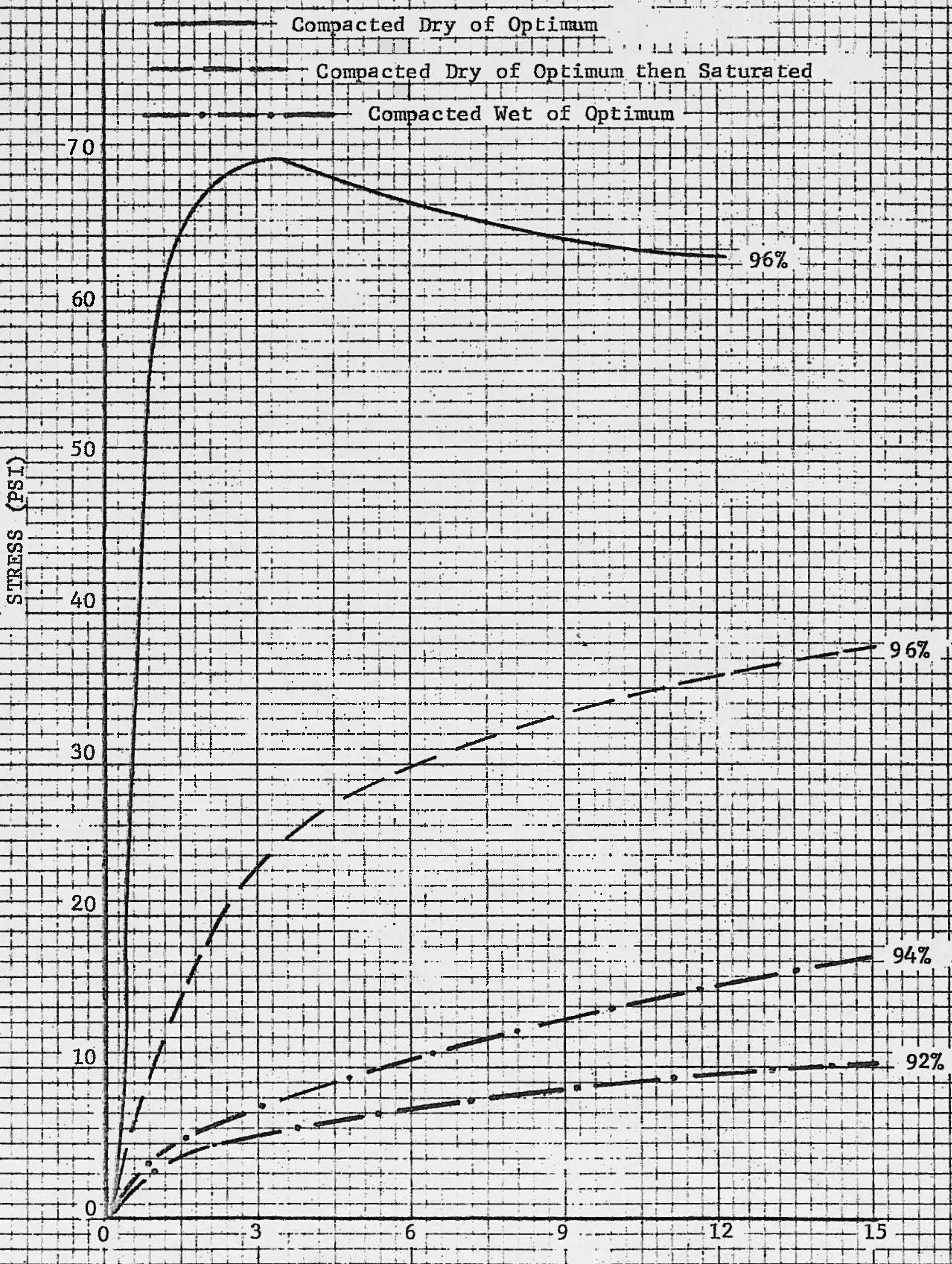
FIGURE 8

Actual Pipe Movement



BOWSER-MORNER
Testing Laboratories, Inc.

FIGURE 9


Compacted Dry of Optimum
Compacted Dry of Optimum then Saturated
Compacted Wet of Optimum
70
60
50
40
30
20
10
0
STRESS (PSI)
96%
96%
94%
92%
0
3
6
9
12
15
STRAIN (%)

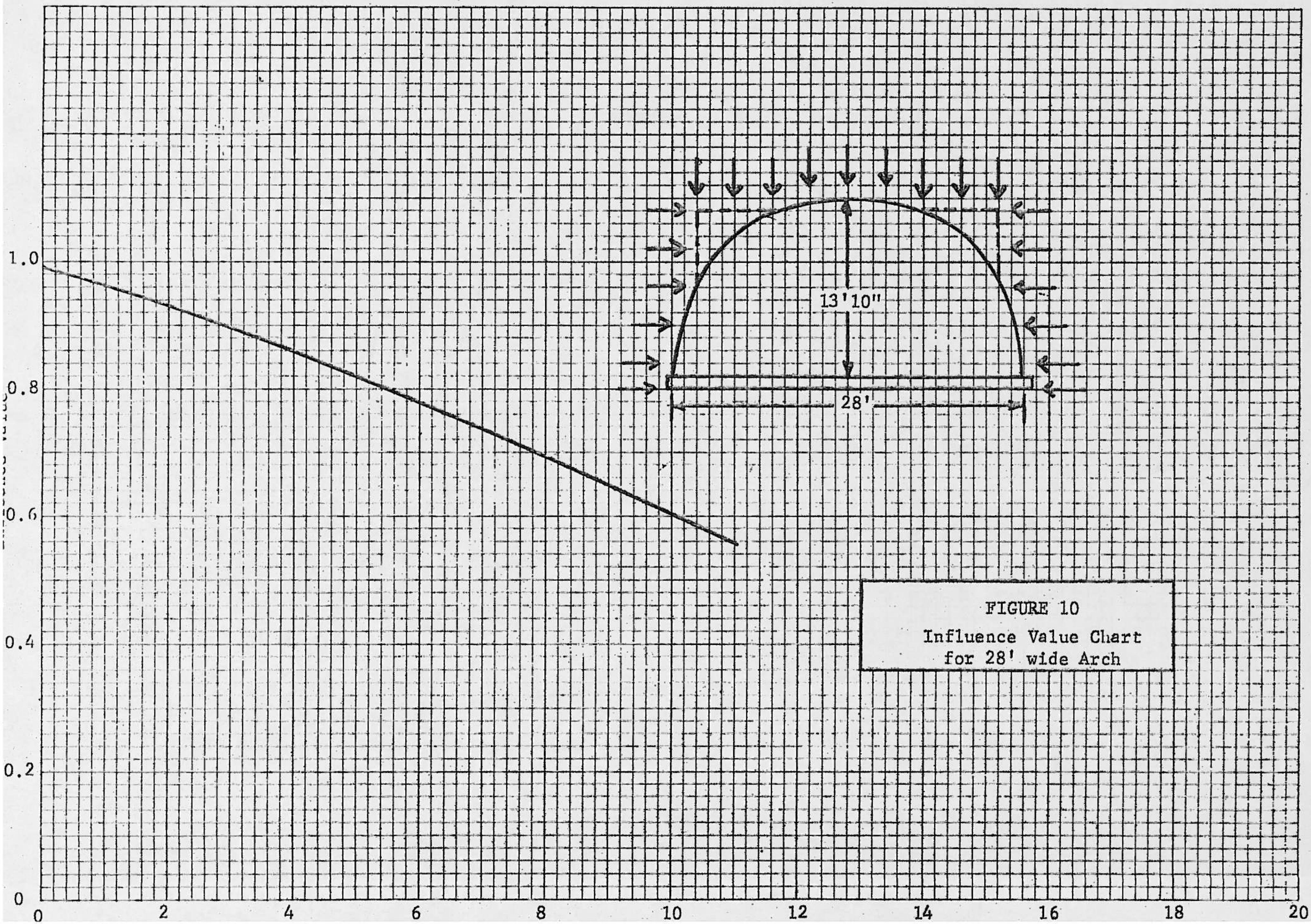


FIGURE 10
Influence Value Chart
for 28' wide Arch

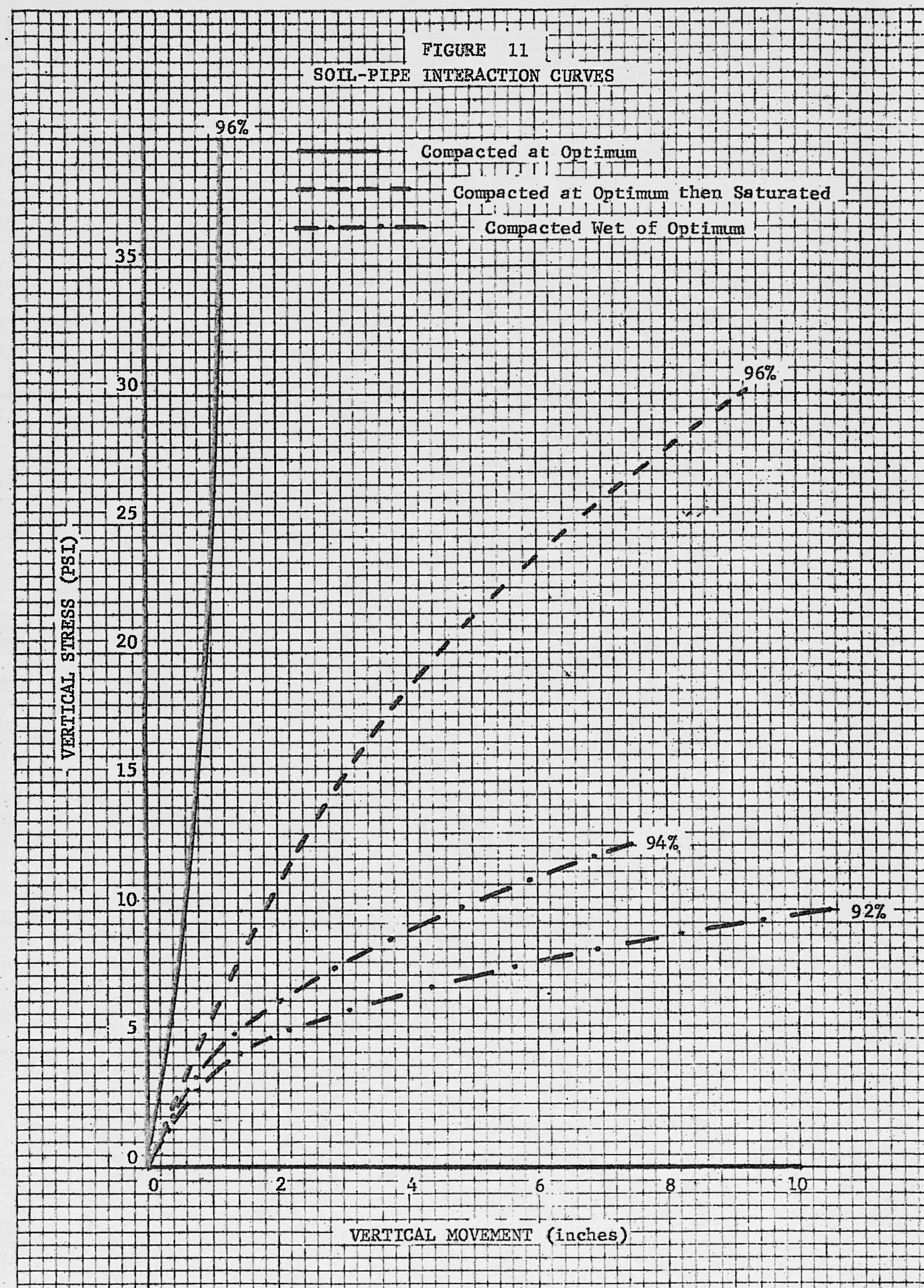
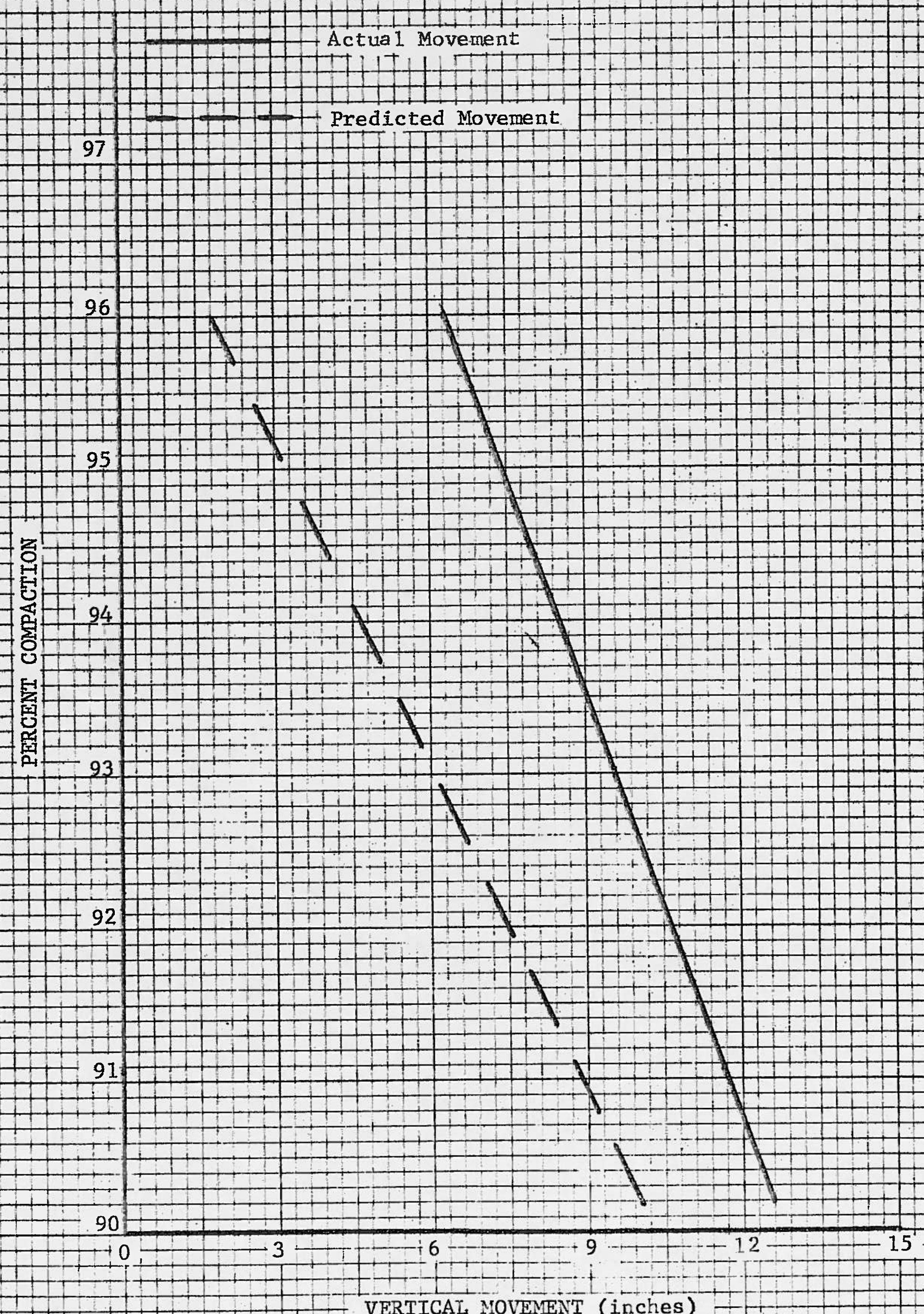


FIGURE 11
SOIL-PIPE INTERACTION CURVES
Compacted at Optimum
Compacted at Optimum then Saturated
Compacted Wet of Optimum
96%
96%
94%
92%
VERTICAL STRESS (PSI)
0
5
10
15
20
25
30
35
0
2
4
6
8
10
VERTICAL MOVEMENT (inches)

FIGURE 12



Actual Movement
Predicted Movement
97
96
95
94
93
92
91
90
PERCENT COMPACTION
0
3
6
9
12
15
VERTICAL MOVEMENT (inches)

Generalized Sliding Wedge Method For
Slope Stability and Earth Pressures Analysis

Vincent P. Drnevich*

INTRODUCTION

Slope failures and failures of retaining walls and braced excavations often cause extensive damage. Repair of these failures usually requires precious time and is expensive. As a consequence, engineers today are trying to minimize the chances of such failures by performing more extensive soil investigations and stability analyses. The digital computer has helped the engineer immeasurably in performing stability analyses and there are now multitudinous programs available in varying degrees of sophistication.

There is one class of slope stability problem where soil conditions are complicated and obvious possible failure surfaces are irregular and noncircular. Methods of analysis of stability which require a circular failure surface or only take average soil conditions into account give at best very crude estimates of factors of safety. Some of these may grossly overestimate the factors of safety on one hand and on the other cause the owner needless expense.

Some excellent and sophisticated methods of analysis have been developed in recent years[2, 4, 5] and have been made available to the practicing engineer. However, they tend to be cumbersome and expensive to use.

The method discussed in this paper is one that can be used for rather complicated soil conditions and failure surfaces. It is known and used by practically all soils engineers and the calculations are usually made by hand or with the aid of the desk calculator. The main contribution of this paper is to generalize the method to allow practically any

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possible failure surface to be analyzed and to extend the method to cover effective stress conditions where excess pore pressures exist. The method remains simple, inexpensive, and easy to use particularly if the digital computer is employed.

Details of the method are covered and several examples are given. Finally, some limitations and cautions for use are discussed.

BASIC SLIDING WEDGE METHOD

In the sliding wedge method, all of the soil that is contained by the failure surface is divided into wedges or blocks having straight sides as shown in Fig. 1. The lower boundary of a wedge or block is a portion of the failure surface. If the slope of this line is negative, then the wedge is called an active wedge much the same as for active Rankine conditions. In a similar fashion, if the slope of the boundary is positive such that the wedge would act to restrain other wedges or blocks, the wedge is termed a passive wedge. Finally, the block of soil between the active and passive wedges is referred to as a central block. In typical problems, there may be a number of each type of wedges or blocks.

The procedure for analysis is relatively simple. It varies somewhat depending on which reference is used. The simplest procedure involves: (1) determining the active or passive forces associated with each wedge (usually a handbook or curves are employed), (2) determining the driving force associated with the central block, (3) determining the sliding resistance beneath the central block, and (4) taking the summation of forces acting on the central block to establish a factor of safety.

A slightly more complicated but more realistic method is recommended in NAVDOCKS[3] where the driving and resisting components of the active and passive wedges are considered separately. The factor of safety is then defined as the summation of the resisting forces divided by the summation of the driving forces.

The procedure used by the author is similar to the one described in NAVDOCKS with the exception that the active and passive forces are calculated for more general conditions. In doing so, the central block can be handled like an active or passive wedge depending on the slope of the failure surface beneath it.

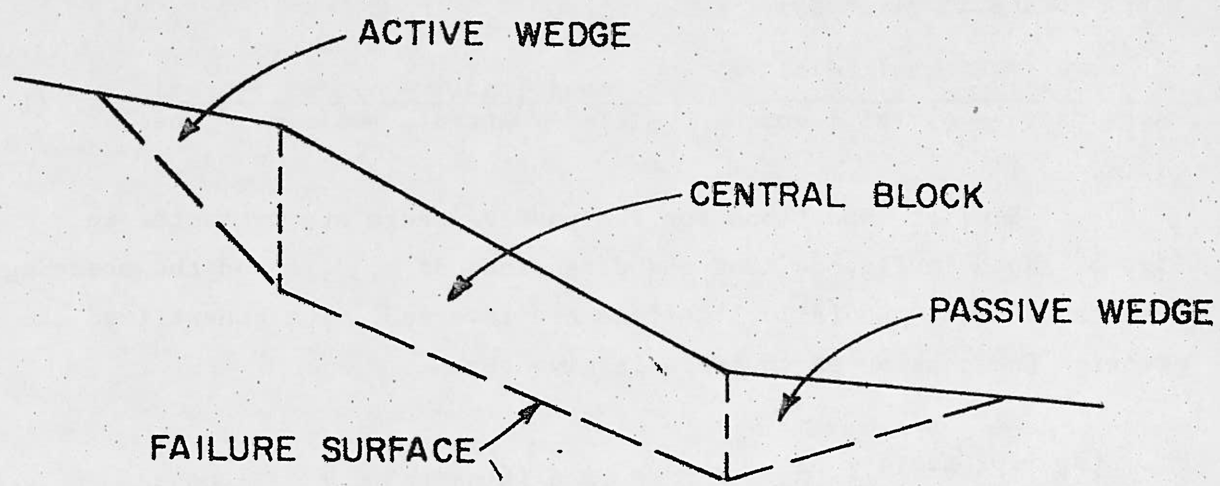


FIGURE I - BASIC SLIDING WEDGE METHOD

The forces and equations for a generalized active wedge are shown in Fig. 2. In Fig. 2a note that the backfill angle, β, and the failure surface angle, α, are completely general. Also, the angle that the active pressure, P_A, makes with the horizontal is given by δ. The force, W_1, applied to the top of the wedge is to account for blocks or wedges of soil above the active wedge in question.

In Fig. 2b the driving force, DF, is determined from a force polygon where no shearing resistance of the soil is assumed to exist.

Finally, in Fig. 2c, the polygon of forces is presented where the soil shearing resistance is considered. The expression for active earth forces is given by:

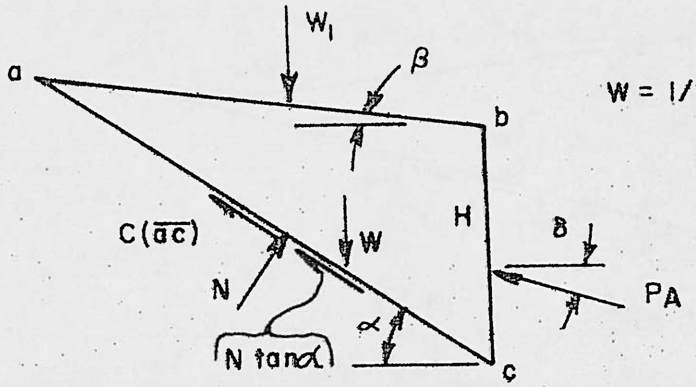
$$P_A = \frac{(W_1 + W)\tan(\alpha - \phi)}{\tan(\alpha - \phi)\sin\delta + \cos\delta} - \frac{cH\cos\beta\,[\tan(\alpha - \phi)\sin\alpha + \cos\alpha]}{\sin(\alpha - \beta)[\tan(\alpha - \phi)\sin\delta + \cos\delta]} \qquad (1)$$

Similar conditions for the passive wedge are presented in Fig. 3. Note in Fig. 3a that the directions of α, β, δ and the shearing resistance along the failure surface are reversed. The generalized expression for passive earth force is given by:

$$P_P = \frac{(W_1 + W)\tan(\alpha + \phi)}{\cos\delta - \sin\delta\tan(\alpha + \phi)} + \frac{cH\cos\beta\,[\sin\alpha\tan(\alpha + \phi) + \cos\alpha]}{\sin(\alpha + \beta)[\cos\delta - \sin\delta\tan(\alpha + \phi)]} \qquad (2)$$

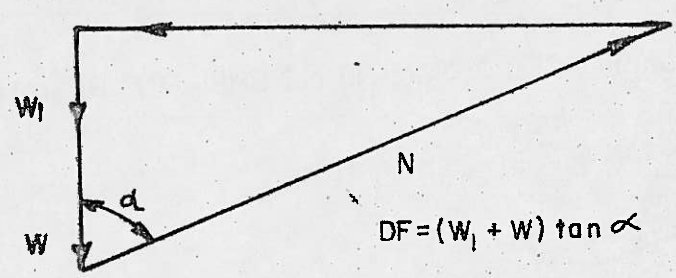
If β and δ are set to zero and the value of α is set to $45° + \phi/2$ for the active case or to $45° - \phi/2$ for the passive case, the above equations give the active and passive earth pressures for Rankine conditions.

In practical problems, there may be one or more central blocks. The central block shown in Fig. 1 is really an active wedge with a surcharge block of soil acting above it. This is shown in Fig. 4 where a central block is composed of an active wedge and two surcharge blocks. The surcharge blocks (or wedges) can be used to simulate different soil layers having different densities or can be used to simulate water table and excess pore pressure conditions in the case of effective stress analysis. The latter will be discussed in detail subsequently.



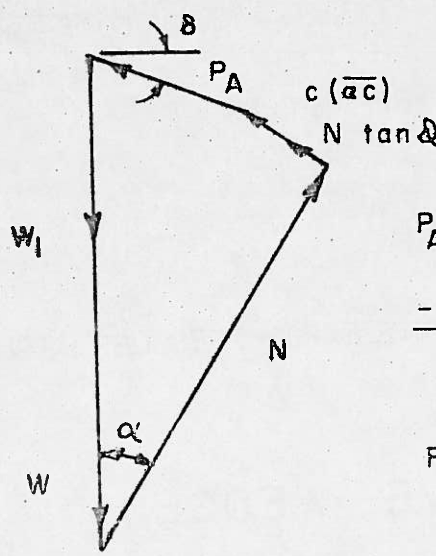
$$W = 1/2\,\gamma\,H^2\,\frac{\cos\alpha\cos\beta}{\sin(\alpha+\beta)}$$

a) FORCES ON WEDGE



$$DF = (W_1 + W)\tan\alpha$$

b) POLYGON OF FORCES ASSUMING NO SHEARING RESISTANCE



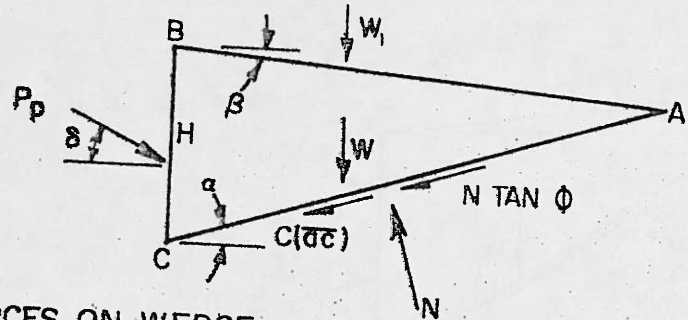
$$P_A = (W_1 + W)\,\frac{\tan(\alpha-\phi)}{\tan(\alpha-\phi)\sin\delta+\cos\delta}$$

$$-\frac{cH\cos\beta\left[\tan(\alpha-\phi)\sin\alpha+\cos\alpha\right]}{\sin(\alpha-\beta)\left[\tan(\alpha-\phi)\sin\delta+\cos\delta\right]}$$

$$RF = DF - P_A\cos\delta$$

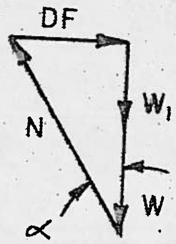
c) POLYGON OF FORCES TO GET ACTIVE EARTH PRESSURE RESULTANT

FIGURE 2 - ACTIVE WEDGE

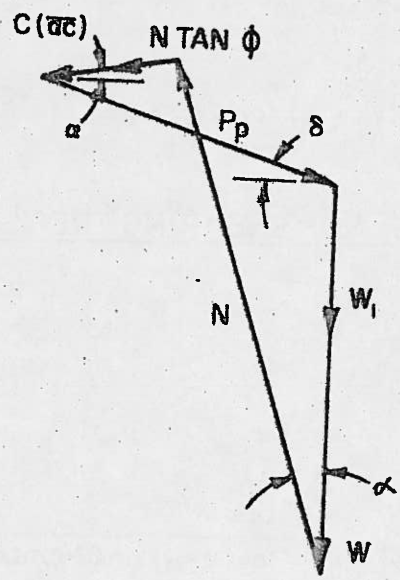


a) FORCES ON WEDGE

$$DF = (W_1 + W)\ TAN\ \alpha$$



b) POLYGON OF FORCES ASSUMING NO SHEARING RESISTANCE



$$P_p = (W_1 + W)\frac{TAN\ (\alpha + \phi)}{COS\ \delta - SIN\ \delta\ TAN\ (\alpha + \phi)} + \frac{c\ H\ COS\ \beta}{SIN(\alpha + \beta)}\left[\frac{SIN\ \alpha\ TAN\ (\alpha + \phi) + COS\ \alpha}{COS\ \delta - SIN\ \delta\ TAN(\alpha + \phi)}\right]$$

$$RF = P_p\ COS\ \delta - DF$$

d) POLYGON OF FORCES TO GET PASSIVE EARTH PRESSURE RESULTANT

FIGURE 3 - PASSIVE WEDGE

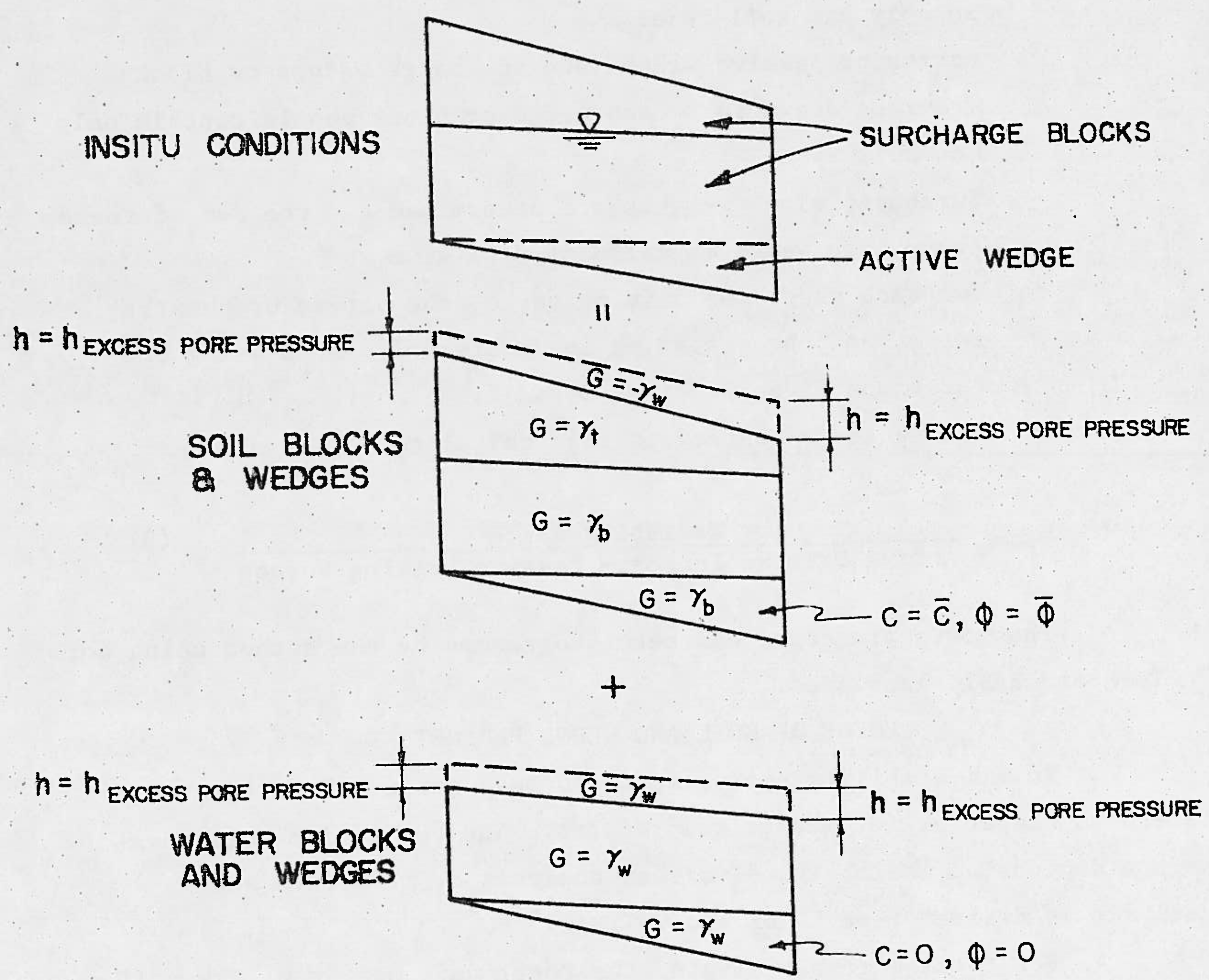


FIGURE 4 - ACTIVE WEDGE WITH SURCHARGE BLOCKS

With the central blocks defined as above, the procedure for analysis is greatly simplified. The steps are as follows:

1. A probable failure surface is drawn on the cross section.
2. The failure surface is fitted with straight line segments. The segments should be chosen such that each segment is in only one soil type.
3. Active or passive wedges and surcharge wedges or blocks are then drawn in. Each wedge or block should contain only one soil type.
4. Surcharge block weights are determined and the sum of these above each wedge is calculated to give W_1.
5. For each wedge the driving force, the active or passive force, and the resisting force are calculated using the equations given in Figs. 2 and 3.
6. The factor of safety is then calculated from:

$$FS = \frac{\text{Resisting Force}}{\text{Active Driving Forces} - \text{Passive Driving Forces}} \qquad (3)$$

The above procedure has been programmed by the author using both FORTRAN and BASIC languages.

CHOICE OF SOIL AND WEDGE PARAMETERS

In all stability analyses, the proper choice of soil parameters is essential for meaningful results. The choice for soil parameters depends on whether total stress analysis or effective stress analysis is desired (see LAMBE and WHITMAN)[1].

For total stress analysis, the total unit weight of the soil is used and c and ϕ in terms of total stress are used. If unconfined compression tests or insitu tests such as vane or penetrometer tests are used to determine the shear strength, the shear strength is defined by c and ϕ is set to zero. This is known as a "$\phi = 0$ analysis" and is probably the most common type of analysis made because it simulates the conditions immediately after construction.

For effective stress analyses, total soil unit weights are used above the zone of saturation and buoyant unit weights are used in the zone. Shear strengths are in terms of effective stress, i.e., $c = \bar{c}$

and $\phi = \bar{\phi}$. The water table is taken into account by superimposing on the soil wedges and blocks, similar wedges and blocks to represent the water table. (See Fig. 4). In these, the unit weight is set equal to the unit weight of water and the shear strength parameters are set to zero. For the water wedges, the active or passive forces are equal to the driving forces for each wedge. Hence, the water wedges produce only values of driving force with the resisting forces all being equal to zero. Thus, the use of water blocks and wedges takes into account seepage forces by essentially considering the boundary water pressures.

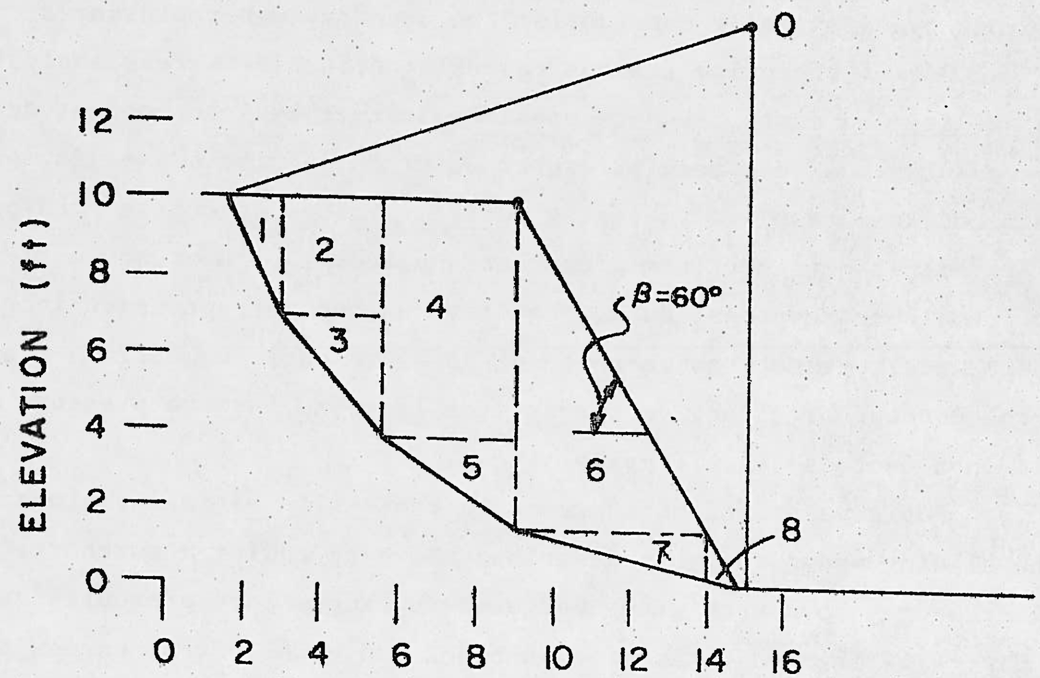
One of the prime reasons for using effective stress analysis for stability and earth pressure problems is to take into account excess pore pressures. Excess pore pressures occur during the consolidation process and as a result of shearing action. Values of excess pore pressure are either obtained from piezometers placed insitu or are estimated from laboratory test data. Positive excess pore pressure increases the pore water pressure above that due to hydrostatic conditions alone. From the concept of effective stress, the increase in pore pressure must result in a decrease in effective stress.

Positive excess pore pressures can easily be taken into account in the sliding wedge analysis described above by adding a surcharge water block or wedge above each water wedge where excess pore pressures exist. (See Fig. 4). The ordinates of each block are equal to the excess pore pressure expressed in terms of pressure head (excess pore pressure/unit weight of water). To satisfy the effective stress equation, an identical surcharge water block having a unit weight equal to minus the unit weight of water is placed above the corresponding soil wedges.

The last item that needs to be specified in the analysis is the friction angle, δ, at the face of the wedge. A value of $\delta = 0$ gives the most conservative results for a given analysis. Terzaghi and Peck[6] point out that δ greater than $\phi/3$ should not be used for passive wedges as it will introduce considerable error. Until the effect of δ is firmly established, the author recommends that a value of $\delta = 0$ be used.

EXAMPLES OF SLOPE STABILITY ANALYSES

The first example is a simple slope in a homogeneous soil as shown in Fig. 5. The slope angle β is 60° and the failure surface is circular with center at point 0. If the failure surface is fitted with straight line segments and the material in the failure zone is fitted



γ = 100 lb/ft^3

c = 192 lb/ft^2

ϕ = 0

FIGURE 5 - SIMPLE SLOPE ϕ = 0 ANALYSIS

with wedges and blocks as shown, the generalized wedge method can be used to estimate the factor of safety. The computer output for this problem is given in Fig. 6. The first portion is simply a printout of the input data. One line describes each block or wedge giving the coordinates of each point, the wedge type (type (1) if on failure surface and type (0) if not), the soil unit weight, the shear strength parameters, and the "δ" angle (denoted by E(J)).

Next, the calculated results are printed out. For each block or wedge, the driving force, the resisting force, the weight and the α and β angles (denoted by A and B) are given. The values of α and β are calculated by the program and serve as a check on the accuracy of the input data. Note that the β angle in wedge 8 should be 60° instead of 56.3°.

Finally, the sums of the wedge weights, resisting forces, driving forces, and the factor of safety are calculated. The sums make it possible to approximately account for earthquake forces. This is done by multiplying the sum of the weights by an earthquake coefficient and then adding this to the sum of the driving forces. A new factor of safety is then calculated using Eq. (3). It should be emphasized that the above procedure for earthquake forces is relatively crude and should only be applied to slopes of minor importance.

The factor of safety calculated for this example is 1.10. If a sliding circle analysis were applied to the same failure surface, the factor of safety also equals 1.10.

An example where the method is used with an effective stress analysis is given in Figs. 7 and 8. The example is taken from NAVDOCKS.[3] The failure surface was fitted with five line segments and a total of 18 blocks and wedges are needed. Note that block and wedge numbers 12 through 18 are water wedges. The computer output is shown in Fig. 8. The factor of safety calculated by this method is 1.24 whereas the one calculated in NAVDOCKS is 1.08. The difference lies in the fact that boundary water forces are incorrectly handled in the NAVDOCKS example. The resultant of the net pore water pressure acting on the two ends of the central wedge should be 6.87 kips instead of 10.54 kips. When this

DRNEVICH SLOPE STABILITY BY SLIDING WEDGE

NUMBER OF WEDGES OR BLOCKS = 8

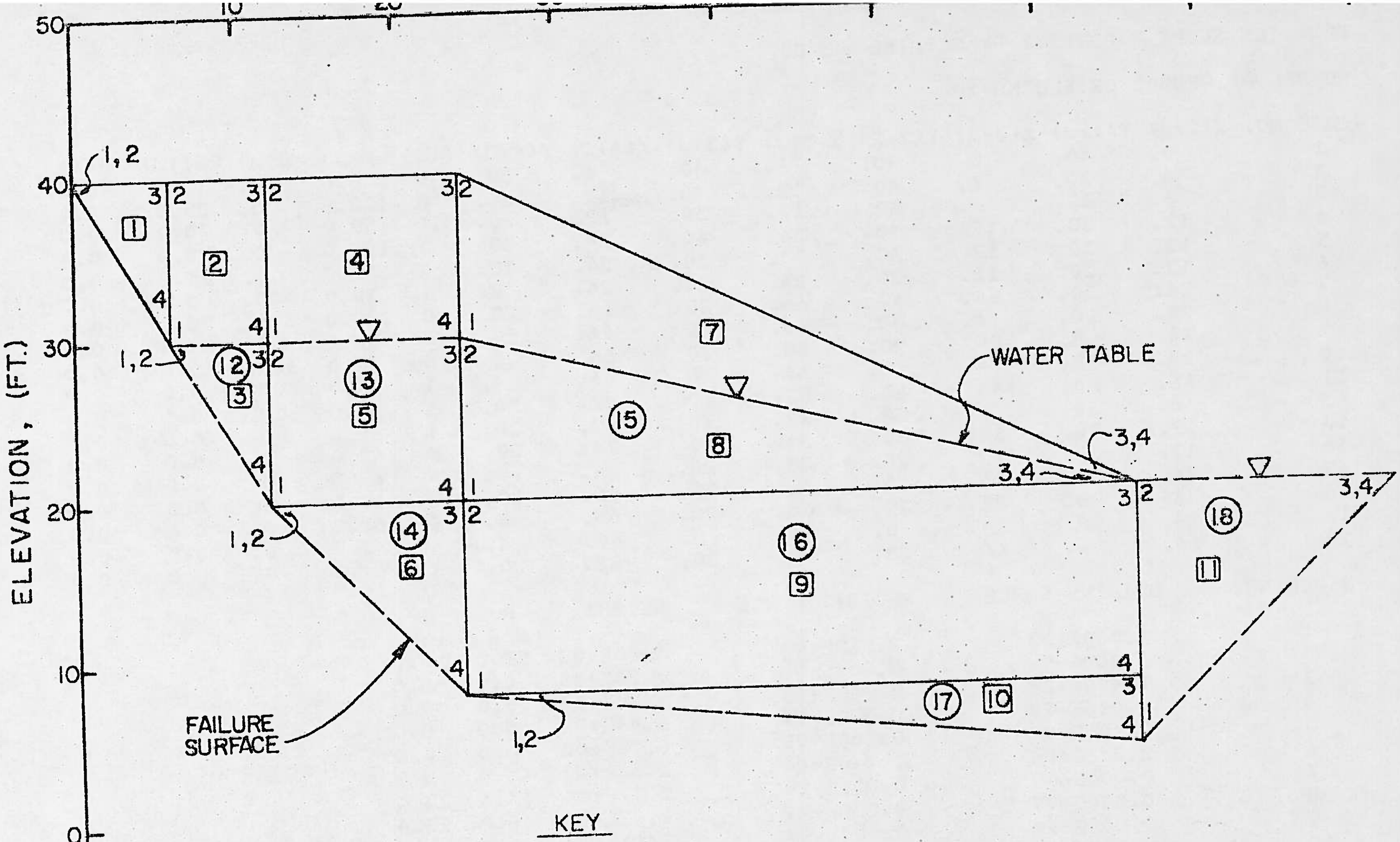
WEDGE NO.	X(1,J)	Y(1,J)	X(2,J)	Y(2,J)	X(3,J)	Y(3,J)	X(4,J)	Y(4,J)	T(J)	G(J)	C(J)	PHI(J)	E(J)
1	1.6	10.0	1.6	10.0	3.0	10.0	3.0	7.0	1	100.0	192.	0.0	0.0
2	3.0	7.0	3.0	10.0	5.6	10.0	5.6	7.0	0	100.0	192.	0.0	0.0
3	3.0	7.0	3.0	7.0	5.6	7.0	5.6	3.8	1	100.0	192.	0.0	0.0
4	5.6	3.8	5.6	10.0	9.0	10.0	9.0	3.8	0	100.0	192.	0.0	0.0
5	5.6	3.8	5.6	3.8	9.0	3.8	9.0	1.5	1	100.0	192.	0.0	0.0
6	9.0	1.5	9.0	10.0	14.0	1.5	14.0	1.5	0	100.0	192.	0.0	0.0
7	9.0	1.5	9.0	1.5	14.0	1.5	14.0	0.1	1	100.0	192.	0.0	0.0
8	14.0	0.1	14.0	1.5	15.0	0.0	15.0	0.0	1	100.0	192.	0.0	0.0

WEDGE NO.	DRIVING FORCE	RESIST. FORCE	WEIGHT	A DEG	B DEG
1	4.500E+02	1.503E+03	2.10E+02	65.0	0.0
2	0.000E-99	0.000E-99	7.80E+02	0.0	0.0
3	1.472E+03	1.255E+03	4.16E+02	50.9	0.0
4	0.000E-99	0.000E-99	2.11E+03	0.0	0.0
5	1.691E+03	9.515E+02	3.91E+02	34.1	0.0
6	0.000E-99	0.000E-99	2.13E+03	0.0	59.5
7	6.930E+02	1.035E+03	3.50E+02	15.6	0.0
8	7.000E+00	0.000E-99	7.00E+01	5.7	56.3

SUM WT = 6.45E+03 SUM RESIST. = 4.75E+03 SUM DRIVE = 4.31E+03 FACTOR OF SAFETY = 1.100

ABOVE RESULTS ARE FOR FAILURE SURFACE SELECTED BY THE USER. THE SURFACE SELECTED MAY NOT BE A CRITICAL SURFACE.

Fig. 6 Computer Output for Generalized Sliding Wedge



KEY

1, 2, 3, 4 — COORDINATE POINTS

1 2 — SOIL WEDGES

(12) (13) — WATER WEDGES

γ_{PCT}	C_{PSF}	ϕ	WEDGE NOS.
120	0	25°	1,2,4,7
58	0	25°	3,5,8
32	600	0	6,9,10,11
62	0	0	12,....,18

FIGURE 7- NAVDOCKS EXAMPLE

DRNEVICH SLOPE STABILITY BY SLIDING WEDGE

NUMBER OF WEDGES OR BLOCKS=18

WEDGE NO.	X(1,J)	Y(1,J)	X(2,J)	Y(2,J)	X(3,J)	Y(3,J)	X(4,J)	Y(4,J)	T(J)	G(J)	C(J)	PHI(J)	E(J)
1	0.	40.	0.	40.	6.	40.	6.	30.	1	120.0	0.	25.0	0.0
2	6.	30.	6.	40.	12.	40.	12.	30.	0	120.0	0.	25.0	0.0
3	6.	30.	6.	30.	12.	30.	12.	20.	1	58.0	0.	25.0	0.0
4	12.	30.	12.	40.	24.	40.	24.	30.	0	120.0	0.	25.0	0.0
5	12.	20.	12.	30.	24.	30.	24.	20.	0	58.0	0.	25.0	0.0
6	12.	20.	12.	20.	24.	20.	24.	8.	1	32.0	600.	0.0	0.0
7	24.	30.	24.	40.	66.	20.	66.	20.	0	120.0	0.	25.0	0.0
8	24.	20.	24.	30.	66.	20.	66.	20.	0	58.0	0.	25.0	0.0
9	24.	8.	24.	20.	66.	20.	66.	8.	0	32.0	600.	0.0	0.0
10	24.	8.	24.	8.	66.	8.	66.	4.	1	32.0	600.	0.0	0.0
11	66.	4.	66.	20.	82.	20.	82.	20.	1	32.0	600.	0.0	0.0
12	6.	30.	6.	30.	12.	30.	12.	20.	1	62.0	0.	0.0	0.0
13	12.	20.	12.	30.	24.	30.	24.	20.	0	62.0	0.	0.0	0.0
14	12.	20.	12.	20.	24.	20.	24.	8.	1	62.0	0.	0.0	0.0
15	24.	20.	24.	30.	66.	20.	66.	20.	0	62.0	0.	0.0	0.0
16	24.	8.	24.	20.	66.	20.	66.	8.	0	62.0	0.	0.0	0.0
17	24.	8.	24.	8.	66.	8.	66.	4.	1	62.0	0.	0.0	0.0
18	66.	4.	66.	20.	82.	20.	82.	20.	1	62.0	0.	0.0	0.0

WEDGE NO.	DRIVING FORCE	RESIST. FORCE	WEIGHT	A DEG	B DEG
1	6.000E+03	3.568E+03	3.60E+03	59.0	0.0
2	0.000E-99	0.000E-99	7.20E+03	0.0	0.0
3	1.490E+04	8.862E+03	1.74E+03	59.0	0.0
4	0.000E-99	0.000E-99	1.44E+04	0.0	0.0
5	0.000E-99	0.000E-99	6.96E+03	0.0	0.0
6	2.366E+04	1.440E+04	2.30E+03	45.0	0.0
7	0.000E-99	0.000E-99	2.52E+04	13.4	25.5
8	0.000E-99	0.000E-99	1.22E+04	0.0	13.4
9	0.000E-99	0.000E-99	1.61E+04	0.0	0.0
10	5.352E+03	2.543E+04	2.69E+03	5.4	0.0
11	-.410E+04	1.920E+04	4.10E+03	45.0	0.0
12	3.100E+03	0.000E-99	1.86E+03	59.0	0.0
13	0.000E-99	0.000E-99	7.44E+03	0.0	0.0
14	1.190E+04	0.000E-99	4.46E+03	45.0	0.0
15	0.000E-99	0.000E-99	1.30E+04	0.0	13.4
16	0.000E-99	0.000E-99	3.12E+04	0.0	0.0
17	4.712E+03	0.000E-99	5.21E+03	5.4	0.0
18	-.794E+04	0.000E-99	7.94E+03	45.0	0.0

SUM WT = 1.68E+05 SUM RESIST.=7.15E+04 SUM DRIVE=5.76E+04 FACTOR OF SAFETY = 1.241

ABOVE RESULTS ARE FOR FAILURE SURFACE SELECTED BY THE USER
THE SURFACE SELECTED MAY NOT BE A CRITICAL SURFACE

Fig. 8 Computer Output for NAVDOCKS Example correction is made, the factor of safety then becomes 1.24. The systematized approach of the generalized wedge method proposed herein would tend to eliminate errors such as that noted above.

USE OF GENERALIZED WEDGE FOR LATERAL PRESSURE

Because of the general formulation of this method, it can be used for other related problems such as the stability of retaining walls against sliding failure and in determining both the magnitude and approximate distribution of pressure against such walls. For the case of sliding stability, the wall itself can be simulated by wedges and blocks. The unit weight is that of the wall construction material and the shear strength along the failure surface beneath the wall is given in terms of adhesion and friction factors associated with the contact of two dissimilar materials.

This method can also be used to determine pressures and pressure distributions wherever plastic equilibrium conditions exist. For example, the force acting on a retaining structure or a wall can be determined with reasonable consideration given to the zone of excavation, backfill surface, and backfill materials. In simple, noncritical cases, a triangular pressure distribution can be assumed. For more important and complex situations, the method recommended in Article 31 of Terzaghi and Peck[6] may be used. The method is based on the fact that for a state of plastic equilibrium, every point on the back of a retaining structure represents a foot of a potential surface of sliding. The generalized wedge method can be used to obtain the forces for a series of these failure surfaces and consequently, the pressure distribution.

METHOD LIMITATIONS AND CAUTIONS FOR USE

It would not be fair to continue without pointing out some of the limitations and deficiencies of the generalized sliding wedge method. This method considers only the static equilibrium in the vertical and horizontal directions of the overall system of blocks and wedges. The overall moment equilibrium as well as the moment equilibrium of the individual blocks or wedges may not be satisfied. It is assumed that each surcharge block or wedge transmit forces only to the wedge or block beneath it. Consequently, this method is probably best suited to cases where the failure surface is long and rather shallow. The accuracy for other situations has not yet been fully explored.

Finally, this method gives driving forces, resisting forces and factors of safety for failure surfaces chosen by the user. It does not seek out the most critical surface. However, the computer program has been written for conversational mode and the user can quickly change parameters or coordinates of any or all wedges or blocks as he chooses.

CONCLUSION

A generalized method for determining slope stability and earth pressures based on the sliding wedge concept has been presented. The procedure is outlined to give a systematized approach. Basic concepts are extended to handle the effective stress condition where excess pore pressures may exist. The method appears to give reasonable results for the cases cited. It should be quite useful for a number of problems commonly occurring to the practicing civil engineer.

ACKNOWLEDGMENT

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LATERAL PRESSURES AND PRESTRESSED TIE-BACK WALLS

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INTRODUCTION

The use of tie-back systems for retaining walls of deep excavations has been rapidly adopted by American contractors over the past decade. Originating in Europe for use at sites where conventional internal bracing was not attractive, the method has since emerged in this country as a rapid, economical method of providing an uncluttered deep excavation. Walther[1] has described an early application of the concept for a railroad station in Germany. In that project, two rows of tie-backs were used to retain a concrete wall which was supported vertically on bearing piles. Nunes and Costa[2] have discussed several applications of tie-back walls used in Brazil dating back as early as 1957. In one application, vertical ties were placed at the heel of a retaining wall to provide resistance to overturning. Nordin[3] has discussed the application in Sweden using hollow drill rods as the tie-back anchors. The system was first introduced in this country by White[4] in 1961. Projects in Los Angeles[5], Montreal[6,7,8], Washington, D. C.[9], Ottawa,[10] San Francisco,[11, 12, 13, 14, 15] Houston,[16] Boston,[17] New York,[18] Pittsburgh,[19] Buffalo,[20] and England[2] have been reported in more recent literature, with the bulk of the discussions being concerned primarily with construction techniques.

McRostie and Schriever,[10] who discussed the Ottawa project, also studied the effects of creep and frost pressures on the tie-backs. The various types have included (1) sheet piling, (2) soldier piles with wood lagging, (3) concrete, and (4) slurry trench walls. The tie-backs are driven H-piles, grouted tendons, and grouted drill rods. Stagg and James[22] have discussed the possible use of cables with packed granular soils rather than grout. This would be attractive in situations where it is mandatory to remove the tie-backs.

D'Appolonia, et al.[23] have described the application of the tie-back system to retain a slope of colluvial material in Weirton, West Virginia. The methods and results at the West Virginia site,[23] initiated the development of a rational design method which is discussed herein, along with applications of the method at two sites in Buffalo, New York. In addition to the usual considerations of cost and ease of construction, movement of the ground surface adjacent to the retaining wall was also of primary concern for both of the reported projects. Qualitive relationships between applied prestress tie-back forces and surface deformation have been developed and substantiated by instrumentation. The method is applicable to most tie-back problems, whether ground movements are a primary or secondary design concern.

DESIGN CONCEPTS

Cases where movements of the adjacent ground surface are of primary concern pertain to the situations where existing structures, machinery, etc., are located within a perimeter zone which would be influenced by the deep excavation. The zone of influence typically extends behind the wall a distance equal to 1 to 1.5 times the depth of the excavation as illustrated in Figure 1a. Consistent with normal foundation engineering terminology, this case is referred to herein as the "at-rest" design case as it would be desirable to make the excvation and maintain the soil in an at-rest condition. In other words, "keeping the soil behind the wall from knowing the excavation is being made." The "active" case, as illustrated in Figure 1b, applies where the cost and time advantages of the tie-back system are the governing criteria rather than displacement of the adjacent ground surface. The design procedure is similar for both cases with the main differences being the design soil pressures and the magnitude of prestressing force applied to the tie.

"At-Rest" Design Case: The effect of prestressing on the wall deflection can be examined by considering the change in the stress state of the in-situ soil during the entire construction sequence in relation to the stress-strain characteristics of the soil. Figures 2a through 2e show typical conditions pertinent to a deep excavation in soil during particular stages of construction. Each figure includes the stage of excavation and the distribution of soil pressures acting on the wall. Also shown in each figure are four elements of soil--Nos. 1, 2 and 3 located at various depths outside the excavation and No. 4 located near the toe in front of the wall.

Figure 3 shows qualitatively using the stress-path method[24] the stress changes which occur at the various soil elements during the process of construction for the case when the tie-backs are prestressed for at-rest pressures. The letter designations, A through E, shown on the curves correspond to the respective construction sequence shown in Fig. 2. In determining the stress paths in Fig. 3, it has been assumed that the vertical and horizontal directions are the principal directions throughout the construction procedure. This assumption is reasonably valid for the at-rest design case because movements are small, and large friction forces do not develop at the wall-soil interface.

Point A represents the initial soil stress at each of the elements, Nos. 1 through No. 4. The initial points for elements No. 3 and No. 4 are common as these elements are at the same depth. During the driving of the sheet piling, only the horizontal stress acting on each of the elements is assumed to increase as indicated by stress path AB. The stress path BC indicates the reduction in horizontal stress behind the wall due to the initial excavation stage to the tie-back level. As shown, the magnitude of this change diminishes with depth and has practically no effect on stress condition at element No. 3. The stress path BC for element No. 4 reflects the decrease in vertical stress caused by the excavation. In going from C to D, the tie-back is installed and prestressed, resulting in increased horizontal stresses behind the wall. The effect is most pronounced in the area of the tie-back, and diminishes quickly with depth.

The last construction stage involves the final excavation to grade. During this stage, the flexibility of the wall allows an outward movement which causes a decrease in the horizontal stress behind the wall as indicated by the stress path DE for soil elements Nos. 1, 2 and 3. Also during the final excavation, the vertical stress in the soil in front of the wall is decreasing, while the horizontal stress is increasing at the embedded portion of the wall. The stress path for this condition is indicated by D_4E_4 in Fig. 3.

<u>"Active" Design Case:</u> Figures 4a through 4e show the condition at the wall for particular stages of construction for the active design case. A single waler system is illustrated for which a triangular distribution is applicable. Conceivably, a greater number of walers would tend to cause the pressure distribution to change from triangular to trapezoidal. Figure 5 shows the corresponding changes in the stress state. States A through C are identical to those previously described for the at-rest case. However,

in this case, the tie-back is not prestressed* and no appreciable changes in soil stress occur during its installation. During the final excavation, the wall deflects outward from its original location and the horizontal stresses behind the wall diminish to the active failure condition as indicated by stress path DE in Fig. 4 for elements Nos. 1 and 2. Element No. 3 lies outside of the failure wedge and reduction of stress in this element does not reach the failure condition. As before, the vertical stress decreases, while the horizontal stress increases in the soil at the toe (Element No.4) during final excavation.

It should be noted that two simplifying assumptions were made in the "active" design case. The change in principal stress directions due to friction forces which develop as the active soil wedge moves vertically with respect to the wall is ignored. Also the variation in active pressure distribution caused by arching of the soil as the wall deflects is not taken into account. Neither of these considerations appear to be critical to design, particularly because a prestress load is normally specified which prevents a full actual failure.

<u>Soil Stress-Strain Characteristics</u>: In order to relate the magnitude of wall deflection to the stress paths shown in Figs. 3 and 5, it is necessary to consider the stress-strain characteristics of the soil. The curves shown in Figs. 6a and 6b represent typical stress-strain relationships (plotted as axial strain vs. the deviator stress) for a granular material as obtained from drained triaxial tests. The various curves illustrate the relationship at different confining pressures. The slope, at any point on the curves,

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*In theory, no prestress force is required, but it is recommended that a nominal amount of prestress be applied for reasons discussed in the following sections.

is the tangent modulus of elasticity, E, for the given stress condition at that point in the soil. The important characteristic is the large reduction in the moduli once a "yielding" strain has been reached. The "K_o curve" represents the initial stress condition in the soil prior to construction.

The shaded area in Fig. 6a represents the range of stress conditions for the "at-rest" design case. While the stress conditions fluctuate to either side of the "K_o" line for this case, they always remain at the steep slope or high modulus portions of the curves and it follows that the strains and deflections will be small. Similarly, the shaded area in Fig. 6b represents the range of stress conditions for the "active" design case. For this case, the stress conditions vary over a far greater range into the regions of relatively small moduli. Thus, much larger strains occur within the soil resulting in larger deflections of the wall and adjacent soil. Therefore, the effect of prestressing the tie-back to loads which effectively resist "at-rest" soil pressures is to restrict positively movement of the ground surface adjacent to the excavation. The same qualitative reasoning is applicable to cohesive soils and walls with other types of toe support. In practice, the magnitude of the prestressed load is taken to 100 percent of the load required to balance statically an "at-rest" pressure distribution as shown in Fig. 2e. In addition, an overload of 10 percent* of the design load is temporarily applied to assure the tie-back integrity. A minimum design safety factor of 1.5 is used for design of the tie-back, whether it be grouted-in drill rods, driven H-piles, or of

*An overload of ten percent is usually adequate when the retaining wall is to be used for a short construction period, and the tie-back properties are confidently known. For retaining walls required for long periods of construction or in areas where the tie-back characteristics are variable, a greater overstress is recommended.

other type. Normally, representative tests to failure are conducted to verify design parameters.

Theoretically, no prestress load is applied to the wall when the active design case is used. Significant movements will occur with magnitudes similar to those associated with conventional internal bracing systems or anchored bulkheads. However, in practice, a prestress load of 50 percent of the load required to statically balance an active pressure distribution similar to that shown on Fig. 4e is specified. Each tie-back is initially loaded to 110* percent of the required design load and then backed off to 50 percent. The purpose of this procedure is (1) to check the adequacy of each tie-back and connection, (2) to provide a more uniform distribution of the load between the ties, waler and sheetpile (or soldier pile) system along the length of the excavation and (3) to diminish to effect of inherent eccentricities in the connection system.

Subsequent sections discuss the application of these design concepts to two sites having different soil conditions. One design utilized a soldier-pile, wood-lagging wall with driven H-piles as tie-backs, while the other used a sheetpile wall with grouted-in drill rods as tie-backs. The site for the soldier-pile, wood-lagging system consisted of a thick sand stratum, while the site for the sheetpile wall system consisted of a relatively thin stratum of silty clay overlying bedrock.

CASE HISTORY NO. 1

The first case history deals with the excavation for a 28-story building constructed in the business district of Buffalo, New York in 1968 through silty sands and gravels typical of the Buffalo area. The excavation

reement was obtained between the measured elastic deformation for a triangular
in friction distribution varying from zero, at the butt, to a maximum at the
p. Corresponding friction parameters were established based on the triangular
istribution at the ultimate failure load.

The second phase of the work consisted of designing the soldier-piles ncluding their size and depth of embedment. Reasonable spacing and sections ere chosen and analyzed as laterally loaded piles using the work proposed y Broms.[25] The depth of excavation varied over the site as did the surcharge oadings. An "at-rest" pressure design was adopted only where movements could ave caused distress to adjacent structures or utilities. The required soldier-ile embedment length was equal to about 0.3 times the depth of excavation ut a final design embedment of 0.45 times the depth of excavation was used n order to provide an adequate safety factor, as well as to provide for ossible overexcavation at spread footing locations. In some instances, the contractor elected to maintain only the design embedment when hard driving indicated better than expected soil conditions. The soldier-pile spacing was maintained at 6 feet using a 12BP53 section for cuts less than 25 feet deep and a 14BP73 section for deeper cuts.

All tie-backs were 12BP73 H-piles spaced at 12 foot centers (at alternate soldier-piles) and designed to be driven at 45 degrees. However, due to mechanical difficulties with the hammer, the contractor was forced to drive the tie-backs at 50 degrees from the horizontal. The prestress forces were altered accordingly. The length of the battered tie-back piles were determined on the basis that the ultimate skin friction on the portion of the pile beyond the theoretical active slip plane was equal to 1.5 times the design load. The length of the tie-back piles varied from 33 feet to about was made to depths varying from 21 feet to 40 feet which were kept open throughout the construction period. The site was bounded on three sides by heavily traveled streets. Utility lines were located under the streets and high-rise buildings existed on opposite sides of the streets.

The foundation contractor elected to use a tie-back system, over conventional bracing, in order to permit rapid excavation and minimal interference with the construction of spread footings. The wall was designed for vertical soldier-piles with wood-lagging and battered H-pile tie-backs.

The initial phase of the work consisted of obtaining test borings and performing pull-out tests on driven battered H-piles. Typical soil conditions, standard penetration blow-counts, pile driving resistance and the test pile arrangement are shown in Fig. 8. A total of four pull-out tests were conducted on driven H-piles and substantial agreement was in evidence. In each test, the butt movement was measured with dial gages, and the tip movement was measured with tell-tale rods. The right side of Fig. 9 shows a typical load-deflection curve for the test piles. The difference between the butt and tip movements is a measure of the elastic deformation of the pile as shown on the left of Fig. 9. The distribution of skin friction along the piles was not measured directly, therefore, it was necessary to calculate the elastic deformation of the pile for several assumed friction distributions for comparison with the measured values. The distributions considered include no friction, uniform friction, triangular and parabolic friction distributions. The "no friction" case was considered a possibility because two of the four test piles had cement grout plugs at their tips. The grout plugs were found to contribute a negligible amount of pull-out resistance and were not adopted in the final design. Satisfactory

CASE HISTORY NO. 2

The second case history concerns a deep excavation for a series of machines on a rolling mill line at a steel plant in Buffalo, New York. The excavation, approximately 70 x 150 feet in plan and 21 feet deep was in the midst of existing mill lines where operation could not be interrupted. The proximity of the existing lines ruled out the possibility of an open-cut and a braced excavation would have been both costly and time-consuming. Also, the owner had encountered difficulties with previous braced excavations under similar site conditions in the form of lateral and vertical movement of adjacent machinery. Therefore, the design goals were to provide an excavation that would minimize movements of surrounding machinery and still be attractive with respect to both cost and time of construction.

Figure 13 shows the general site conditions and the outline of the excavation. Fig. 14 is a typical section through the wall. In general, the excavation extended about 21 feet below floor level. The wall was designed using ZP32 sheetpile sections driven to rock with grouted-in high-strength steel drill rods for the tie-backs. Only one row of ties was required with a horizontal spacing of 3.5 or 5.5 feet, depending on the particular pressure conditions. An at-rest pressure design, with a 100 percent prestressing load, was used where machinery abutted the walls. An active pressure design, with a 50 pereent prestressing load, was used at the less critical areas. On this particular project, an overload of 50 percent was applied and held for about five minutes and then reduced to the design pressure load.

An interesting detail of this particular design was the development of adequate toe support. The depth from the bottom of excavation to top of rock at aprticular areas around the perimeter of the excavation was only

60 feet. Two rows of ties spaced at the third points on the open face of the excavation were used on the 37-foot cut along Main Street, as illustrated in Fig. 10, and along the 20-foot cut adjacent to the existing bank.

Wood-lagging, two inches thick, was initially cut in the field to fit between the webs of adjacent soldier piles. However, during the course of construction, the contractor found it more economical to use an anchor system on the flange of the piles and cut all of the lagging to the same length. Arching of the soil between the soldier piles was readily apparent at the site; and if not for minor sloughing of the sand and gravel, no lagging would have been required.

The general construction procedure involved making an initial excavation of about 10 feet. At this elevation, the soldier piles were driven around the perimeter of the excavation and the top row of tie-backs and wäles were installed. The tie-backs were prestressed to 50 percent in active pressure design areas, and 100 percent of the design load in at rest pressure design areas, plus a 10 percent temporary overload. The stressed tie-backs were welded to the wales through a series of cover plates and shims as illustrated on Fig. 12. The installation of the lower ties in the areas requiring two rows was similar to that for the upper ties.

The cost of the wall to the contractor ($8.00 to $10.00 per square foot in 1966) was slightly higher than estimated for a conventional raker system. However, the excavation schedule was shortened by approximately thirt percent and a considerable savings was realized in the actual earthwork and the spread footings.

orced through the hollow drill rod was used to wash the cuttings from the ole to the surface. Upon completion of the hole, the wash water was replaced ith a 1 to 1 (water to cement) grout mixture which was allowed to flow until t appeared at the ground surface. The three-inch casing was pulled during routing operations, but the hollow drill rod and the bit were never removed rom the hole once drilling of the rock had commenced. Because the drill bits ere left in place, a less expensive soft rock bit was used in lieu of a more xpensive bit normally required for a hard shale. The design prestress load lus a 50 percent overload which was subsequently released, were applied and locked in the rod using the connection shown in Fig. 15.

about six feet through soft-silty clay, which did not have sufficient streng to develop the required toe support. Also, past experience at the plant had shown that seating the sheetpiles into rock was not possible. Therefore, to support was provided with 10-3/4 inch diameter drilled-in pipe piles spaced at 9-foot centers. The addition of these toe piles added little to the cost of the wall because they serve as permanent vertical support for the heavily loaded concrete perimeter walls of the pit. The drilled-in pipe piles were constructed inside a 14 inch diameter casing, which was driven and seated into rock after the sheet piling had been driven. A 10-3/4 inch diameter steel casing, 11 feet in length, was dropped and grouted into the socket as shown in Fig. 14.

The reinforcing steel for the concrete wall was designed to withstand the soil pressures with rock ties in-place or cut at some future date. The sheetpiling was tied to the concrete with shear studs to permit use of the sheetpile wall as tension reinforcing steel in the cantilevered condition. Calculations indicated that, in this instance, this scheme was more economical than using additional reinforcing bars.

The first phase of construction consisted of a perimeter trench excavat of 8 feet to gain head room for pile-driving leads. Adjacent foundations were all founded below this elevation and, therefore, no appreciable movements occurred. The sheetpiling was driven to rock and the tie-back-waler system installed. The ties were 1.5-inch O.D., high-strength steel drill rods having continuous external threads and a 0.625-inch center hole. After three inch casing had been driven at a 45^{o} angle, seated into rock, and washed, a rotary drill was used to advance the hole about 18 feet into the shale. Water ıcrease to the design load. First, it was not possible to install all of the ıges prior to jacking and, therefere, an accurate initial reading was not available and the true load in the tie was not known. Secondly, the magnitude nd distribution of active earth pressure used in design was different o that which actually occurred. A triangular pressure distribution was ssumed in the analysis. Finally, observation of the in-situ soil onditions during excavation indicated that, in general, the soil was denser han indicated by the borings, and, therefore, the design soil parameters vere somewhat conservative.

Figure 17 shows similar plots for a 21 foot cut designed for at-rest earth pressures.

The load in the tiebacks for this portion of the wall tended to decrease asymptotically as the excavation proceeded. The decrease in load was attributed to the small deflection of the wall toward the excavation which caused the lateral earth pressures to reduce from the K_o (at rest) to the K_a (active) condition. The most interesting observation in Fig. 17 is the small deflections which occurred for the "at-rest" design case; the ratio between the deflections shown in Figs. 16 and 17 being 5 to 10.

These data from the instrumentation at Case History No. 1 are particularly gratifying as they are consistent with what was expected from the design method used. The results illustrate the effect of the two design cases and point out the value of instrumentation on a project of this type.

DISCUSSION OF INSTRUMENTATION AND WALL PERFORMANCE

Instrumentation including slope indicators, vibrating wire strain gages and offset line surveys were used on the above projects to monitor the movement of the walls and the magntidue of the tieback loads. These instruments were used to verify the validity of the design dissumptions as well as to warn of impending dangerous conditions. Gross movements and/or failures at either site would have been extremely costly to the owners and contractor when compared to the relatively small cost of the instruments.

At the high rise building site, Case History No. 1, two vibrating wire gages were installed on the web of six H-pile tie-backs to measure changes in tie-back load which occurred during the construction period. Three slope indicators were also installed adjacent to the strain gages to measure the lateral deformation of the wall. The actual location of these instruments is shown in Fig. 7. Figure 16 shows the change in the upper and lower tie-back forces and the lateral movement of the wall at each tieback level for a 37 foot cut designed for active pressures. The load in both of the ties increased with time in an asymptotic manner, and the wall defected in the direction of the excavation. Since the tie-backs for the active design case were prestressed to about 50 percent of the calculated desi load, an increase in load of about another 50 percent should have occurred. Although this trend is evident, the load increased only about 15 to 25 percent. The deflections were of the order expected for a flexible wall and, as expected, only minor cracking occurred at the street elevation.

There are several possible explanations why the tie-back did not appear f the wall moved inward. Movements varied from about 0.5 inches at the ottom of the excavation to about 1.25 inches at the top of rock. As reviously discussed, toe support for this wall was derived from drilled-in-ipe pins. The movements probably stemmed from the deformation of the soil ap between the sheetpile wall and the pins caused by misalignment and the leformation of the pins. These movements were not critical on this particular project, but conceivably could cause problems elsewhere.

The tieback load was not monitored as in Case History No. 1, but was checked at four ties suspected of fracture due to large eccentricities. An increasing load was applied to the tie-back with a jack until the hold-down nut could be twisted by hand. One of the ties, No. 71, was found to be fractured immediately, while the remaining three were observed to have a load within one ton of the original prestress load of 36 tons.

The instrumentation for the steel mill excavation, Case History No. 2 consisted of offset survey lines to measure the movement of the top of the tie-back wall and ten slope indicators. No gages were placed directly on the tie-backs, but the load was checked on several rods using a jack to relieve the load on the bearing plate.

Figure 13 shows the location of the survey points and slope indicators. The survey readings were measured to the nearest 0.01 feet. In all cases, the outward movement decreased after the post-tensioning operation. In zones where at-rest pressure designs were used, the data indicated inward movements ranging from 0.06 feet to 0.12 feet prior to jacking and outward movements of 0.01 feet to 0.07 feet after jacking and excavation. Similar movements occurred for the active design zones, but, in general, were of lower magnitude. All points in at-rest design zones ended up outside of the original excavation line, while those in active zones tended to oscillate.

Although it is not readily apparent, the results are consistent with those observed for Case History No. 1. In general, the active design zones had a greater net movement toward the excavation. Overall movements were great for the at-rest design case, primarily because of the much greater surcharge pressures in these zones.

As shown on Fig. 13, the slope indicators were located on the wall itself and three to five feet back from the wall. The slope indicator casing and sand filler were inserted into an angle which had been welded to the sheet-pile prior to driving. Since slope indicators actually measure only a change in slope, horizontal "control" was obtained by survey. The most meaningful data obtained from the slope indicator suggested that the embedded portion

SUMMARY AND CONCLUSIONS

The use of tie-back walls for retaining deep excavations has been adopted many American contractors as a means of providing an open excavation, free from ıventional rakers and bracing. The cost of a tie-back wall is about equal to or ıater than conventional bracing systems, depending on site conditions, but ›stantial savings are usually realized in excavation and construction of ındations because of the open working area. Although numerous articles ve appeared in the literature discussing the approach, none have suggested sign methods or techniques and few have given performance data.

This paper presents a rational method for designing tie-back wall stems for deep excavations with two design cases relating to the allowable vement of the soil behind the wall. The method has been applied to two latively different projects. Quantitative field measurements tend to pport the design method which appears to be adaptable to most soil conditions d construction procedures.

Detail of Tie-Back to Wale Connection: No detrimental movements or distress were in evidence for either of the two case histories. The only troublesome detail at both projects was the connection of the tie-back to the wale. As illustrated in Fig. 11, the H-pile-to-wale connection design proved to be adequate, but a few isolated welds did fail during construction. After modification of the field welding procedures by the contractor, no additional failures occurred. At the steel mill site, the connection was less complicat in concept, but more troublesome. As shown in Fig. 15, one-half of a drill rod connection sleeve was originally used as the nut for the tie-back to wale connections. Seven out of a total 91 ties were misaligned during installation, and a failure of either the drill rod or the nut occurred in each case. The misalignment caused large secondary bending moments at the tie-to-wale connection, resulting in tie-back failures. In each of these cases, the rod and nut were removed and replaced using shims and aligned to alleviate most of the eccentricities in the system. As pointed out by Sowers and Sowers[26] and as illustrated by these two case histories, details of this type rather than the accuracy of pressure distributions and the like may well be the mechanism that could precipitate a large-scale failure.

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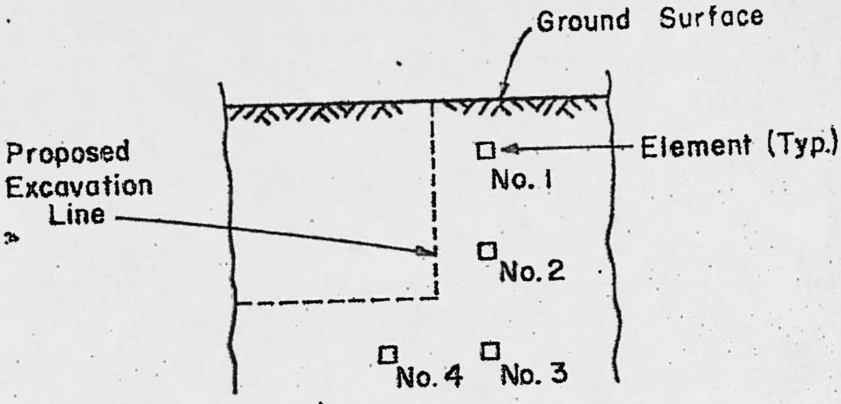
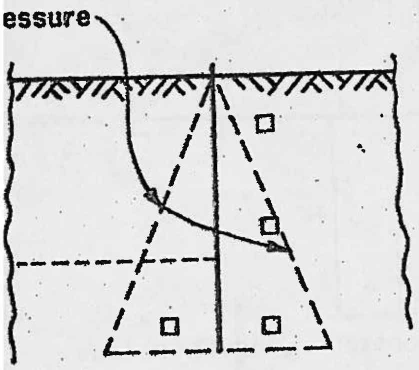


FIG. 2a. - INITIAL CONDITION



2b. - SHEET PILING OR SOLDIER'
S INSTALLED

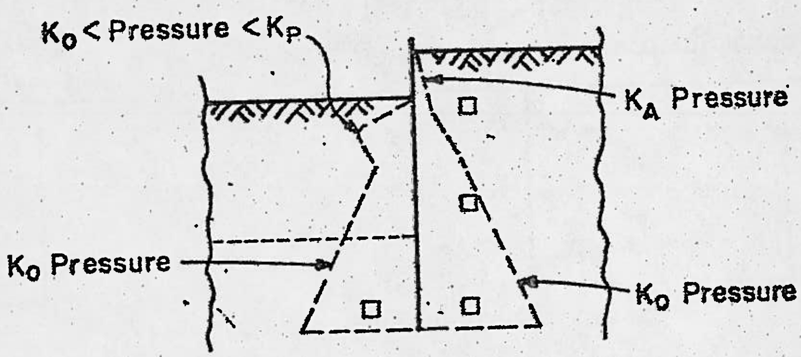
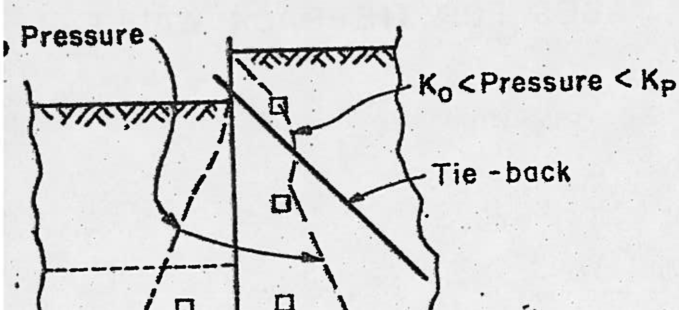


FIG. 2c. - INITIAL EXCAVATION



. 2d. - INSTALLATION AND PRE-
RESSING TIE-BACK

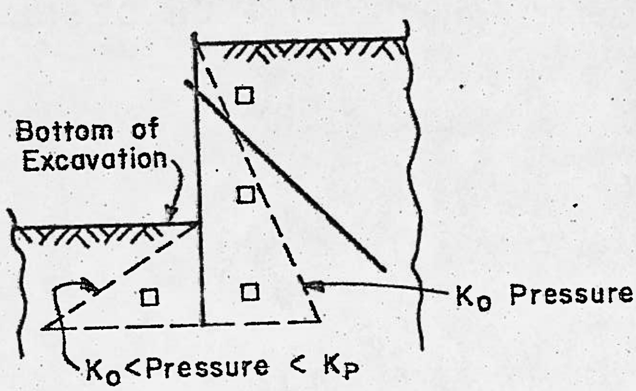


FIG. 2e. - FINAL EXCAVATION

FIG. 2 - CONSTRUCTION SEQUENCE FOR TIE-BACK RETAINING WALL-"AT REST" DESIGN CASE

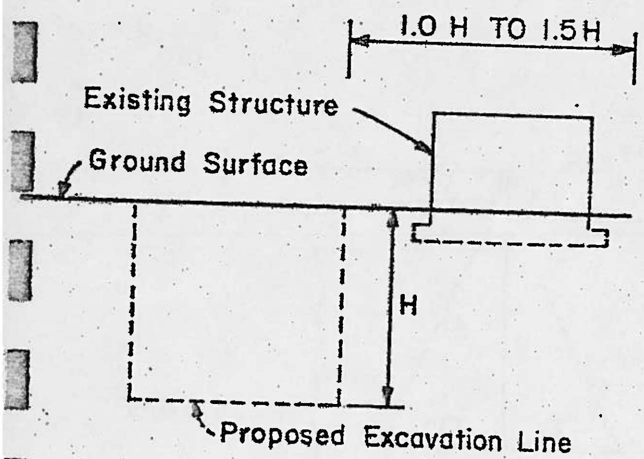


FIG. 1a. – TYPICAL "AT REST" DESIGN CASE

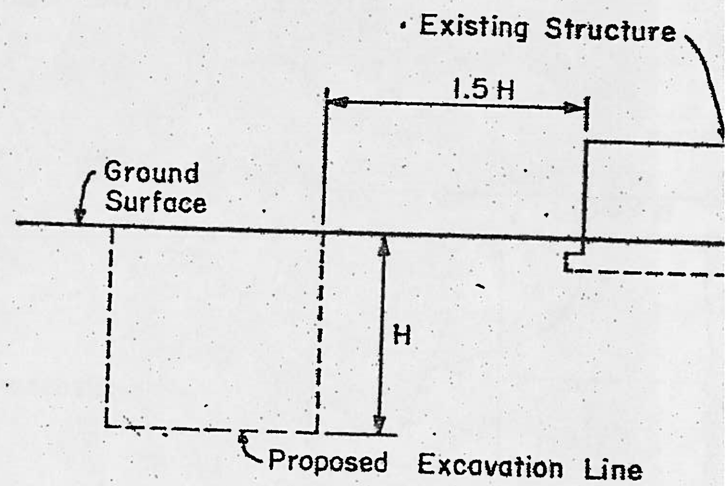


FIG. 1b. – TYPICAL "ACTIVE" DESIGN CASE

FIG. 1 – SCHEMATIC OF DESIGN CASES FOR TIE-BACK WALLS

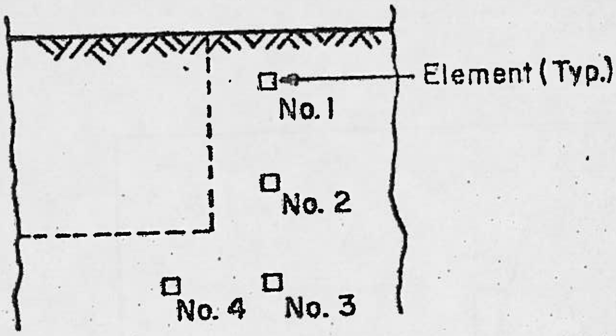
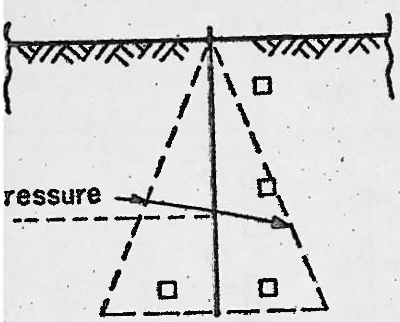


FIG. 4a. - INITIAL CONDITION



4b. - SHEET PILING OR SOLDIER
ES INSTALLED

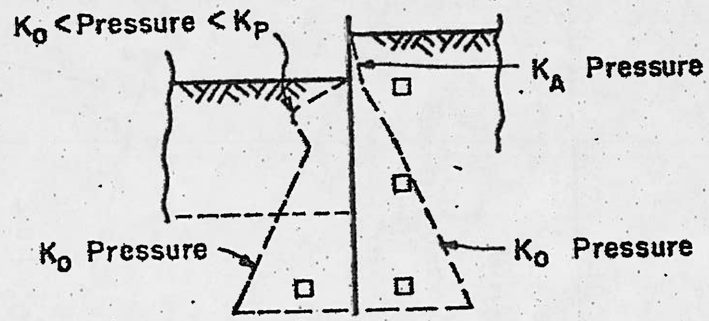
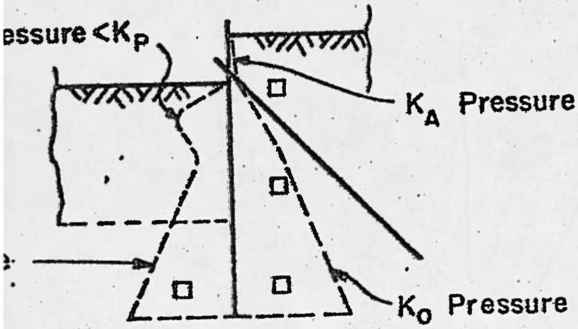


FIG. 4c. - INITIAL EXCAVATION



4d. - INSTALLATION OF TIE- BACK

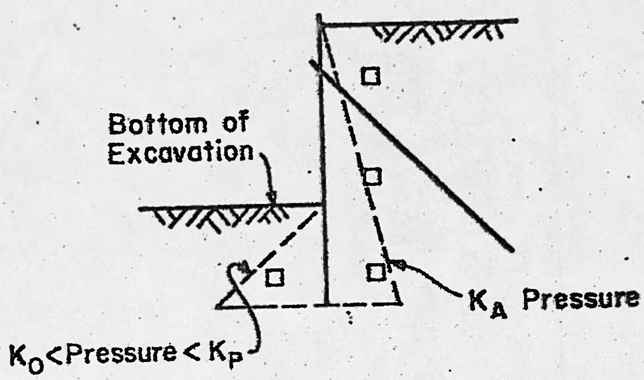


FIG. 4e. - FINAL EXCAVATION

FIG. 4 - CONSTRUCTION SEQUENCE FOR TIE-BACK RETAINING WALL - "ACTIVE" DESIGN CASE

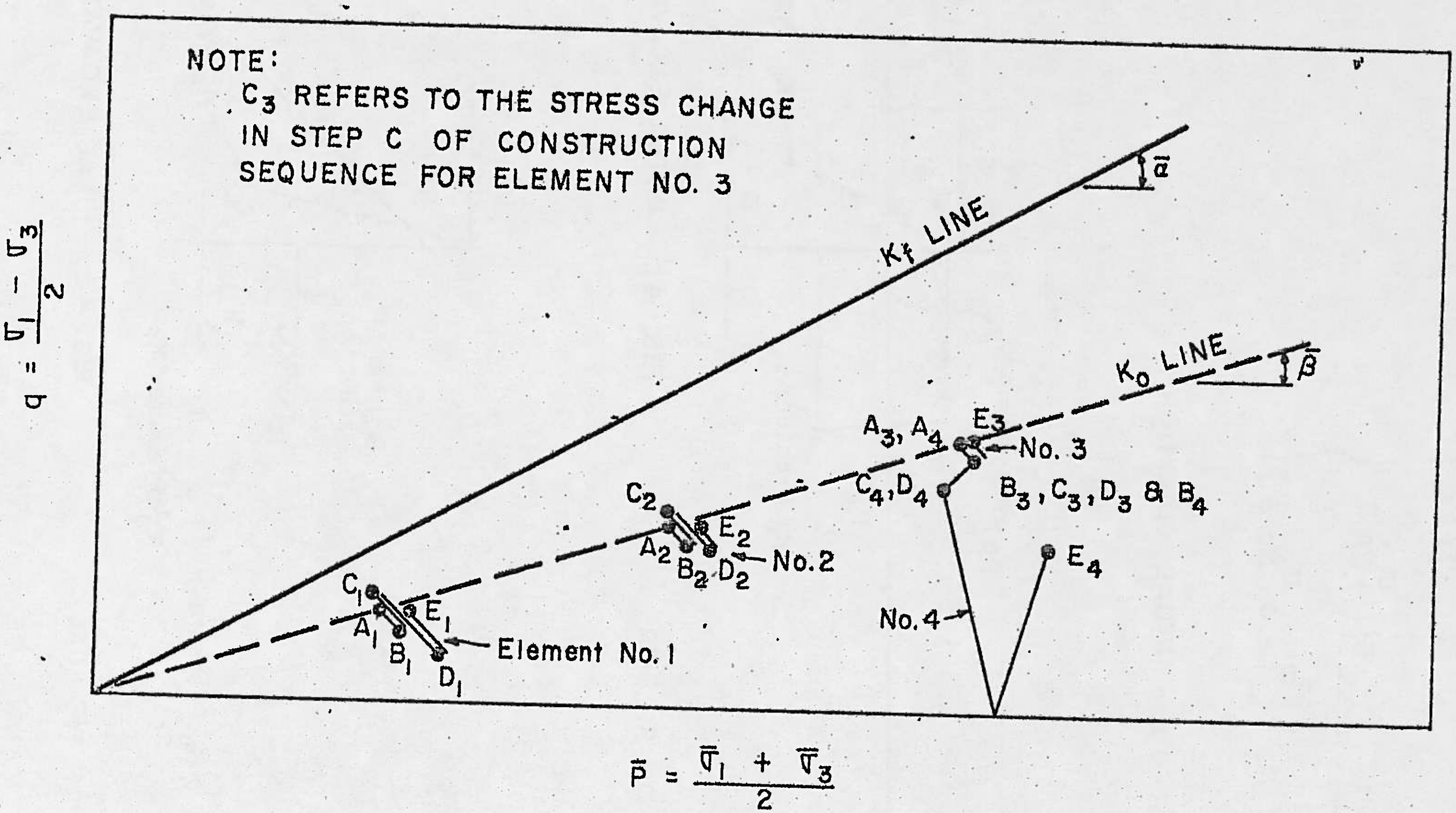
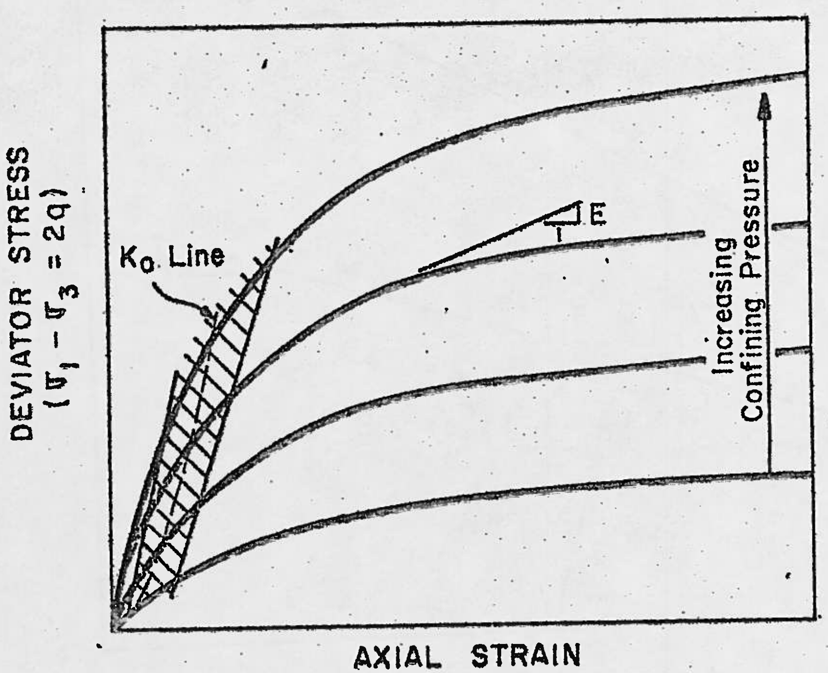
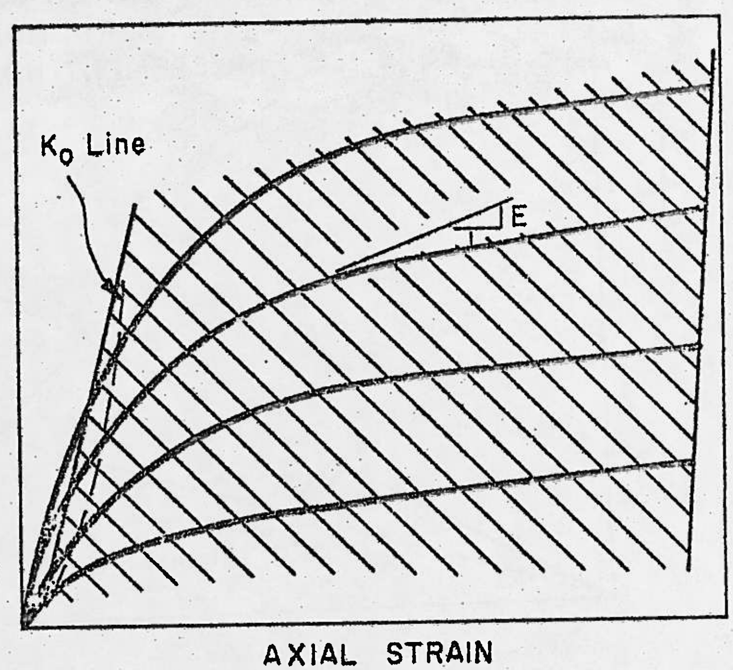


FIG. 3 - STRESS CHANGES DUE TO CONSTRUCTION SEQUENCE FOR "AT REST" DESIGN CASE



a.) RANGE OF MODULII FOR "AT REST" DESIGN CASE



b.) RANGE OF MODULII FOR "ACTIVE" DESIGN CASE

FIG. 6 - RANGE OF MODULII FOR "AT REST" AND "ACTIVE" DESIGN CASES

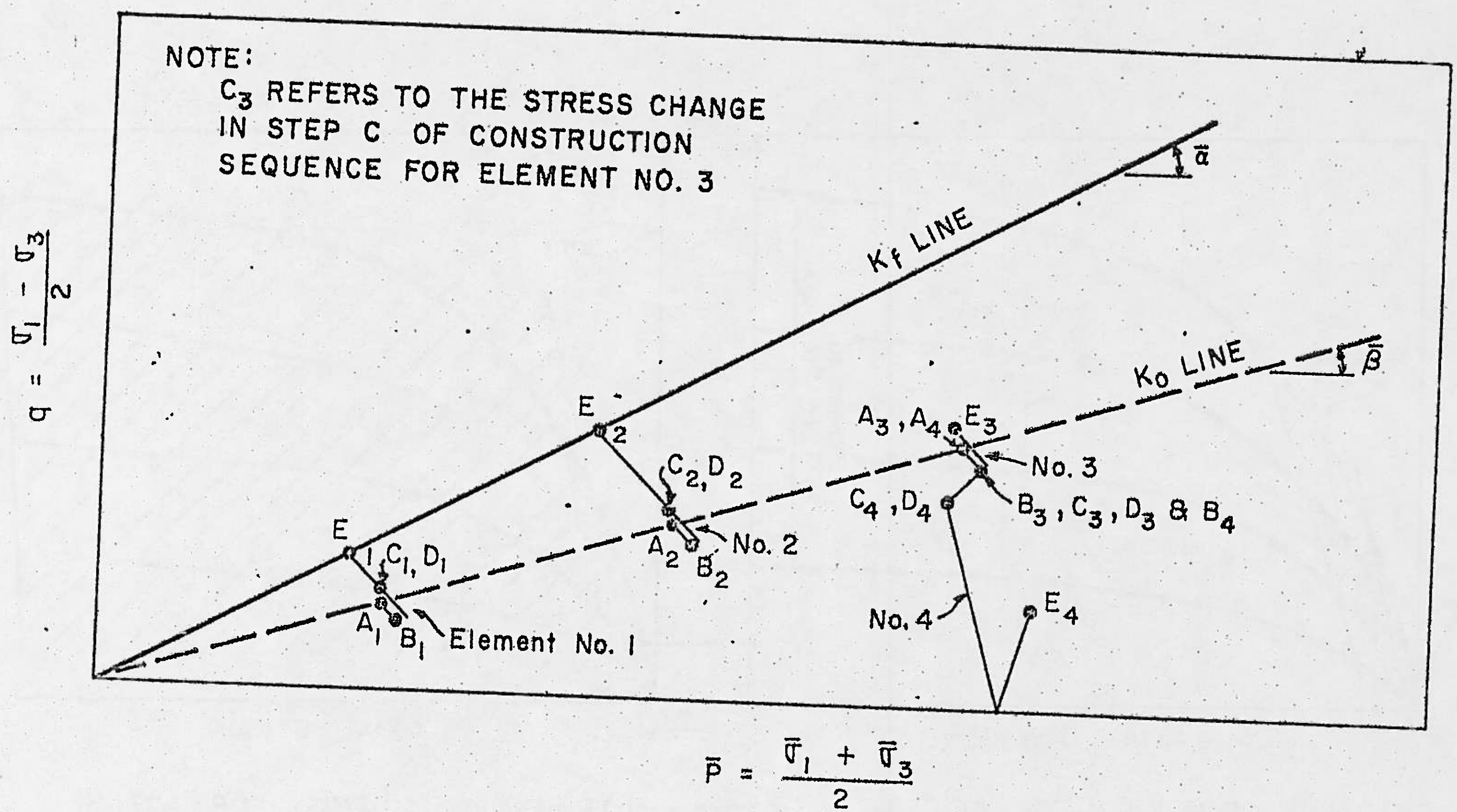
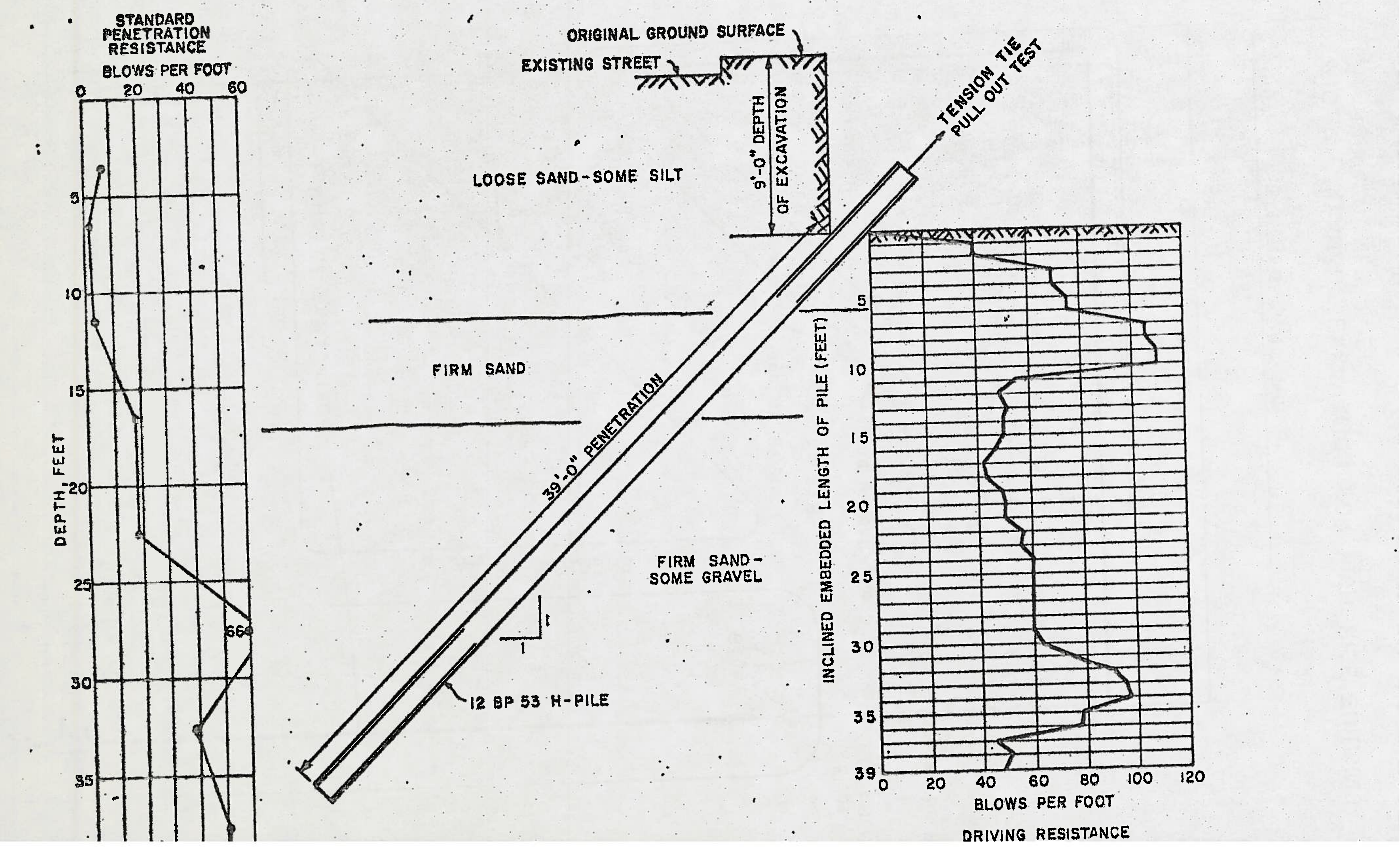


FIG. 5 - STRESS CHANGES DUE TO CONSTRUCTION SEQUENCE FOR "ACTIVE" DESIGN CASE


STANDARD PENETRATION RESISTANCE
BLOWS PER FOOT
0 20 40 60
DEPTH, FEET
5
10
15
20
25
30
35
66
ORIGINAL GROUND SURFACE
EXISTING STREET
9'-0" DEPTH OF EXCAVATION
TENSION TIE PULL OUT TEST
LOOSE SAND - SOME SILT
FIRM SAND
FIRM SAND - SOME GRAVEL
39'-0" PENETRATION
12 BP 53 H-PILE
1
1
INCLINED EMBEDDED LENGTH OF PILE (FEET)
5
10
15
20
25
30
35
39
0 20 40 60 80 100 120
BLOWS PER FOOT
DRIVING RESISTANCE

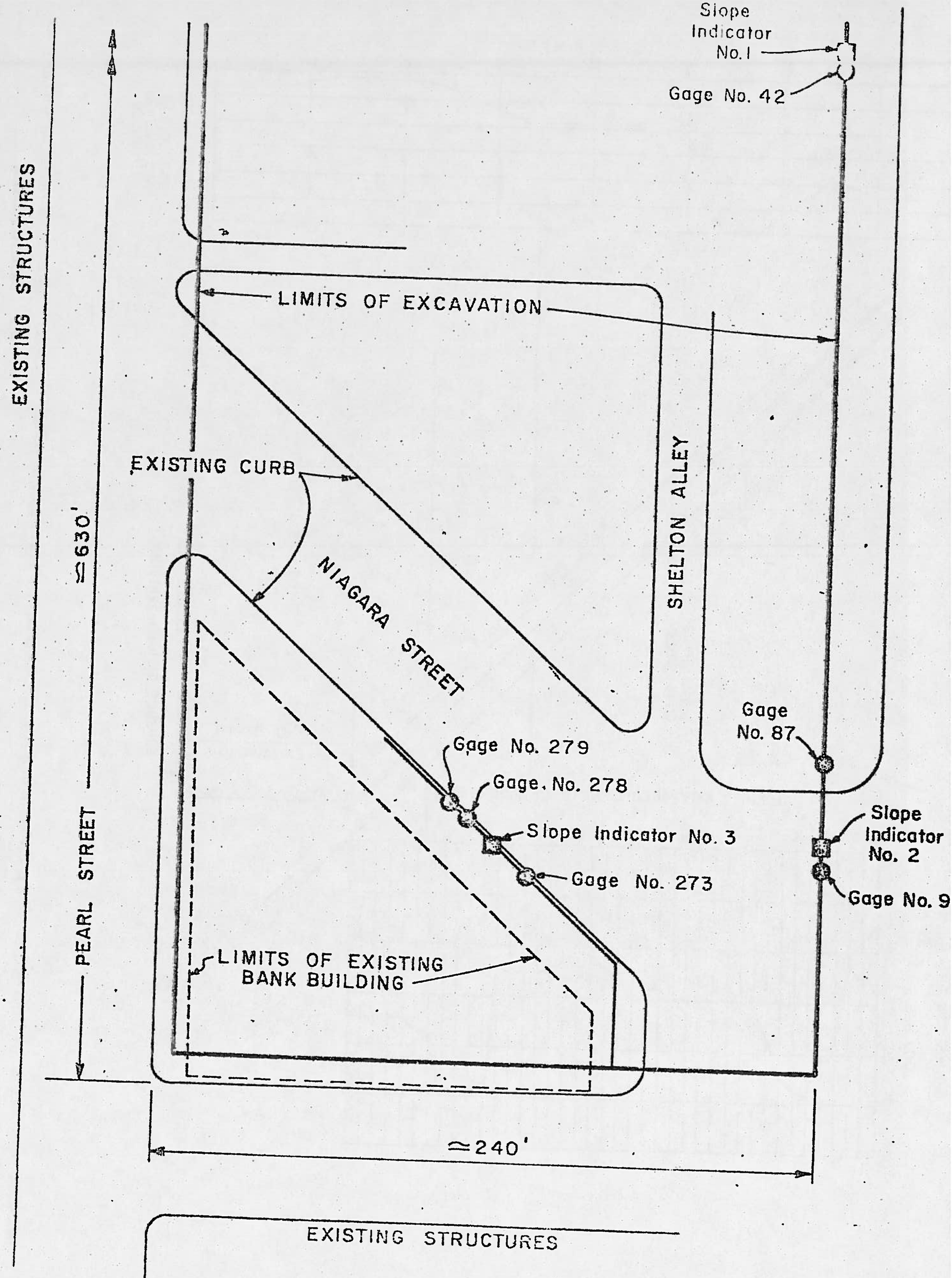
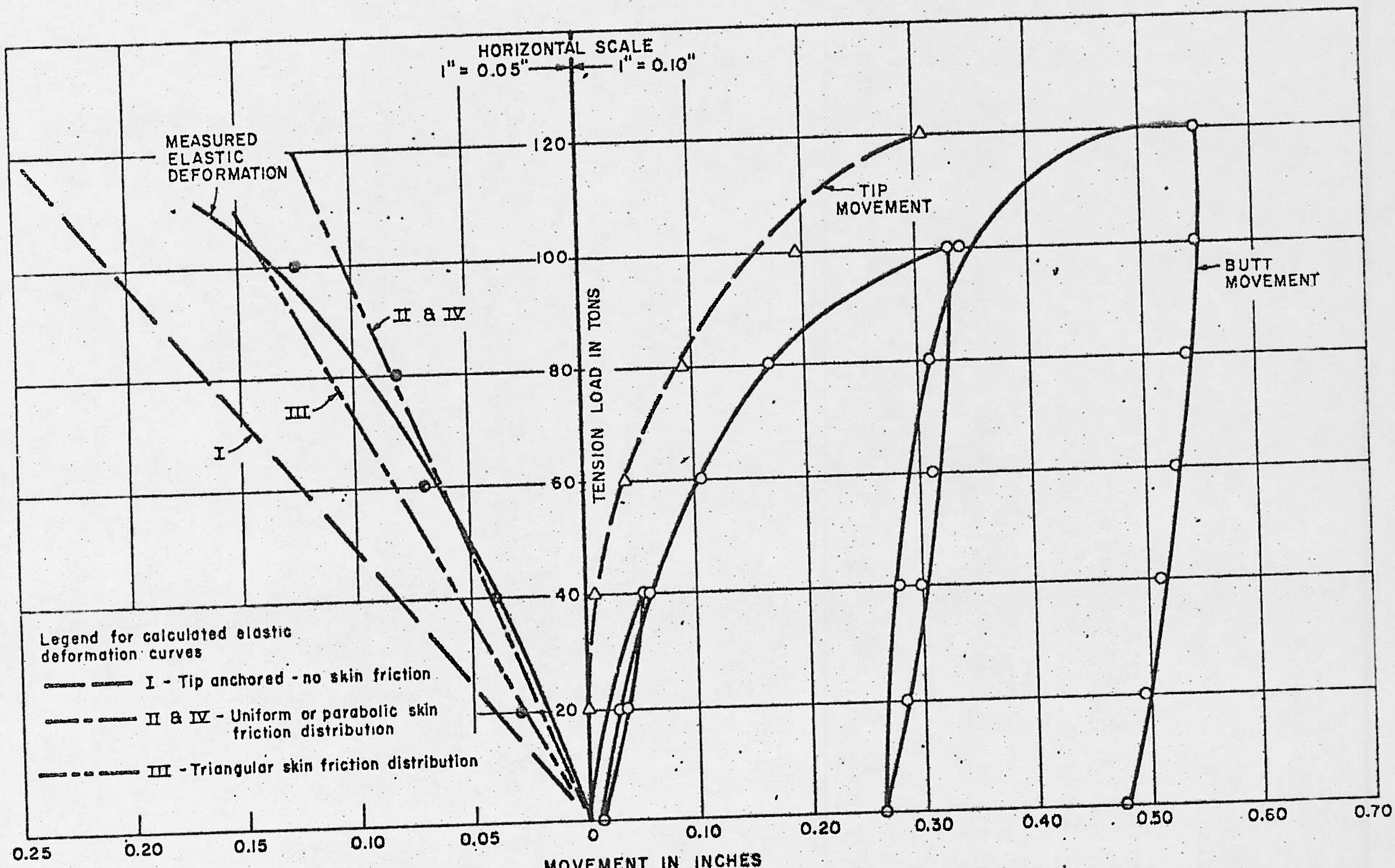


FIG. 7 – PLAN OF EXCAVATION FOR HIGH-RISE BUILDING IN BUFFALO, NEW YORK



PILE SIZE: 12 BP 53, 39 FEET EMBEDMENT

MEASURED ELASTIC DEFORMATION IS EQUAL TO BUTT MOVEMENT MINUS TIP MOVEMENT

FIG. 9 - PILE PULL-OUT TEST RESULTS

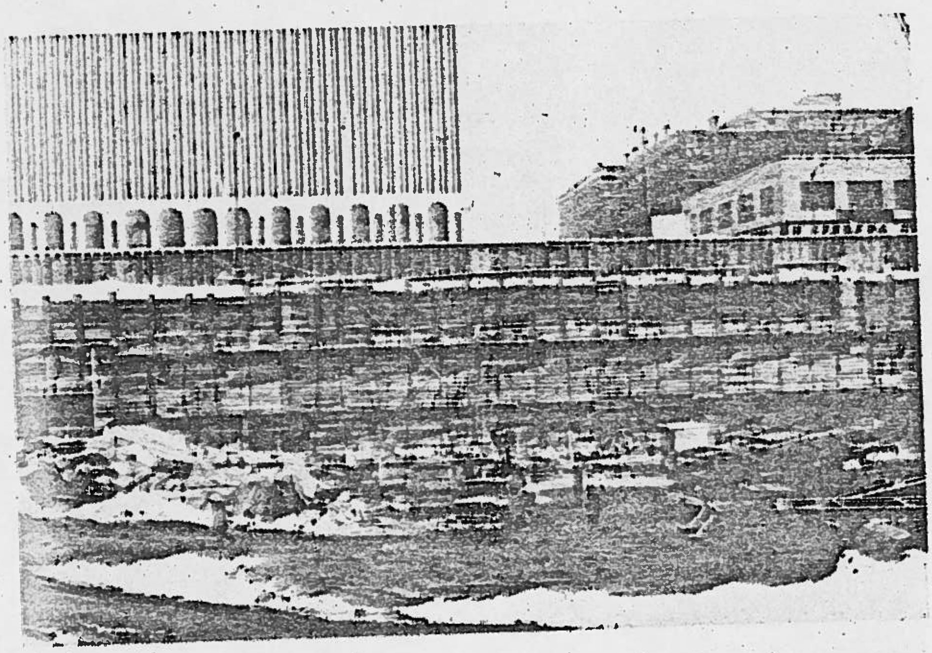


FIG. 10 - VIEW OF SOUTHEAST PORTION OF TIE-BACK WALL

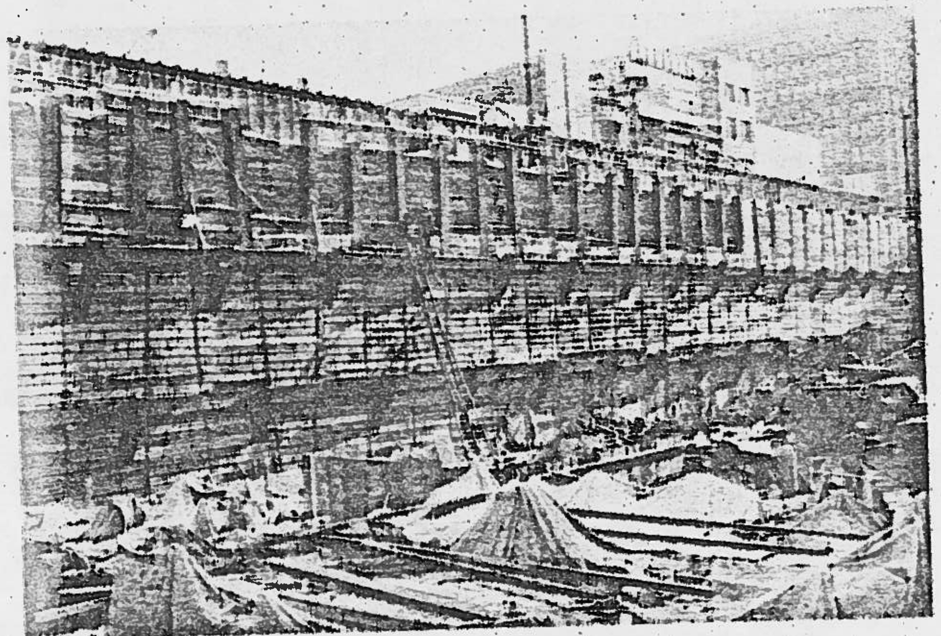


FIG. 11 - VIEW OF NORTHEAST PORTION OF TIE-BACK WALL

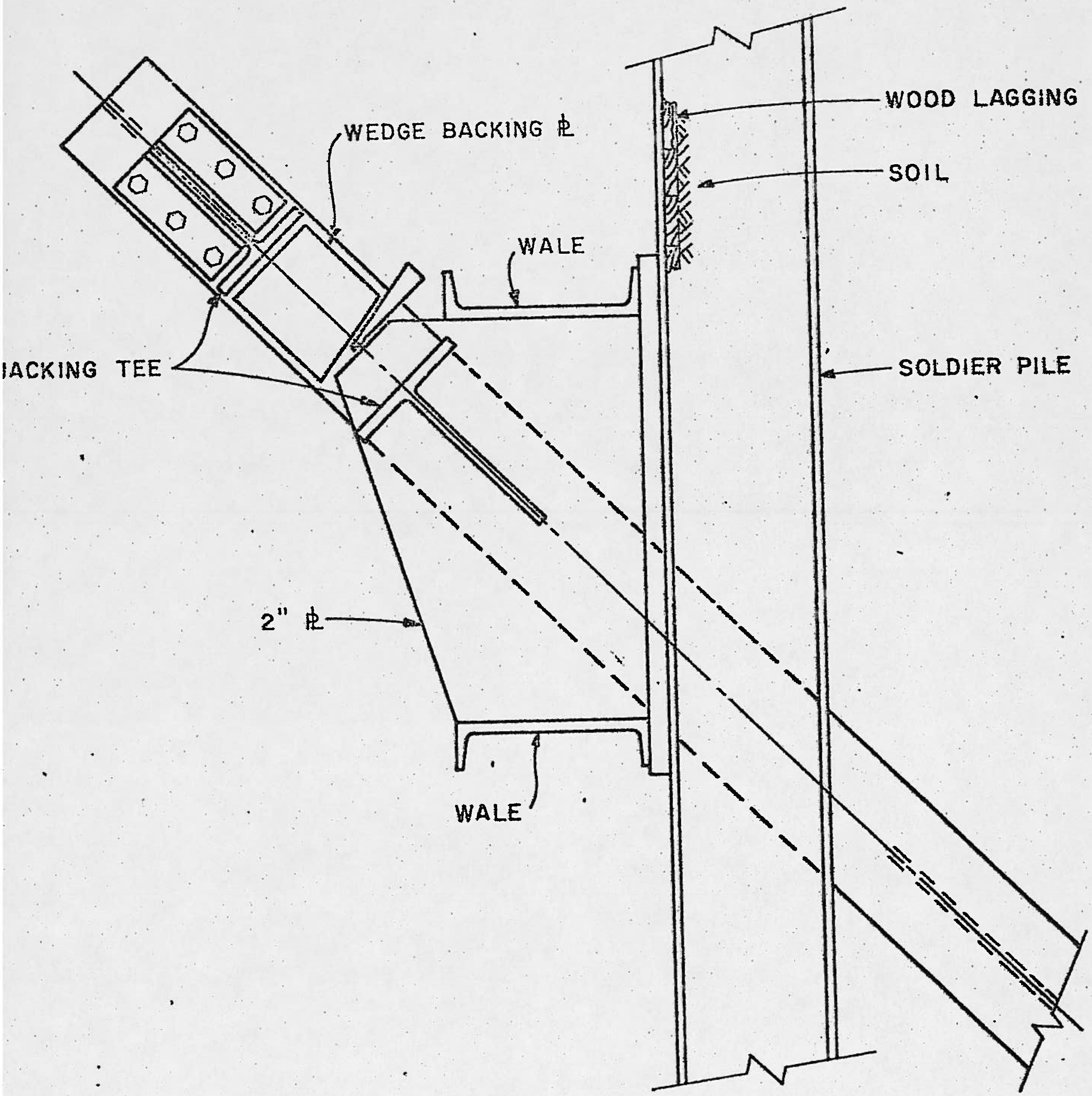


FIG. 12 - DETAIL OF H-PILE TIE-BACK TO WALE CONNECTION

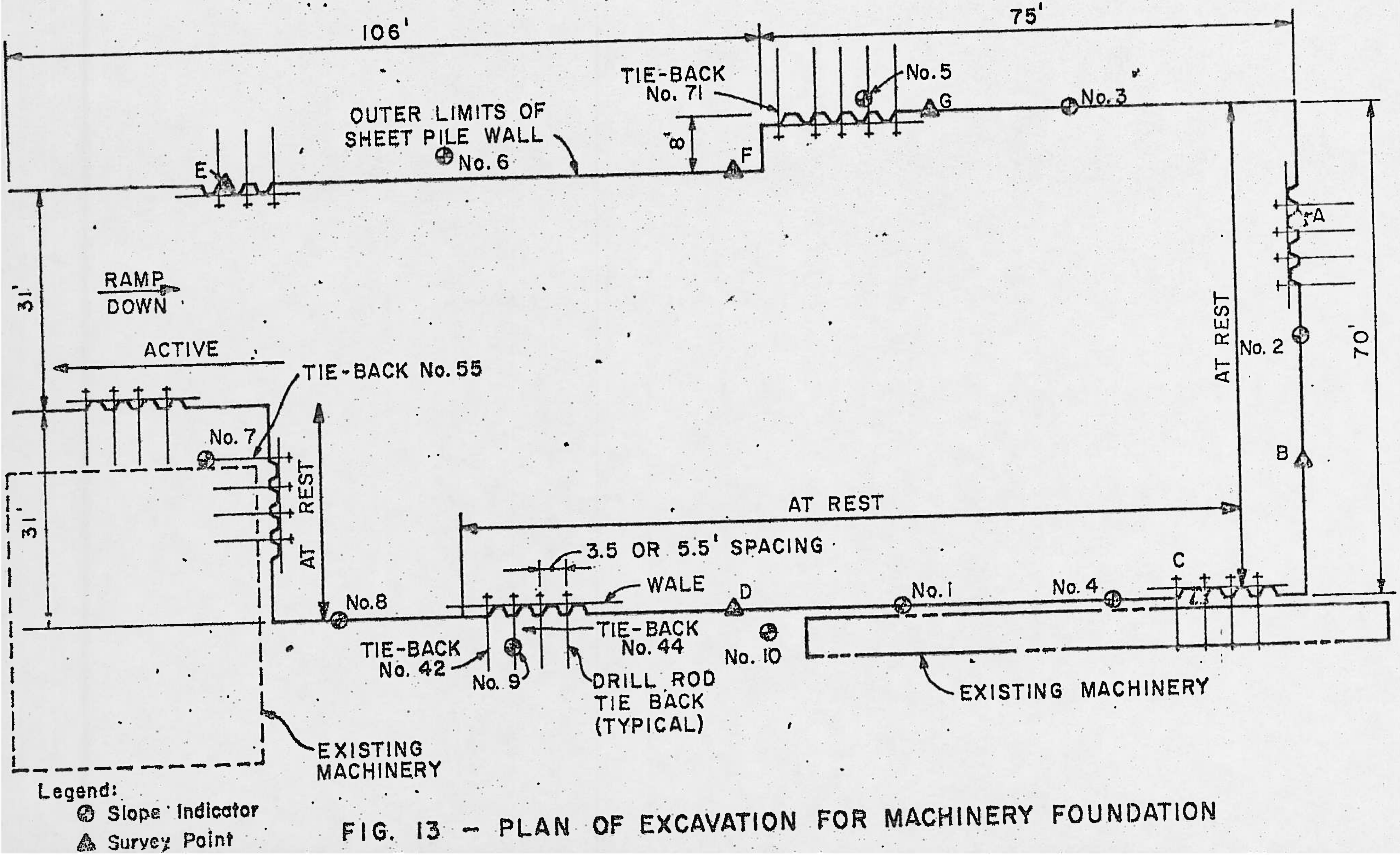


FIG. 13 – PLAN OF EXCAVATION FOR MACHINERY FOUNDATION

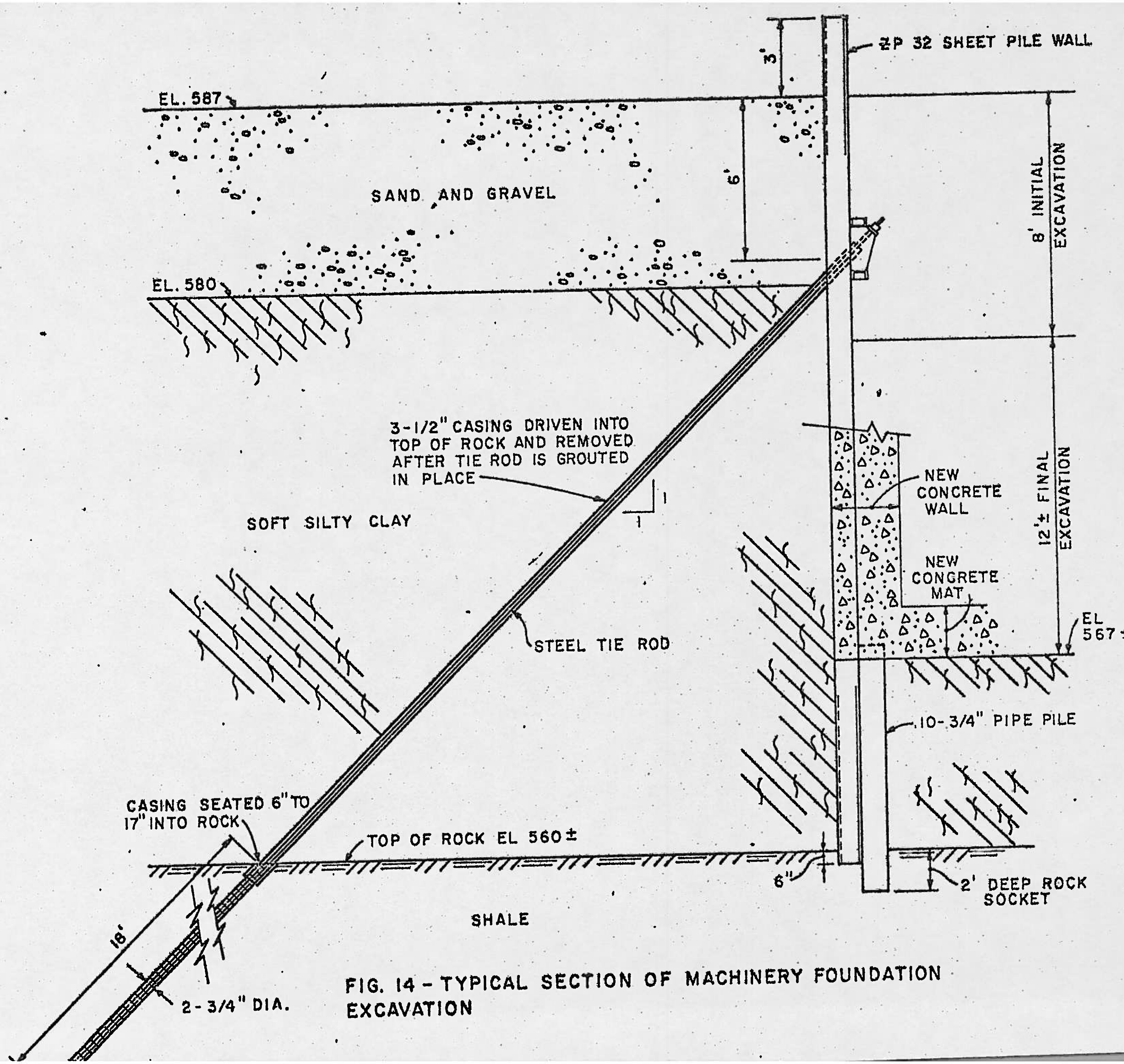


FIG. 14 - TYPICAL SECTION OF MACHINERY FOUNDATION EXCAVATION

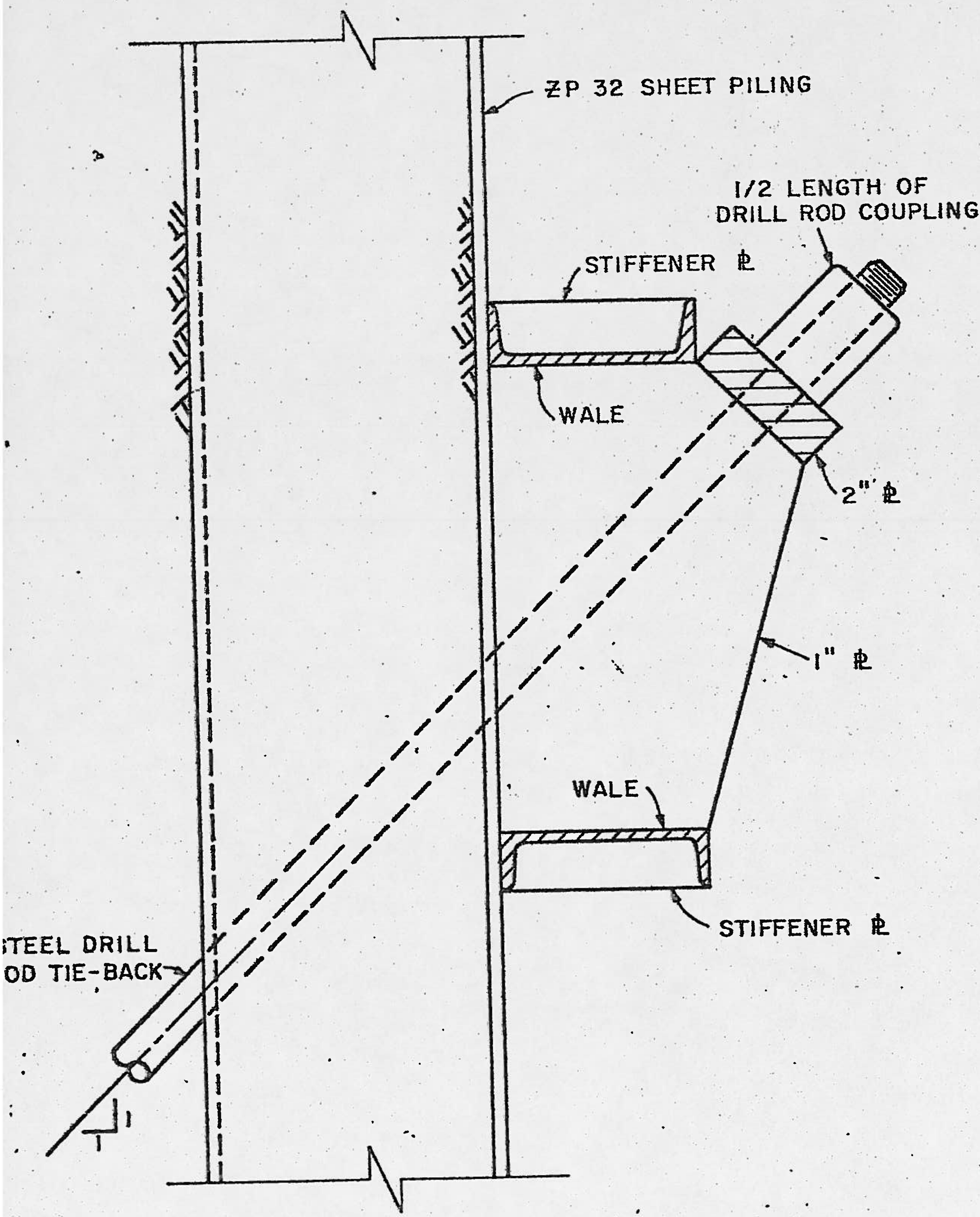


FIG. 15 - DETAIL OF DRILL ROD TIE-BACK TO WALE CONNECTION

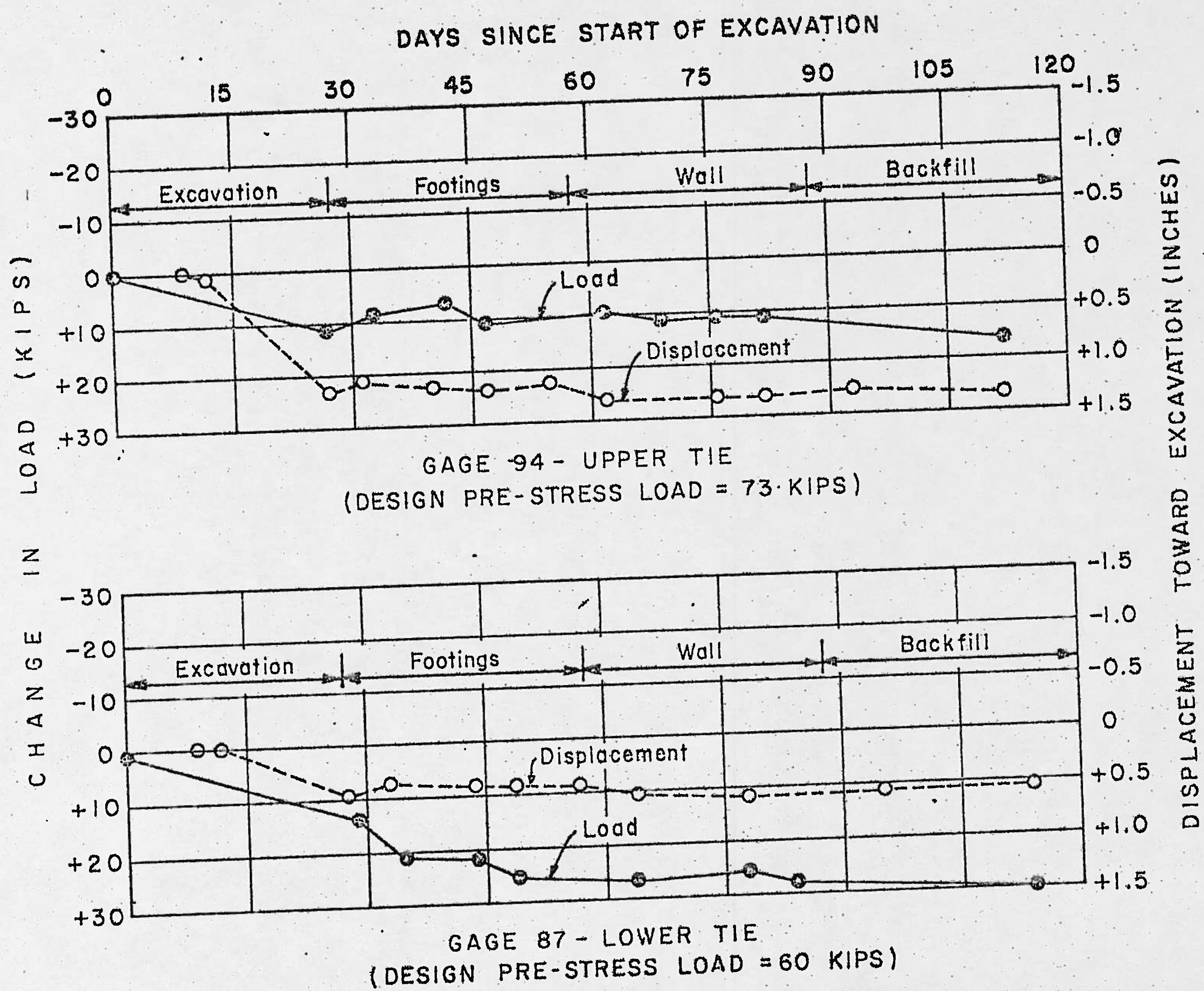
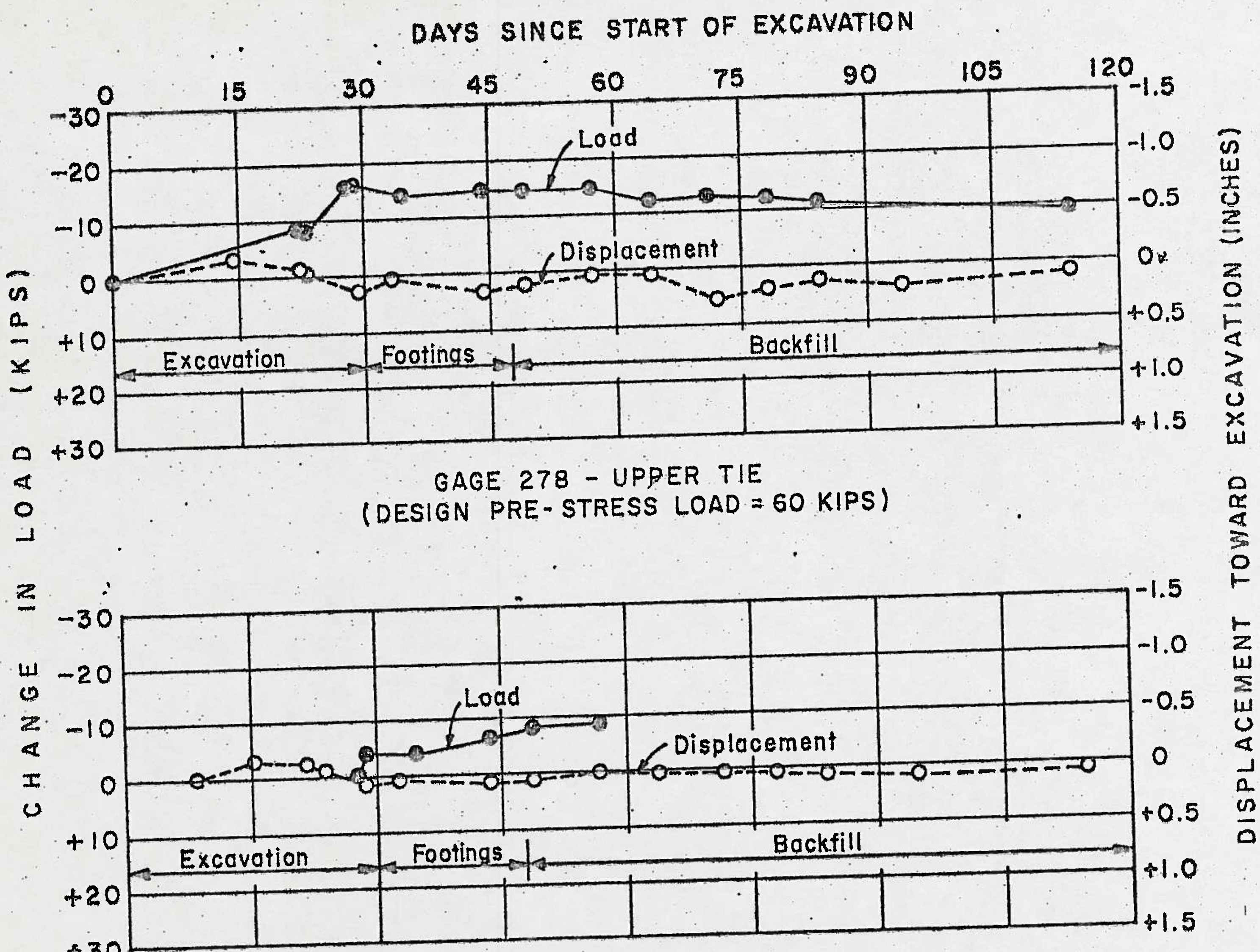


FIG 16 - "ACTIVE" DESIGN CASE



GAGE 278 - UPPER TIE
(DESIGN PRE-STRESS LOAD = 60 KIPS)

GAGE 27? - LOWER TIE
(DESIGN PRE-STRESS LOAD = 82 KIPS)

FIG. 17 - "AT REST" DESIGN CASE

DESIGN AND CONSTRUCTION OF SUPPORTED TEMPORARY EXCAVATIONS IN URBAN ENVIRONMENT

by

Yves Lacroix and Walter T. Jackson

Woodward-Moorhouse & Associates, Inc.

presented at Third Ohio River Valley Soils Seminar

27 October 1972

SCOPE

When an excavation is required, it is generally more economical to slope the sides than to support them. However, if space requirement does not permit the construction of stable slopes, the sides of the excavation are made vertical and supported.

This paper presents some of the problems which arise during design and construction of deep supported temporary excavations in urban areas. Such excavations are often required in conjunction with the construction of subways, underground garages, and basements of major buildings. The most essential requirement during construction of these excavations is often to avoid objectionable movement of the adjacent structures. Overall economy generally is attained by a design which permits efficient excavation of the soil and practical construction of the permanent structure rather than by economizing on the materials used to support the sides of the excavation.

INTRODUCTION

Earth pressure theories lead to a hydrostatic or triangular pressure distribution on a vertical plane. However, field measurements have shown that the distribution is approximately uniform with depth, with perhaps a reduction of the magnitude of the pressure near the top and bottom of the excavation, when the following conditions are fulfilled: (1) the upper portions of the vertical side walls of the excavation are supported in stages as the excavation is deepened; (2) the walls of the excavation are pervious enough so that water pressure does not build up behind them; and (3) the lateral movements of the walls of the excavation are kept small; ie, probably smaller than two percent of the depth of the excavation. However, with time, the approximately uniform pressure distribution might change into a triangular distribution; this is the reason why permanent retaining walls are excluded from this paper. Conditions (2) and (3) usually exclude cases such as sheet-pile cofferdams built in a river bed. Actually, the sheet piles are impervious enough so that water pressures can build up behind them; also, these cofferdams are often permitted to deform to such an extent that arching, which is responsible for the uniform distribution of lateral pressures, is destroyed.

HISTORY

Although contractors have known for a long time that the upper struts in a sewer excavation carry loads as great as the lower struts, the first engineer who became aware of it was probably J. C. Meem (Meem 1908), who was actively connected with the construction of the New York Subway at the turn of the century. Karl Terzaghi, using measurements made by A. Spilker during the construction of the Berlin Subway in the thirties, developed a rule which permitted the calculation of the maximum strut loads in excavations in unsaturated dense sand. During the construction of the Chicago Subway, in the early forties, Ralph B. Peck extended Terzaghi's rule, in a slightly modified form, for clay. The essential object of these efforts was to determine the size of the struts, which is only one of the many problems which have to be solved during design and construction of supported excavations.

During the past years, further work has been done in England and Chicago in conjunction with the construction of major structures, and in Oslo, Tokyo, Paris, Toronto, Montreal, San Francisco, and Washington in conjunction with subway work. It is unlikely that these studies will result in new design rules. Any improved rule would have to be limited to narrowly defined soil and construction conditions which would drastically limit its application. The current studies should change emphasis from the determination of strut loads to the determination of the effects of some significant factors such as: sequence of excavation and installation of supports, type of supports, arching, temperature, time, stability number of excavation in clay ($N_S = \gamma H/c$). Other factors such as the time and anisotropy should be considered when selecting the undrained shear strength of soft clay (Bjerrum 1972). Also, the depth of soft clay below the bottom of the excavation should be considered (Henkel 1971). The method used for these studies should consist of preparing well documented case histories including the results of measurements obtained by an adequate program of field instrumentation.

DESIGN AND CONSTRUCTION

1. Important Requirements

Design and construction of supported excavations are engineering assignments where cooperation between the owner, engineer, and contractor is of primary importance. A clear understanding of the many facets of the problem is needed among all parties. Because the cost of excavations might be significantly decreased when factors of safety are decreased or when some movements of the adjacent properties are tolerated, the engineer should study, then describe to his client, the possible consequences of failures or partial failures of portions of the excavation. If the consequences are bearable, the factors of safety may be decreased; they should not be decreased in the opposite case. For example, the only way to assure that a building adjacent to a proposed excavation will not suffer structural damage might be to prescribe underpinning. The contractor may decide to save the cost of the underpinning. If the building is not damaged by the construction of the excavation, the saving becomes a profit. If the building is damaged, the contractor might buy the building, fix it and sell it, or turn over the problem to his insurance company. The loss on this operation, if any, may be much smaller than the cost of underpinning. This, understandably, is a course of action which cannot be taken if the building cannot be bought; eg, a school, museum, or historic building.

In most supported excavations, there is an interrelationship between the design assumptions, the construction procedure and sequence, and the details of the structural supports. Therefore, the engineer should remain closely associated with the project during the construction period. In this manner, he can think through the entire problem. Also, he is in the best position to assess the validity of the design assumptions and to request practical modifications of the design should he discover conditions that he did not expect.

2. Steps of the Work

The design and construction of supported excavations may be divided into several steps.

a. Reconnaissance of existing conditions

Existing structures, such as sewers and adjacent buildings, considerably affect the magnitude of the earth pressure. The designer should inspect, often personally, the adjacent tunnels and basements to assess the hindrance or help that they may provide.

b. Subsurface investigation

Borings, water level determinations, sampling, and soil testing are required to determine the parameters from which earth pressures are calculated.

c. Actual and apparent pressure diagrams

For soft clay or clay when the stability number ($N_S = \gamma H/c$) is greater than six, the sum of the actual strut loads $P = \Sigma P_i$ is often 10% to 15% greater than the Rankine active force.

For sand, the sum of the actual strut loads $P = \Sigma P_i$ is often 5% to 10% greater than the Rankine active force.

d. Earth pressure

The total force per unit length of excavation can be calculated for a given excavation depth by means of the Rankine theory of active earth pressure. Actually, it is believed that the inward movement of the sides of the excavations is generally sufficient to create the active state of stress. The Rankine theory gives a horizontal force. However, if the supports are inclined, the total force must be slightly modified on account of the wall friction.

If the supports are horizontal (eg, cross-lot bracing) and the grouund surface outside the excavation is horizontal, the Rankine active force is calculated using the Rankine active earth pressure coefficient:

Sand: $K_A = \tan^2 (45° - \phi/2)$

Clay: $K_A = 1 - 4c/\gamma H$

If the force is directed upward (eg, rakers supported at the bottom of the excavation), the horizontal component of the force is perhaps 0.90 to 0.95 that calculated with K_A. If the force is directed downward (eg, tieback anchored walls), the force is perhaps 1.1 to 1.2 that calculated with K_A. These factors for inclined forces are given in text books (eg, Fig. 30.3 Terzaghi and Peck 1967, Caquot and Kerisel 1948).

The effects of special conditions such as when the water table is above the bottom of the excavation, surcharges are in the vicinity of the excavation, and the ground surface is not horizontal, are shown in Fig. 3.

e. Design of the Structural Supports

The sum of the actual strut loads, $P_1 + P_2 + P_3$ in Fig. 1(a) is roughly equal to the total force P_A given by the Rankine theory; however, the distribution of the total force in the three struts depends to a great extent upon the construction procedures. Therefore, the struts should be designed for loads Q_1, Q_2, and Q_3, the sum of which is often 20% to 50% greater than P. Fig. 2 proposes rules which are adapted from the rules suggested by R. B. Peck (Peck 1969).

The design of the wales and walls of the excavation is less critical than the design of the struts. Whereas the latter may fail by buckling with little warning, the former are stressed in bending; therefore, overstressing increases deflection, which leads to arching, which in turn reduces bending stresses. Until additional field observations are available, the following approach may be used to design structural members for excavations with depths smaller than about 70 ft.

<u>Struts and Rakers</u> The maximum probable loads in struts and rakers are calculated from the apparent pressure diagrams. Failure of struts and rakers occurs by buckling which is an instability mode of failure taking place suddenly. The designer must answer the question: how should I design a member against buckling when I know the maximum probable load?

The AISC Code uses an allowable stress f_a such that the factor of safety against buckling is at least 1.8. Such a factor of safety appears to be unnecessarily high. A factor of safety against buckling of 1.5 or even 1.25 appears to be sufficient. Struts and rakers generally are assumed to be hinged at the ends. The strut or raker buckling length is equal to their total length unless they are adequately restrained at some points along their total length.

<u>Tie-Backs</u> Because there is still much uncertainty about the behavior of tie-backs anchored in rock or soil, good practice dictates that all tie-backs be pretested to a load equal to 1.25 or even 1.5 times the design load calculated from the apparent pressure diagram. The tie-back itself should be designed so that it is stressed below the elastic limit under the pretest load.

Tie-backs should not be installed too steeply; near-failures have occurred due to overloading of soldier beams in end bearing, due to the vertical component of the tie-back load. For soldier beams not driven to bedrock or firm supporting strata, tie-backs should be no steeper than 15° to 20° from the horizontal.

Steel Sheet Piles and Soldier Beams Steel sheet piles and soldier beams may be designed using a pressure diagram calculated from the apparent pressure diagram by decreasing the earth pressure by 20% for sand and 50% for other soils. AISC allowable stresses for temporary construction may be used; ie, 1.2 times the allowable stresses for permanent construction.

Field experience has shown that steel sheet-pile walls performed satisfactorily when designed as explained above with the modification of the distribution of the pressure diagram shown in Fig. 4. The modification consists of distributing the load between the strut levels so that it is maximum at the strut levels and zero midway between the strut levels. Sheet piles and soldier beams are assumed continuous, as indeed they are.

Wales Wales may be designed by distributing the loads as shown on Fig. 5b, depending on: (1) whether deflections at struts are small, as when struts are prestressed, and (2) whether it is convenient to construct them as continuous members with full moment splices. AISC allowable stresses for temporary construction may be used; ie, 1.2 times allowable stresses for permanent construction.

Lagging Lagging may be designed to reflect arching between soldier beams, except when the lagging is placed outside the excavation behind the soldier beams; see Fig. 5a. In the past, this arching has lead to empirical determinations of the thickness of the lagging, which is then adjusted in the field when bowing of the lagging boards is observed.

f. Heave and blowout

The possibility of heave or bottom failure of the excavation should be investigated. The Terzaghi and Peck method (Terzaghi et al 1967) or the method described by Bjerrum and Eide (Bjerrum et al 1956) is recommended for use.

In clay, a plastic zone forms below the bottom of the excavation. The depth of the plastic zone depends upon the height of H and the width B of the excavation.

The magnitude of the heave is related to the stability number $N_s = \gamma H/c$. When N_s is greater than 4, heave should be expected. Large heave is likely to occur when $N_s = 6$ to 8.

Blowouts through openings between soldier beams or lagging boards should be expected when $N_s = 6$ to 8 (Broms 1967).

g. Estimate of movement

The magnitude of the lateral movement and settlement of the side of an excavation and adjacent properties cannot be readily calculated. In clay, significant movements usually take place when N_s is greater than 3 or 4. When N_s is between 5 and 10, settlements as great as one or two percent of the excavation depth cannot be avoided. Settlements are due to two causes: (1) deflection of the wall of the excavation and heave; and (2) consolidation. Empirical estimates of probable settlements are shown on Fig. 6. Typical movements for average workmanship are shown on Fig. 7.

h. Type of supports and construction sequence

The choice of the type of supports and the planning of the construction sequence are generally considered to be the prerogative of the contractor. This view is sound, but should not prevent the engineer from preparing a design memorandum in which the relative merits of several procedures are outlined.

Because the magnitude of the movements of the adjacent properties depend upon the length of time during which the excavation remains open, the work schedule should be arranged so that the length of time the excavation remains open in the vicinity of sensitive structures is minimum. Because experience is always acquired during the first stages of the work, it is recommended that construction be started in the less critical portions of the excavation to obtain the maximum profit from the observations made at the beginning of the project.

Table 1 gives a list of usual construction procedures.

The following is a list of statements which may be used as guides when the type of support and construction sequence is being determined.

(1) Staged construction leads to less movement of the neighboring properties and is not neecessarily more expensive than excavating the entire site before starting construction of the permanent structure.

(2) Walls consisting of steel sheet piles or reinforced concrete piers, which are later used as part of the permanent wall, are satisfactory; this also applies to a cast-in-place reinforced concrete wall built in a slurry trench.

(3) Soldier beams, such as H-piles, or in some cases, cast-in-place reinforced concrete piles, are economical. They are sometimes necessary when sheet piles cannot be driven. Timber lagging is installed between the soldiers as the excavation proceeds. To permit arching and reduce settlement, lagging should not be placed outside the soldier beams; see Fig. 8.

(4) Some settlement of the adjacent ground cannot be avoided because the sides of the excavation have the tendency to move below excavation level; that is, before the supports can be installed; see Fig. 9.

(5) To reduce movement to a minimum, the following steps should be taken.

- The struts should be prestressed. The amount of prestressing is not very significant; the struts are usually prestressed to 50% of their design loads. Often, one or two days after prestressing, the load has decreased to 25% of the design load. In some cases, when the struts are being prestressed, the excavation wall moves outward. It is recommended that prestressing be stopped before the movement of the wall toward the soil reaches: (a) 1.5 in. when the upper strut is prestressed, and (b) 0.5 in. when the other struts are prestressed.

- Tie-backs may also be prestressed to 50% of the design load; however, because their ultimate capacity is never known, they should be pretested to 125% or even 150% of their design loads.

- The vertical spacing between strut levels should be minimum. However, when the bottom slab has been constructed, some intermediate struts may be removed providing the wall has been designed for the greater vertical span; see Fig. 10.

- Reduce the length of time between berm excavation adjacent to the wall and placement of struts. It may be more efficient to excavate from the perimeter of the cut towards the center, but this results in a longer time during which the berms and wall creep inward prior to street placement.

(6) Although there is no rule concerning damage to adjacent property which can reasonably be expected, Fig. 7 gives values that may be used as a guide.

(7) Unavoidable movement is often sufficient to create, or increase, leaks in old utilities such as sewers and water mains. These leaks considerably facilitate loss of soil, which is the major cause of objectionable movement. Serious consideration should be given to removing these utilities before they create difficulties, or exposing and supporting them on a row of deep driven piles.

(8) If cracks develop outside the excavation or if side slopes start to slough in, they should be sealed or covered with tarpaulins, plastic sheets, or gunite to prevent rainfall from entering the cracks and increasing instability.

(9) Because clay creeps and consolidates, its passive resistance should not be used for strut reaction. Passive resistance of dense sand may be used if the struts are pretested to about design load.

(10) Supported excavations rarely fail because members, such as struts and wales, are designed too small. Most failures are due to poor connections between members, lack of stiffeners, or omission of significant details during design of supports, or damage during construction.

(11) The sequence of removal of supports as related to backfilling and construction of floor slabs and permanent supports is significant. Without care, movements after removal of the supports can be surprisingly large, greater than those due to all the preceding operations of excavation and placement and stressing of supports. This is more important for struts than for tie-backs.

(12) The effect of temperature on strut loads is small when the wall is flexible because the soil deforms plastically. If the wall is rigid, a temperature variation of 100°F may create stress in a long strut on the order of one kip per degree Fahrenheit. Freezing may cause the formation of ice lenses behind the wall, resulting in movement and significant increases of the loads in the supports; heating of the wall might be necessary; see Fig. 11 and 12.

(13) Earth pressure has the tendency to increase with time because the strength parameters of soils have the tendency to decrease.

i. Construction inspection and instrumentation

As mentioned earlier, it is rare that all design assumptions are correct. Intelligent inspection during construction permits modification of the pre-established procedures, often in the direction of economy.

Substantial savings may be made by reducing the design factors of safety and by watching for the precursory signs which, in most soils, precede objectionable deformation or failure. A well-designed instrumentation program should provide warning signals which give time to take corrective steps before catastrophic events occur.

Reference points for settlement and lateral movement observations should be installed on all adjacent structures and on the walls of the excavation. Strut loads may be measured, although not very conveniently, by means of mechanical, electrical, or vibrating wire strain gages. Direct measurements of earth pressures (total pressures) are justified only for research projects. The same comment applies to measurements of pore-water pressures in impervious materials. However, static water levels are important to know and may be easily measured with standpipes or wellpoints. Measurements of heave are extremely valuable; although the measurements are simple, they are often poorly done. Measurements of the lateral movement of the wall along its entire depth can be made very well with the Wilson slope indicator, which was used to obtain the data in Fig. 9. These measurements are not very difficult, but are expensive in terms of man-hours.

In addition to the saving that field instrumentation usually makes possible in the latter stages of any given project, the observations provide the basis for a better understanding of earth pressure phenomena.

CONCLUSIONS

There is place for much improvement in the design and construction of supported temporary excavations. It does not appear that significant progress would be achieved by better earth pressure theories or even by an improved knowledge of the soil properties. Significant savings will come from a better understanding of the factors which separate the avoidable from the unavoidable movement of the adjacent soils and from an acute identification of the precursory signs of failure. It is not thought that this understanding and knowledge can be obtained from model tests. It is believed that full-scale field measurements and complete construction records, ie, fully-documented case histories, will help in improving the design and construction of supported excavations in urban areas.

EXCAVATION	WALLS	SUPPORTS	REACTION
1. Excavate narrow trenches at periphery in which permanent walls are built. Then, excavate center portion and build structure.	1. Timber (rare) or steel sheet pile wall.	1. None, use cantilever action of walls (structural engineers often assume a too shallow depth of fixity).	1. Cross lot bracing.
2. Excavate entire site and build permanent walls and structure.	2. Soldier beam: steel H piles driven or installed in pre-bored holes, concrete piles, short sections of permanent wall built in hand-dug pits. Lagging placed as excavation proceeds between soldier beams which are at 6- to 10-ft intervals. Lagging consists of 2- to 4-in. lumber or sometimes reinforced gunite membrane.	2. Horizontal or inclined struts. Wales are recommended to ensure lateral stability.	2. Passive resistance of soil or rock.
3. Excavate and build permanent walls and structure in stages.	3. Reinforced concrete wall cast in place in a slurry trench.	3. Pretested tie-backs in rock or soil.	3. Adhesion of base slab to foundation soil or rock.
		4. Beams of permanent structure.	4. Lateral resistance of foundation of structure: piles or piers.

CONSTRUCTION PROCEDURES

Table 1

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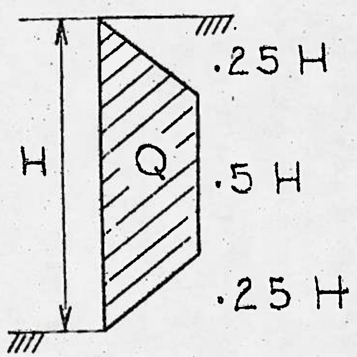
Clay

Stiff

$N = \gamma H/c < 4$



$K\gamma H$

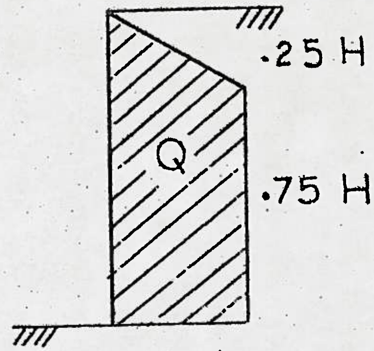
$K = .2$ to $.4$

$P = .5 K \gamma H^2$

$Q = 1.5\ P$

Soft

$N = \gamma H/c > 5$ or 6



$K\gamma H$

$K = 1 - m4c/\gamma H$

$P_A = .5 K \gamma H^2$

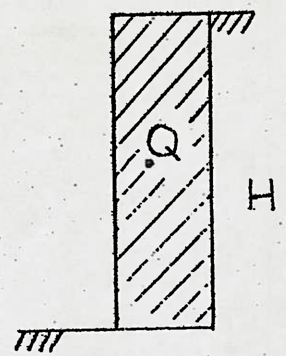
$P = 1.15\ P_A$

$Q = 1.75\ P_A \approx 1.5\ P$

For non-strain-sensitive clay; or when stiff clay below excavation; or for strain-sensitive clay (Oslo, Mexico City) when $\gamma H/c < 4$, use $m = 1$

For strain-sensitive clay when $\gamma H/c > 5$, use $m = .4$

Sand



$.65 K_A \gamma H$

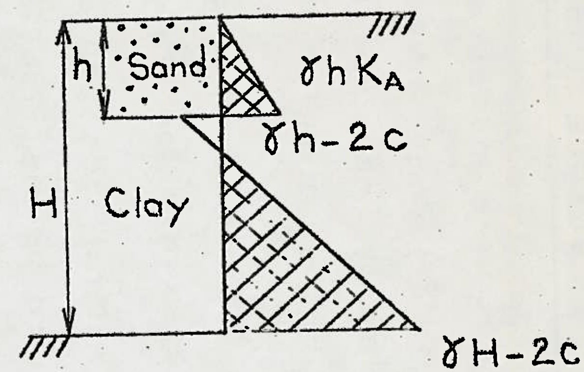
$K_A = \tan^2(45° - \phi/2)$

$P_A = .5 K_A \gamma H^2$

$P \approx 1.1\ P_A$

$Q = 1.3\ P_A \approx 1.2 P$

Mixed Soil



P_A = crossed area

$P = 1.2\ P_A$

$Q = 1.8\ P_A = 1.5\ P$

Distribute Q using as a guide apparent pressure diagrams for clay and sand

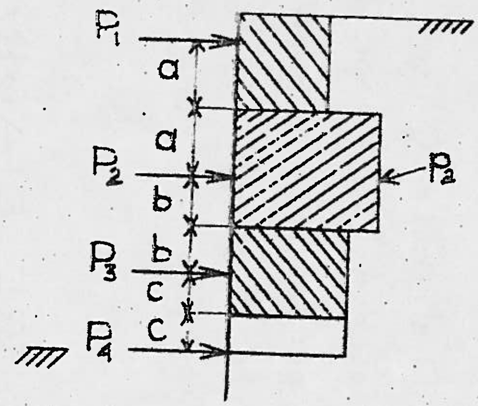
Water table below bottom of excavation

H = 30 - 60 ft, supports placed as excavation is made

GUIDE FOR APPARENT PRESSURE DIAGRAMS

Fig. 2

a. Development of Actual Pressure Diagram



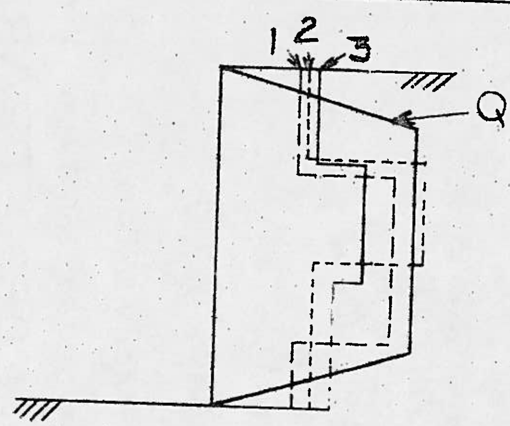
P_1, P_2, P_3 are measured loads in three struts of a vertical profi divided by the strut horizontal spacing; eg Kips/ft

P_4 is the assumed load at the b of the excavation. $P_4 = P_3\ c/b+c$

The actual pressure diagram is obtained by distributing P_1 to P_4 shown; eg $p_2 = P_2/a+b$

The total load $P = \sum_1^4 P_i$ is often between P_A and 1.2 F (P_A is the Rankine active force per unit length of excavat

b. Development of Apparent Pressure Diagram

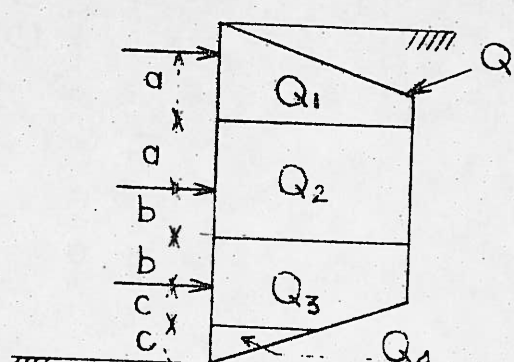


1, 2, 3 are actual pressure diagra for different vertical profiles

The apparent total load Q is equal to area of the apparent pressure diag which envelopes the actual pressure diag

Q is often 1.5 P

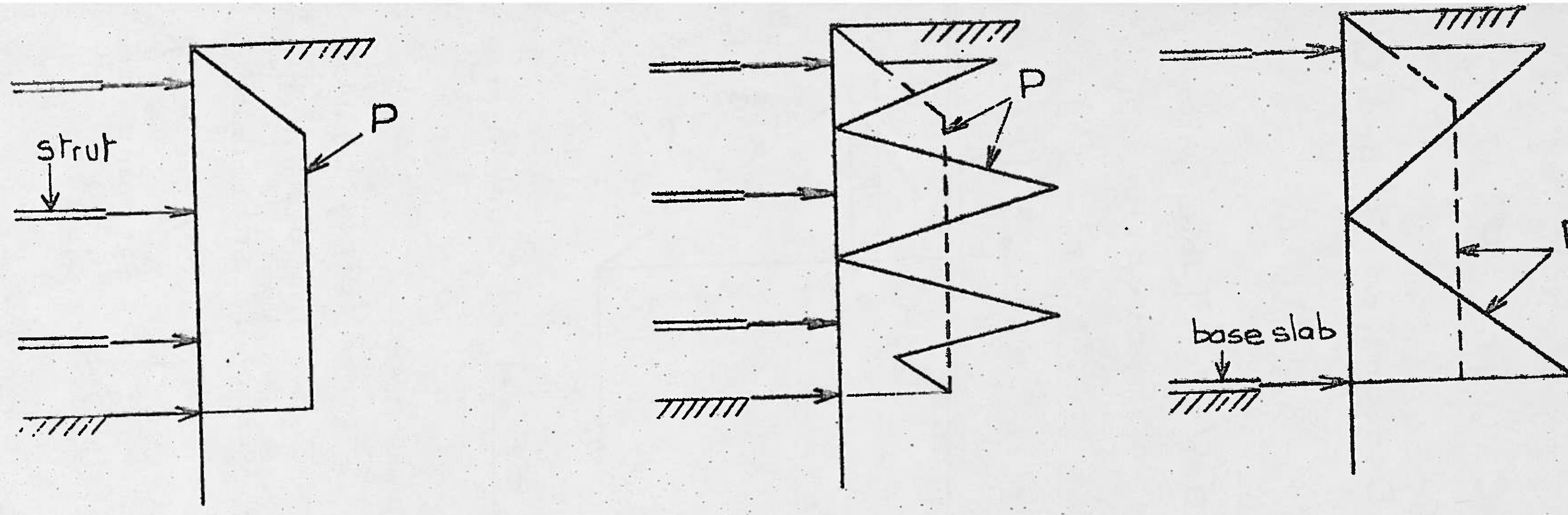
c. Maximum Probable Strut Load



The maximum probable strut loads Q_1, Q_2, Q_3 are calculated from t apparent pressure diagram the s way (but in reverse) the actual pressure diagram was obtained f the loads P_1, P_2, P_3

ACTUAL AND APPARENT PRESSURE DIAGRAN

Fi



Wall supported by three struts and soil below bottom of excavation

Wall supported by top strut and base slab Intermidiate struts removed

Rigid Wall

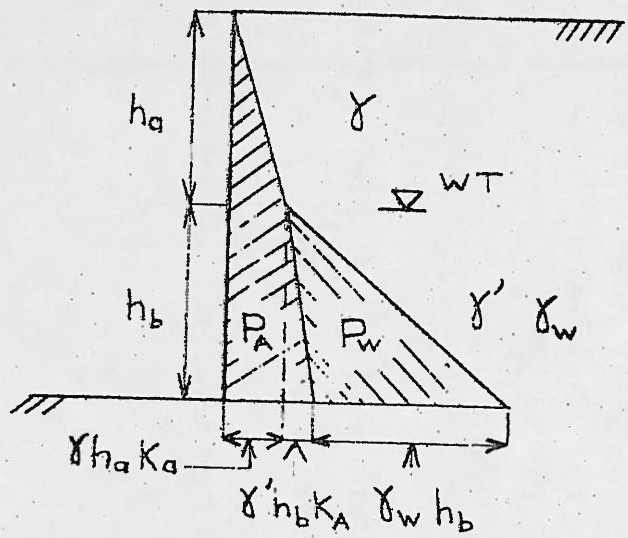
Flexible Wall

Pressure diagram P = Apparent pressure diagram = Q/1.2 for sand and = Q/1.5 for clay and mixed soil

Note: Use AISC allowable stress for temporary structures ; ie 1.2 allowable stress for permanent structures.

SUGGESTED WALL DESIGN

Fig. 4

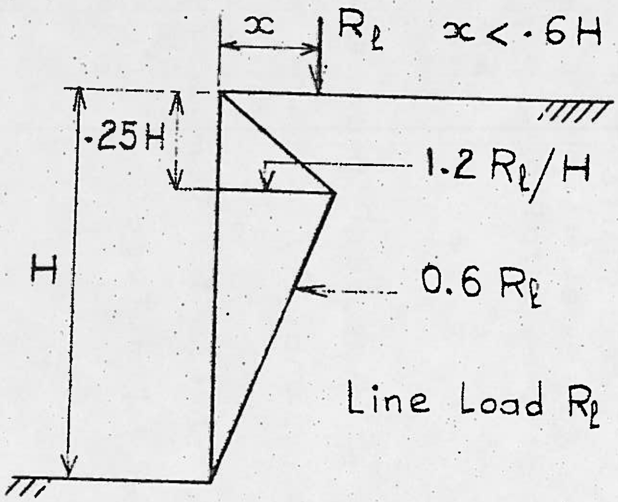


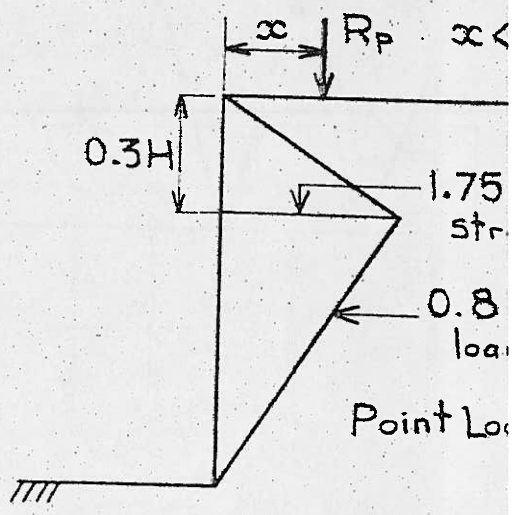
$P = (1.1 \text{ to } 1.2) P_A$ $P' =$ F

$Q = (1.3 \text{ to } 1.8) P_A$ $Q' =$ G

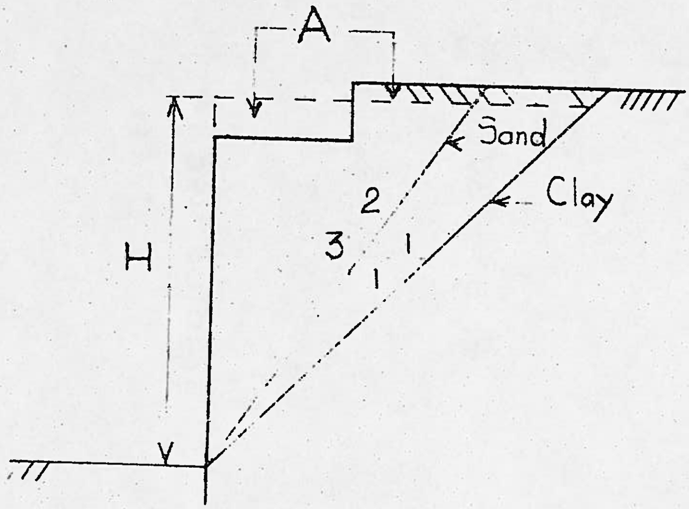
Design for P' and Q'

a. Water Table Above B
of Excavation





b. Effect of Surcharge

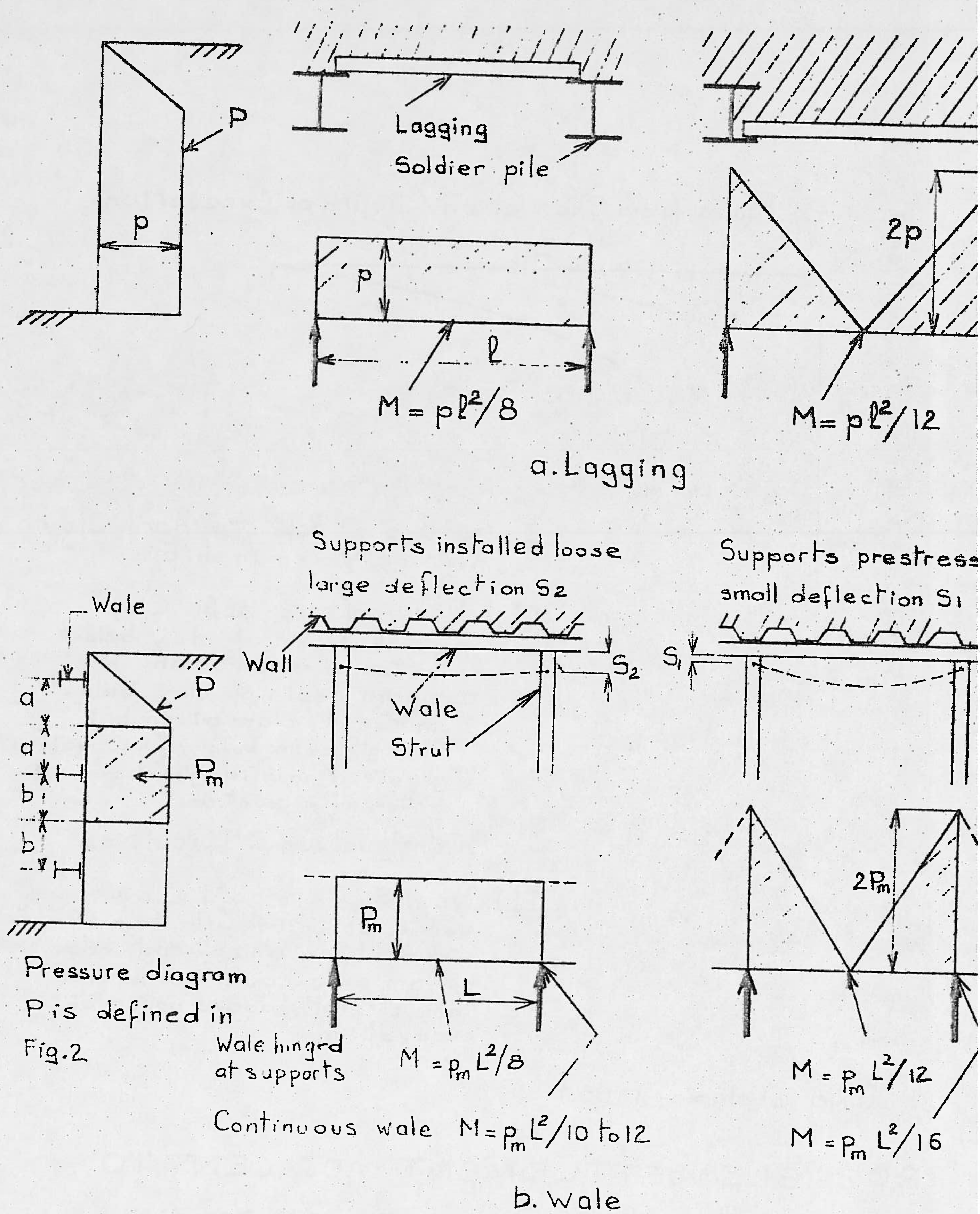


Approximately Equivalent
Excavation Depth H when g
surface is not horizontal.
two surface A are equal.
2:3 slope for sand and
slope for clay

c. Ground Surface is Not
Horizontal

EFFECT OF SPECIAL CONDITIONS

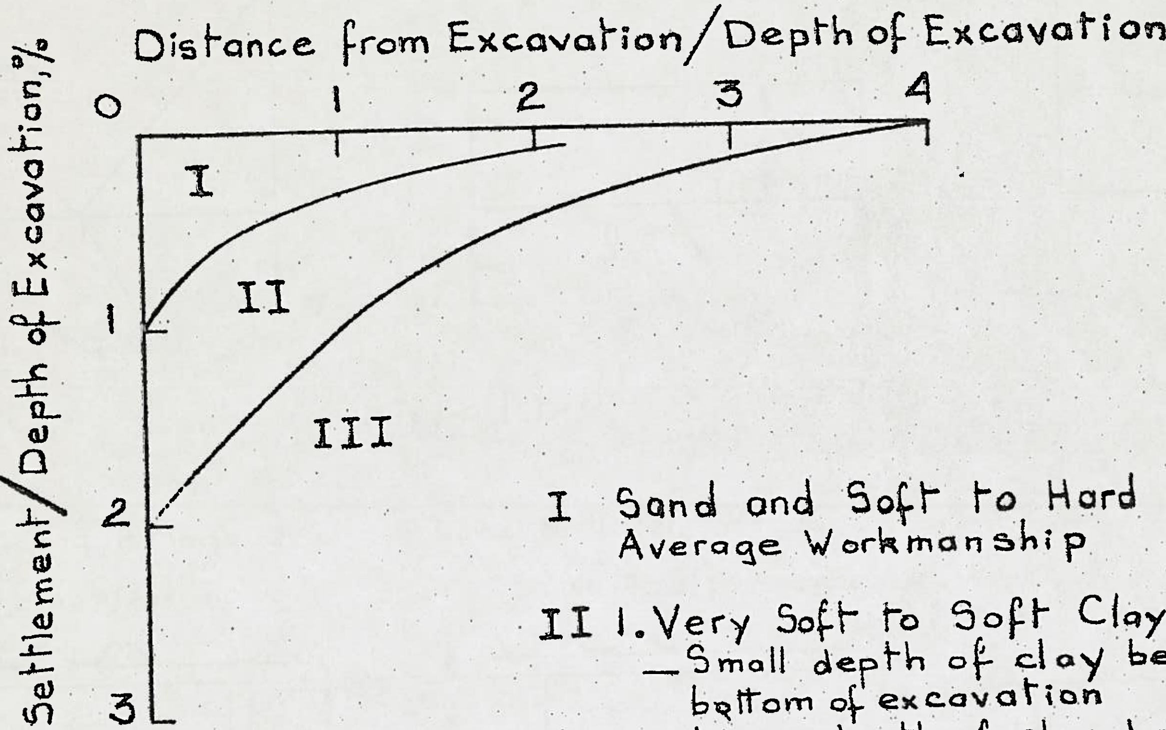
Fi



Note: Use AISC allowable stress for temporary structures; ie 1.2 allowa[ble]
stress for permanent structures

SUGGESTED LAGGING AND WALE DESIGN

Fig



I Sand and Soft to Hard Clay
Average Workmanship

II 1. Very Soft to Soft Clay
— Small depth of clay below bottom of excavation
— Large depth of clay below bottom of excavation but strength below bottom of excavation greater than strength above bottom of excavation

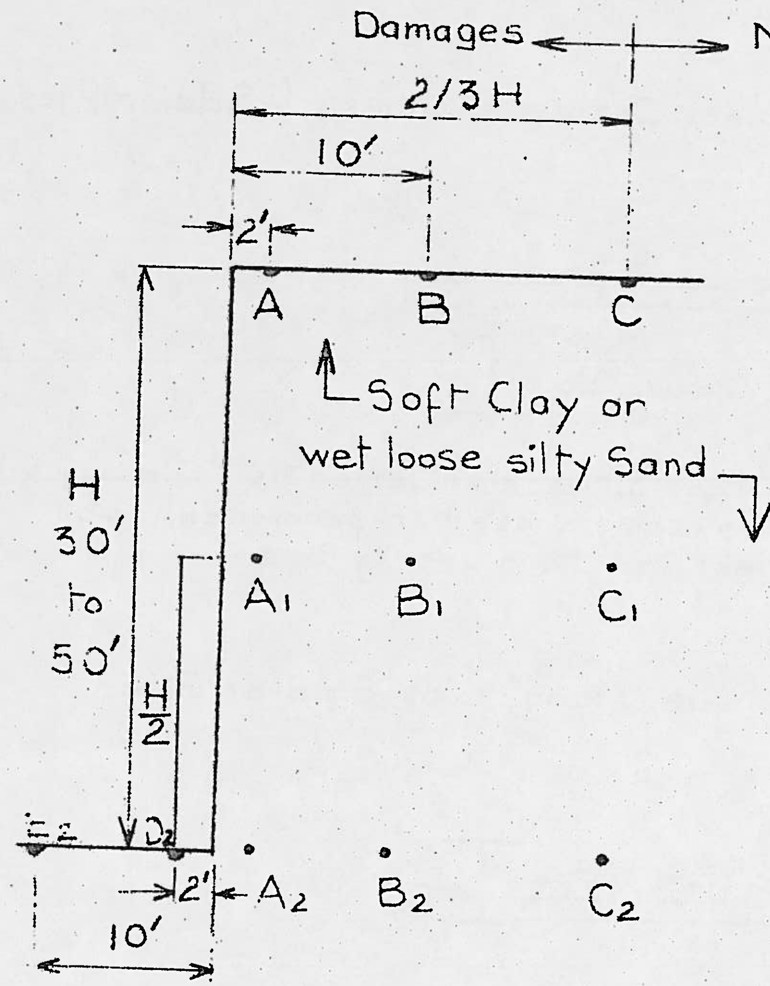
2. Construction Difficulties

III. Very Soft to Soft Clay
Large depth below bottom of excavation and strength below bottom of excavation smaller than strength above bottom of excavation

(after Ireland 1955)

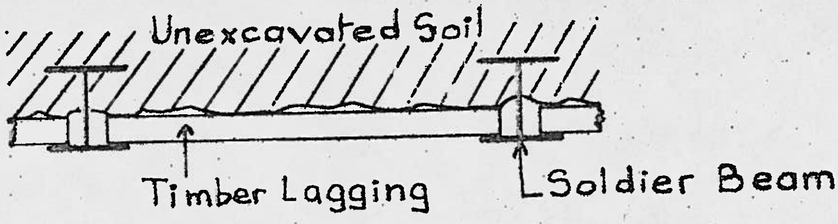
PROBABLE SETTLEMENT ADJACENT TO BRACED EXCAVATION AS FUNCTION OF DISTANCE FROM EDGE OF EXCAVATION

Fig. 6

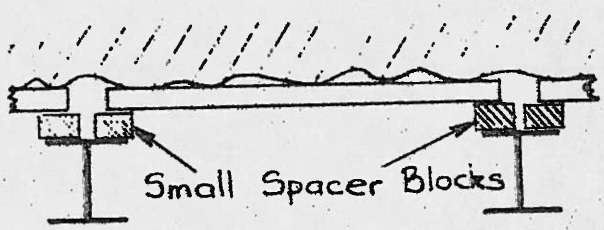


Point	Settlement, in	Lat Mvt
A	2 to 6	1
B	1 to 3	1/2
C	1/4 to 3/4	1/8
A_1	2 ?	2
B_1	1/2	1 t
C_1	1/8	
A_2	1 to 2	1 t
B_2	1	
C_2	0	
D_2	-1 to -2	
E_2	-1 to -2	

TYPICAL MOVEMENTS FOR
AVERAGE WORKMANSHIP

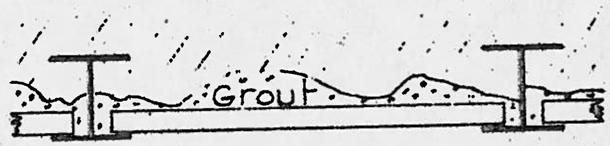


a. Lagging Wedged Against Inside Flanges of Soldier Piles

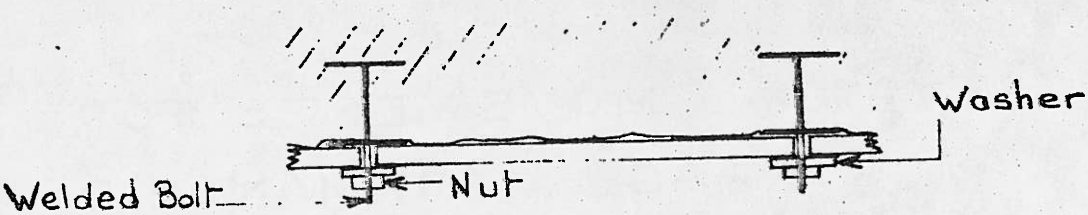


Less satisfactory than a. when settlement is to be avoided; however soldier beams can be used in permanent wall and building may extend to property line

b. Lagging Set Behind Outside Flanges of Soldier Piles



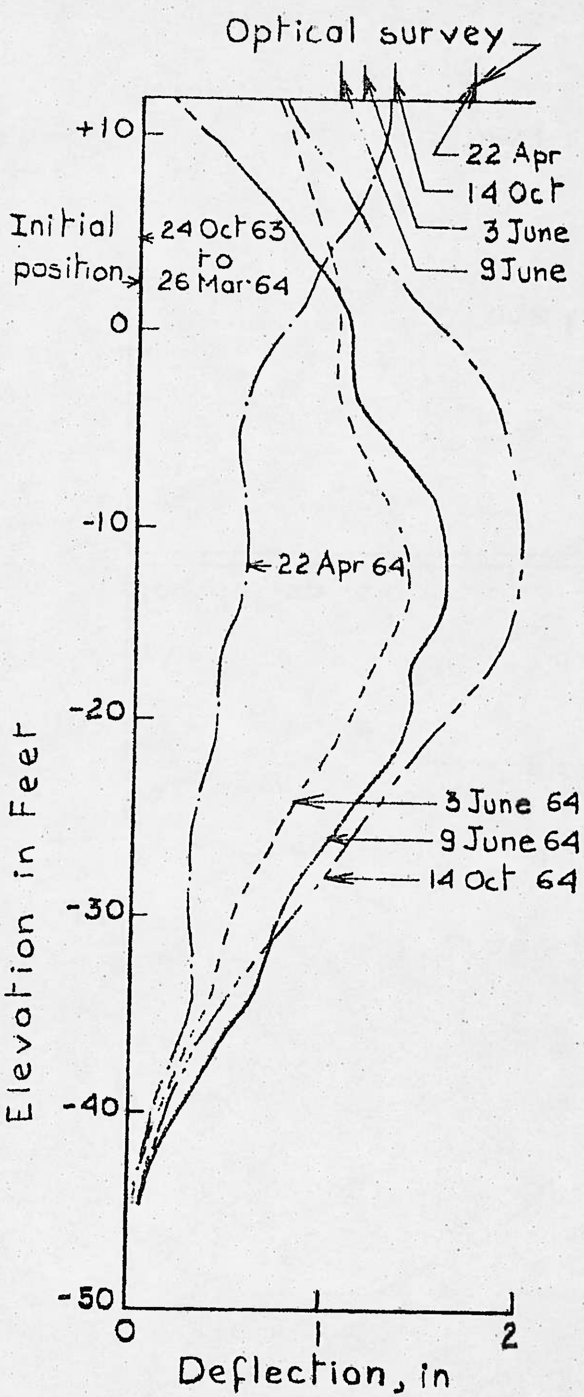
c. Grout, Mortar or Sand Filling Between Lagging and Soil



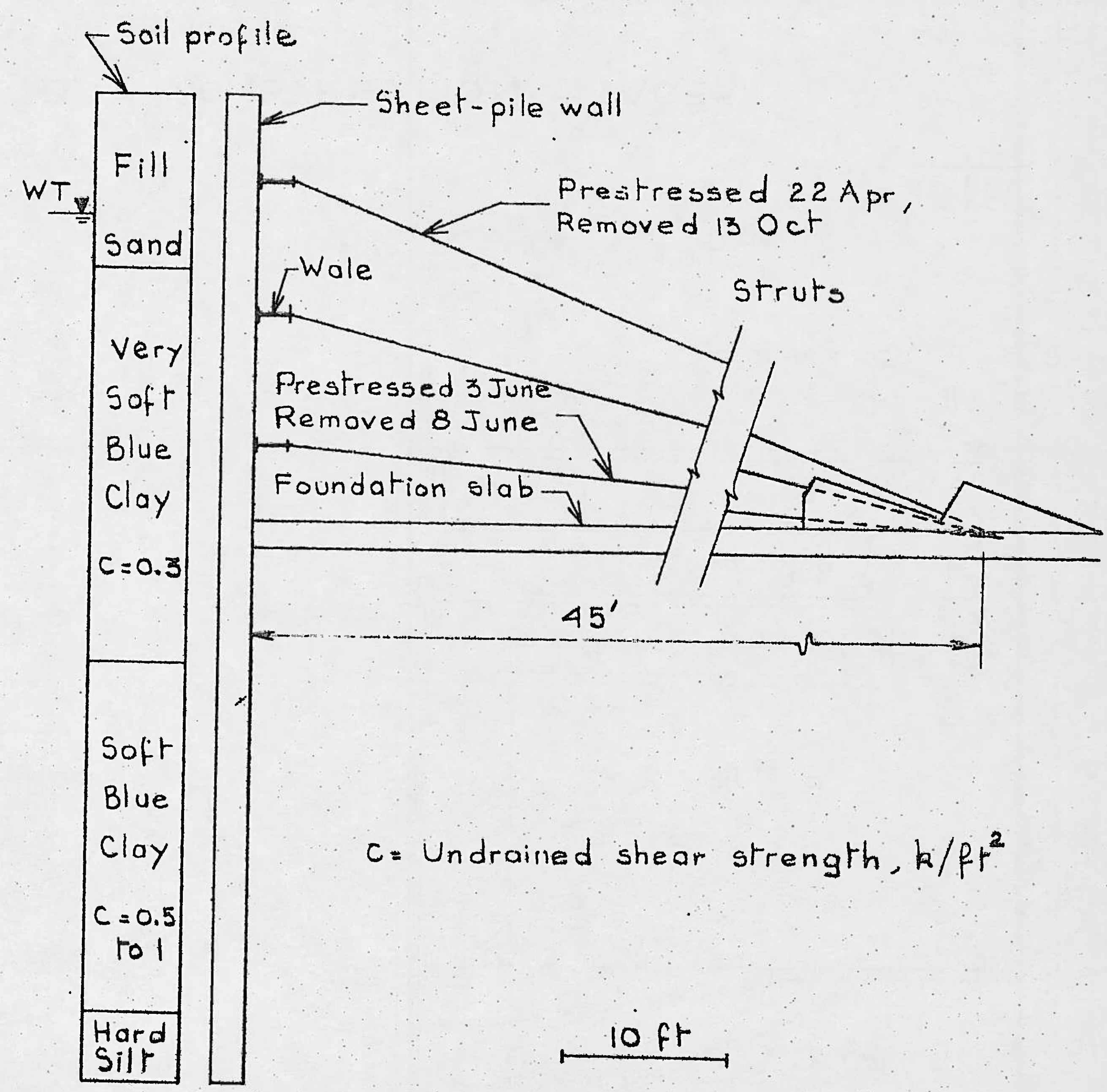
d. Contact Sheeting (after Peck 1969)

METHODS OF TRANSFERRING EARTH PRESSURE FROM LAGGING TO SOLDIER PILES

Fig. 8


Optical survey
22 Apr
14 Oct
3 June
9 June
Initial position
24 Oct 63 to 26 Mar 64
22 Apr 64
3 June 64
9 June 64
14 Oct 64
Elevation in Feet
+10
0
-10
-20
-30
-40
-50
0
1
2
Deflection, in

a.


Soil profile
Fill
WT
Sand
Very Soft Blue Clay
C=0.3
Soft Blue Clay
C=0.5 to 1
Hard Silt
Sheet-pile wall
Prestressed 22 Apr, Removed 13 Oct
Wale
Struts
Prestressed 3 June
Removed 8 June
Foundation slab
45'
C= Undrained shear strength, k/ft²
10 ft

b.

η

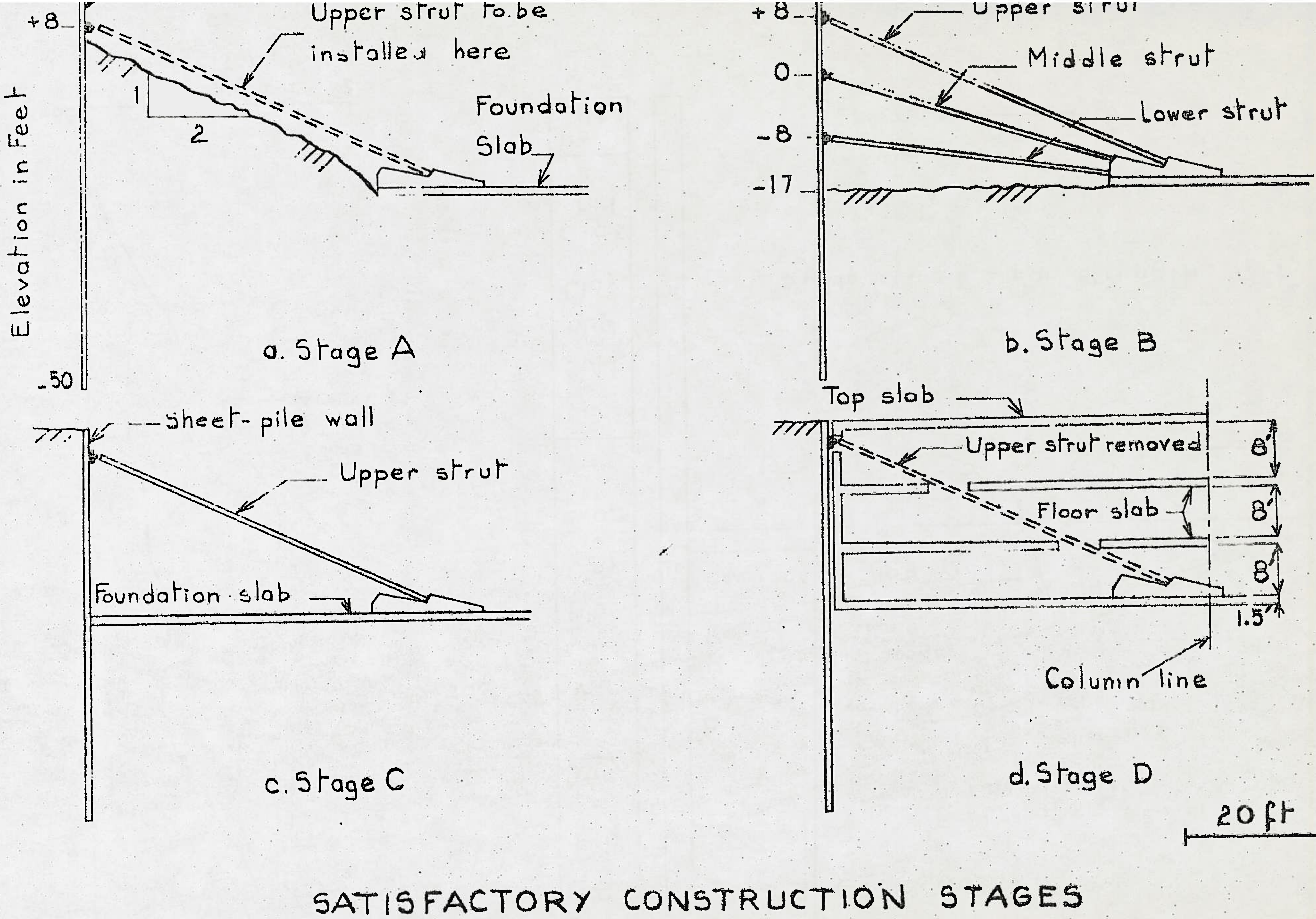
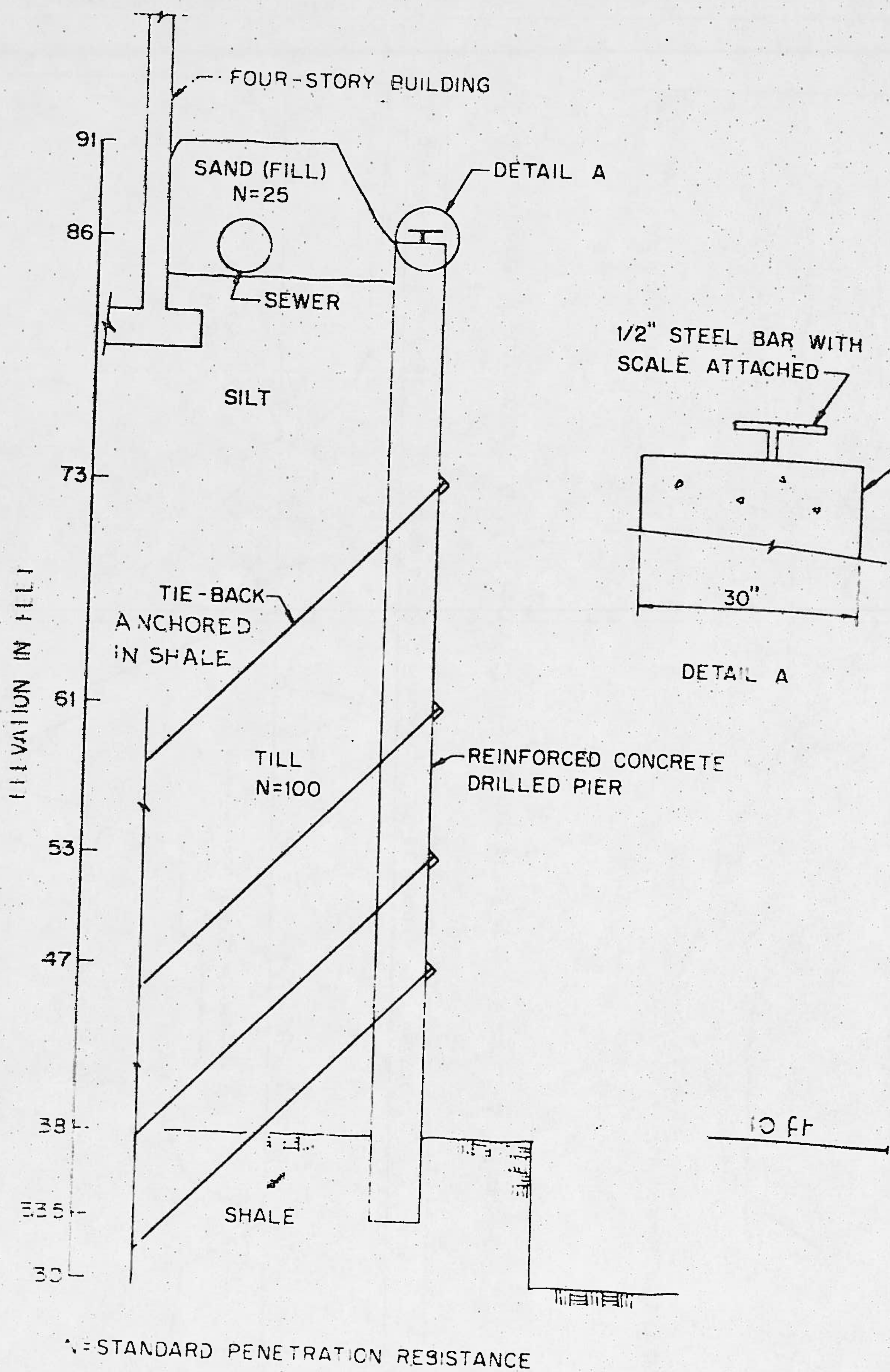
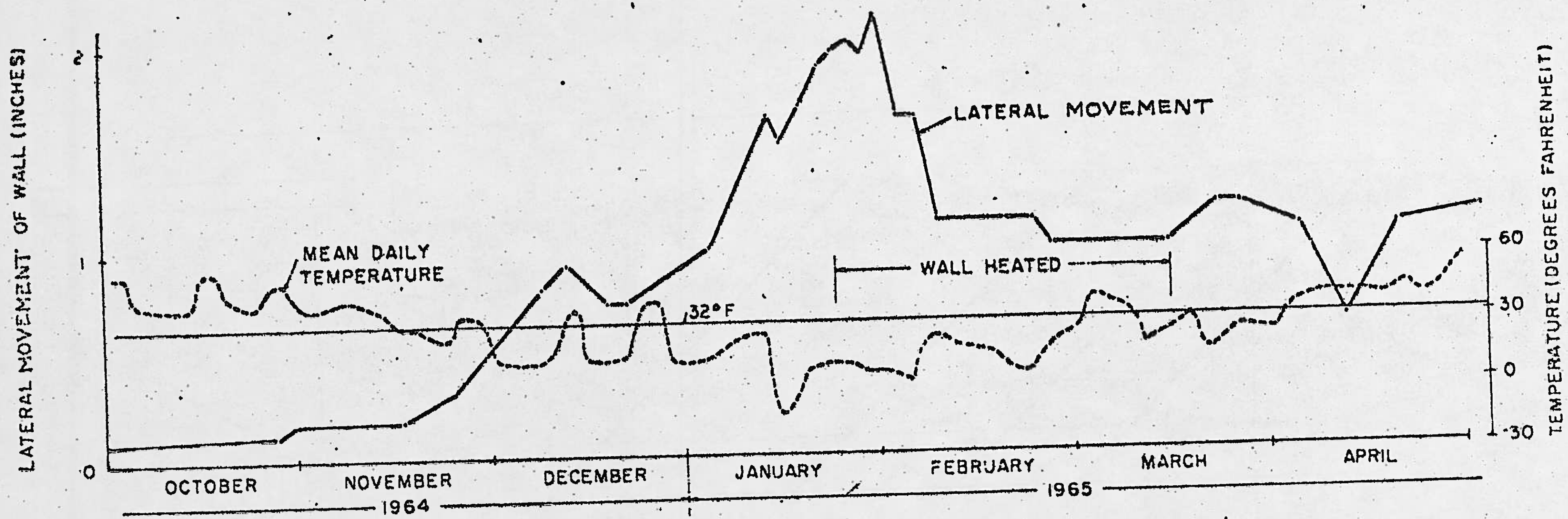

+8
Upper strut to be installed here
1
2
Foundation Slab
Elevation in Feet
-50
a. Stage A
+8
0
-8
-17
Upper strut
Middle strut
Lower strut
b. Stage B
Sheet-pile wall
Upper strut
Foundation slab
c. Stage C
Top slab
Upper strut removed
Floor slab
8'
8'
8'
1.5'
Column line
d. Stage D
20 ft
SATISFACTORY CONSTRUCTION STAGES

Fig. 10

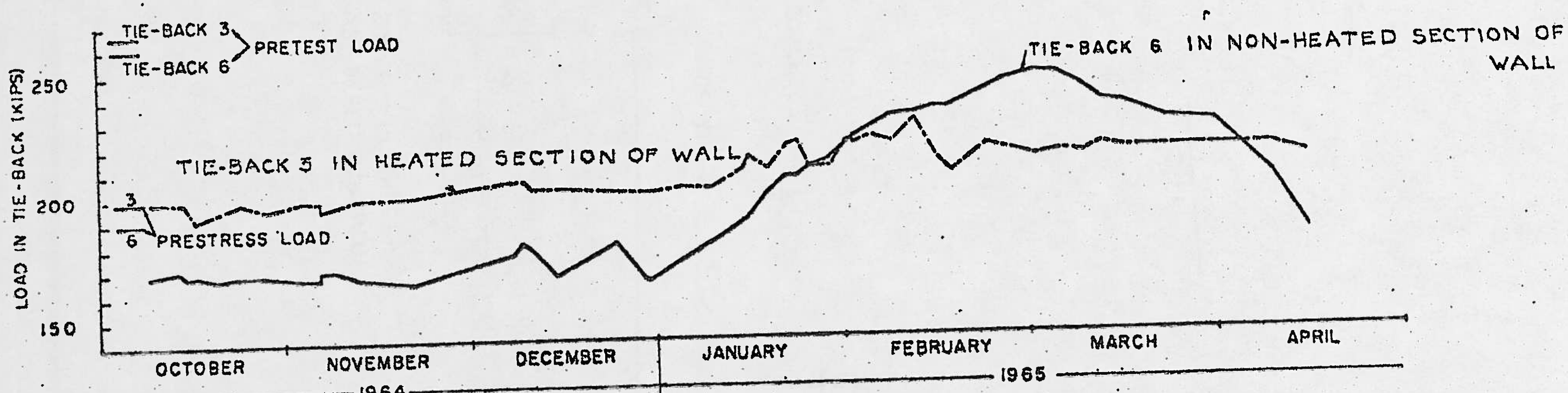


SECTION OF AN ANCHOR WALL

Fig. 11



a. Lateral Movement and Mean Daily Temperature Versus Time



b. Tie-back Load Versus Time

Fig. 12